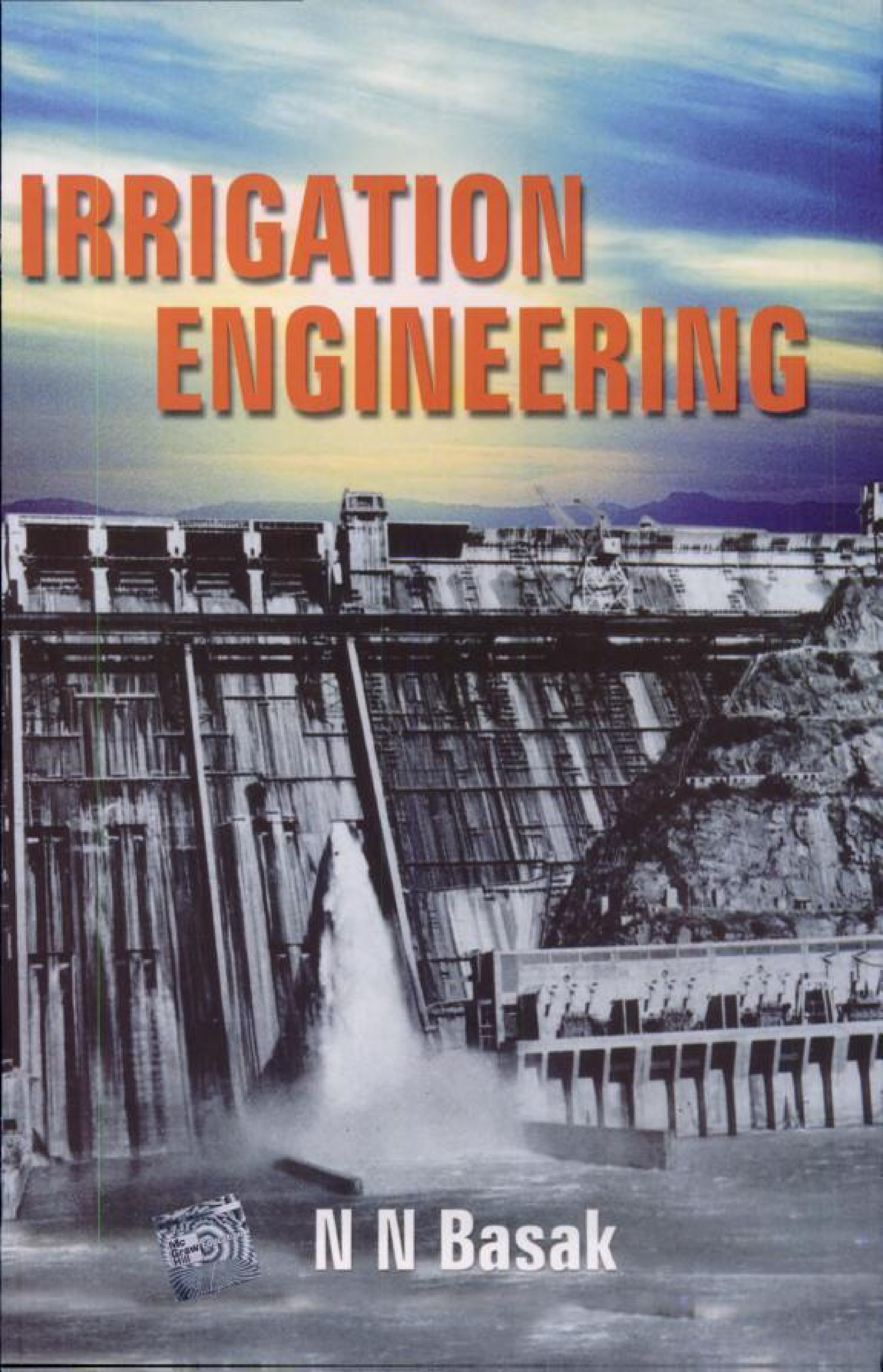
IRRIGATION
ENGINEERING
N N Basak

Irrigation Engineering

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To
my Parents

PREFACE

This book is the outcome of the experience gained by me during the investigation works for 'Minor Irrigation Scheme in Hilly Area' and 'Irrigation Cum Navigation Project' under Irrigation Department, Government of West Bengal and Central Water and Power Commission, Government of India, respectively.

I joined Malda Polytechnic in 1963 as a teacher of 'Field Surveying', 'Irrigation Engineering' and 'Water Resources Engineering. During my teaching years, I felt the students' need for a comprehensive, systematic and user friendly book on Irrigation Engineering. This book is an effort in that direction. About 70 per cent of the syllabus under 'Water Resources Engineering' has also been incorporated in this book.

The book covers the syllabi of diploma, degree and AMIE courses. It will also be helpful to practising engineers and students appearing in various competitive examinations.

Salient Features

- Objective and multiple choice questions are provided at the end of each chapter and in Appendix-A.
- Model questions (with hints) are provided in Appendix-B.
- Every topic is illustrated with self-explanatory figures.
- Numerous worked-out problems are provided to improve the problem-solving efficiency of the students.

I am grateful to the reviewers whose critical suggestions helped me in making major useful revisions in the book. My heartiest thanks to Mr Swapan Chakraborty for preparing the typed copies of the manuscript.

N N Basak

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1

INTRODUCTION

1.1 DEFINITION OF IRRIGATION

The process of artificial application of water to the soil for the growth of agricultural crops is termed as irrigation. It is practically a science of planning and designing a water supply system for the agricultural land to protect the crops from bad effect of drought or low rainfall. It includes the construction of weirs, dams, barrages and canal systems for the regular supply of water to the culturable (i.e. cultivable) lands.

1.2 NECESSITY OF IRRIGATION

Throughout the crop period adequate quantities of water is required near the root zone of the plants for their growth. At times during the crop period the rainfall may not be adequate to fulfil the water requirement. The intensity of rainfall is practically uncertain and beyond the control of human power and it may not be well distributed throughout the crop season or the culturable area. So, irrigation becomes absolutely necessary to fulfil the water requirement of crops. The following are the factors which govern the necessity of irrigation.

(a) Insufficient Rainfall When the seasonal rainfall is less than the minimum requirement for the satisfactory growth of crops, the irrigation system is essential.

(b) Uneven Distribution of Rainfall When the rainfall is not evenly distributed during the crop period or throughout the culturable area, the irrigation is extremely necessary.

(c) Improvement of Perennial Crops Some perennial crops like sugarcane, cotton, etc. require water throughout the major part of the year. But the rainfall may fulfil the water requirement in rainy season only. So, for the remaining part of the year, irrigation becomes necessary.

(d) Development of Agriculture in Desert Area In desert area where the rainfall is very scanty, irrigation is required for the development of agriculture.

1.3 BENEFITS OF IRRIGATION

The following are the important benefits of irrigation:

(a) Yield of Crops In the period of low rainfall or drought, the yield of crop may be increased by the irrigation system.

(b) Protection from Famine The food production of a country can be improved by ensuring the growth of crops by availing the irrigation facilities. This helps a country to prevent famine situation.

(c) Improvement of Cash Crops Irrigation helps to improve the cultivation of cash crops like vegetables, fruits, tobacco, etc.

(d) Prosperity of Farmers When the supply of irrigation water is assured, the framers can grow two or more crops in a year on the same land. Thus the farmers may earn more money and improve their living standard.

(e) Source of Revenue When irrigation water is supplied to the cultivators in lieu of some taxes, it helps to earn revenue which may be spent on other development schemes.

(f) Navigation The irrigation canals may be utilised for inland navigation which is further useful for communication and transportation of agricultural goods.

(g) Hydroelectric Power Generation In some river valley projects, multipurpose reservoirs are formed by constructing high dams where hydroelectric power may be generated along with the irrigation system.

(h) Water Supply The irrigation canals may be the source of water supply for domestic and industrial purposes.

(i) General Communication The inspection road along the canal banks may serve as a communication link with the otherwise remote villages.

(k) Development of Fishery The reservoir and the canals can be utilised for the development of fisher projects.

1.4 ILL-EFFECTS OF IRRIGATION

The following are the ill-effects of irrigation:

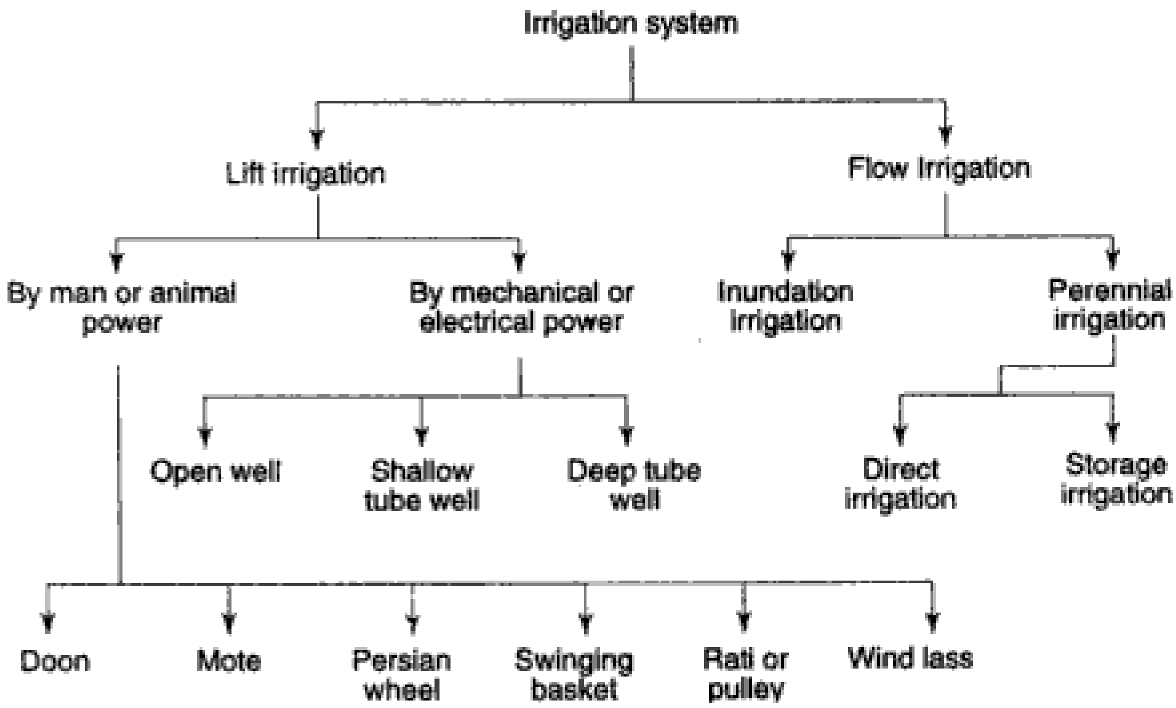
(a) Rising of Water Table Due to the excessive seepage of water through the bed and banks of the canals, the water table in the surrounding area may be raised which may constantly saturate the root zone of the crops and the soil may develop alkaline property which is harmful to the crops.

(b) Formation of Marshy Land Excessive seepage and leakage of water from the irrigation canals may lead to formation of marshy lands along the course of the canals. These marshy lands form the colonies of mosquitos which may be responsible for diseases.

(c) Dampness in Weather The temperature of the commanded area of an irrigation project may be lowered considerably and the area may become damp. Due to dampness, the people residing around the area may suffer from cold, cough and other such diseases originating from dampness.

(d) Loss of Valuable Lands Valuable land may get submerged when storage reservoirs are formed by constructing barrages or dams and, it also may be lost while constructing irrigation canals.

1.5 SYSTEMS OF IRRIGATION



A. Lift Irrigation

When water is lifted from surface sources or underground sources by man or animal power, mechanical or electrical power and directly supplied to the agricultural land, then it is known as *lift irrigation*. In this method isolated small areas can be irrigated. The vast areas cannot be included in this system. Lift irrigation can be divided into two groups:

(a) Lifting of water by man or animal power.
(b) Lifting of water by mechanical or electrical power.

When mechanical or electrical powers are not available in villages or the economic condition of the farmers is not good enough to afford this expensive method, the lifting of water is done by the following method from the surface sources (like ponds, lakes, rivers, etc):

(i) Doon
(ii) Mote
(iii) Persian wheel
(iv) Swinging busket
(v) Dhenkli
(vi) Rati or pulley
(vii) Wind lass

When mechanical or electrical power is available in villages or the farmers can afford the expenditure for the installation of the same, the underground water is lifted by pumps (diesel pumpset or electrical pump set) and directly supplied to the agricultural land. The underground water may be available from the following sources:

(i) Open well
(ii) Shallow tube well
(iii) Deep tube well.

Now-a-days, the pumping system of lift irrigation from shallow or deep tube well is widely practised.

(Details of lift irrigation systems will be studied in Chapter 4.)

Advantages of Lift Irrigation

The following are the advantages of lift irrigation

(i) The farmers can supply water to their fields according to the requirement, and hence there is no possibility of over irrigation.
(ii) The water table is lowered when water is lifted from the wells thereby reducing chances of water logging in the area.
(iii) As water is supplied directly to the fields, there is no water loss due to conveyance.
(iv) Initial cost is low as there is no necessity of constructing hydraulic structures.
(v) As the loss of water is low, the duty of water is high.
(vi) The maintenance cost is low.
(vii) More than one crop can be grown in a year on the same land.
(viii) Loss of valuable land is prevented as there is no necessity of constructing the network of canals.
(ix) The water of the well is cooler in hot season and warmer in cold season. This phenomenon is favourable for the crops.

Disadvantages of Lift Irrigation

The following are the disadvantages of lift irrigation

(i) In summer the surface water may be dried up and the water table may go down below the suction head. Hence, the lift irrigation from the surface source and from the shallow tube well may fail in summer.
(ii) If the lifting mechanism (i.e. pump) fails due to mechanical or electrical failure, then water cannot be supplied until the mechanism is restored.

(iii) The well water has no silt content. The yield of crop therefore depends on chemical fertiliser, which is costly.

(iv) The lift irrigation in consideration with the yield of crop is not cost effective.

B. Flow Irrigation

When water flows under gravitational pull through the artificial canal towards the agricultural land, it is termed as *flow irrigation.* In this system, the head of the canal should always be at higher elevation than the land to be irrigated. The following are the different systems of flow irrigation.

1. Inundation Irrigation System

In this system, a canal is excavated from the bank of the inundation river (i.e. the river which overflows in rainy season but nearly dried up in summer and winter) [Fig. 1.1 (a)].

In this case water flows to the agricultural land in rainy season only. There is no regulator at the head of the canal to control the flow of water. The bed level of the canal is fixed at such level that the water can flow through the canal only when the water level of the river rises above the canal bed [Fig. 1.1(b)].

Again, the flow of water through the canal stops automatically when the water level of the river falls below the canal bed [Fig. 1.1(c)]

So, this system of irrigation depends completely on the water level of the river. As there is no regulator at the head of the canal, over irrigation is possible resulting in damaging the crops.

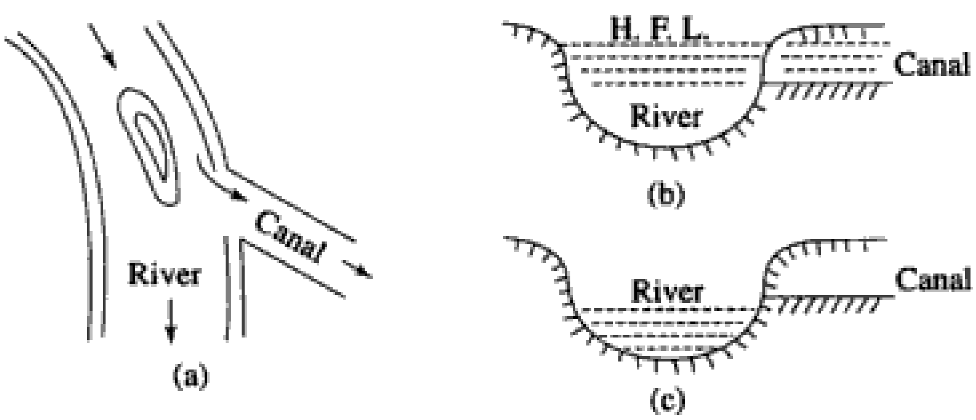


Fig. 1.1 Inundation canal

2. Perennial System of Irrigation

In this system, a weir or a barrage is constructed across the perennial river (i.e., the river which flows throughout the year in its full capacity) to raise the water level on the up stream side or a dam is constructed to form a storage reservoir. Then main canal is constructed on either or both the banks of the river. Regulator is constructed at the head of the canal to control the flow of water through the canal towards the agricultural land. This system is reliable as water is available throughout the year. The perennial system of irrigation may be of the following types:

(a) Direct Irrigation System In this system, a weir is generally constructed across a perennial river to raise the water level on the up stream side up to a certain limit, so that the water can flow through the canal. Here, the water level on the up stream side will remain at a constant height and the excess water flows over the weir. Sometimes, a barrage is constructed, in place of weir, to regulate the water level on the upstream side. The hydraulic structure which is constructed in direct irrigation system is known as Diversion Head Works (Fig. 1.2).

(b) Storage Irrigation System In this system a dam is constructed across a river valley to form a storage reservoir (Fig. 1.3). The main canals may be taken from both sides of the dam. The flow of water through the canal is controlled by head regulator. This storage reservoir is also known as multipurpose reservoir as it serves the following purposes:

(a) Irrigation
(b) Water supply
(c) Hydro-electric power generation
(d) Fishery
(e) Flood control.

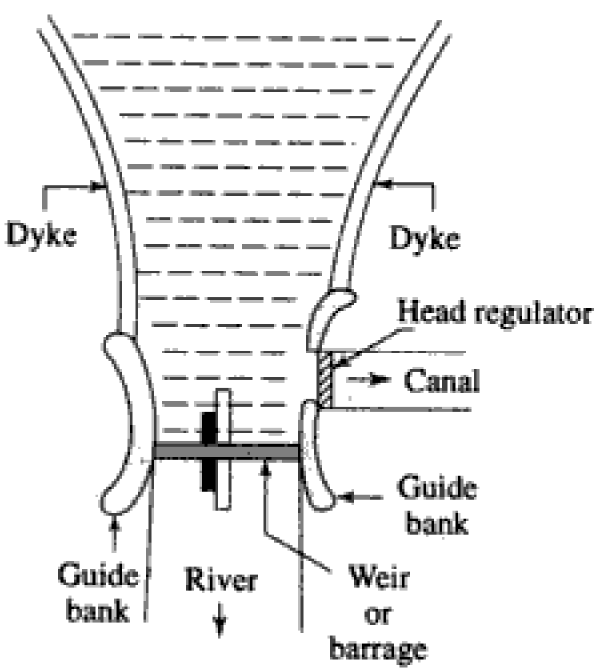


Fig. 1.2 Diversion head works

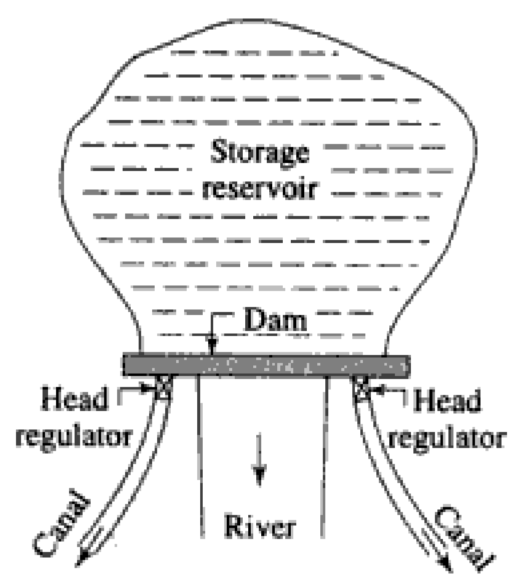


Fig. 1.3 Storage reservoir

1.6 DIFFERENCE BETWEEN WEIR, BARRAGE AND DAM

Weir An impervious barrier which is constructed across a river to raise the water level on the up stream side is known as weir. Here the water level is raised up to the required height and the surplus water is allowed to flow over the weir. Generally it is constructed across a inundation river (Fig. 1.4).

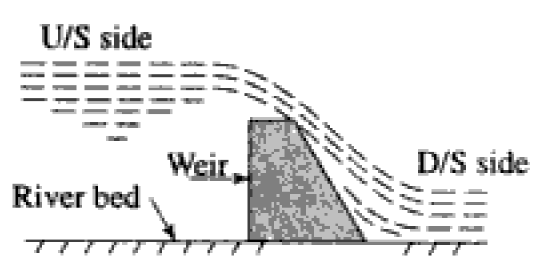


Fig. 1.4 Weir

Barrage When adjustable gates are installed over a weir to maintain the water surface at different levels at different times, then it is known as barrage. The water level is adjusted by operating the adjustable gates or shutters. The gates are placed at different tiers and these are operated by cables from the cabin. The gates are supported on the piers at both ends. The distance between pier to pier is know as Bay (Fig. 1.5).

Dam The high impervious barrier constructed across a river valley to form a deep storage reservoir is known as *dam*. The surplus water is not allowed to flow over the dam, but it flows through the spillways provided at some designed level (Fig. 1.6).

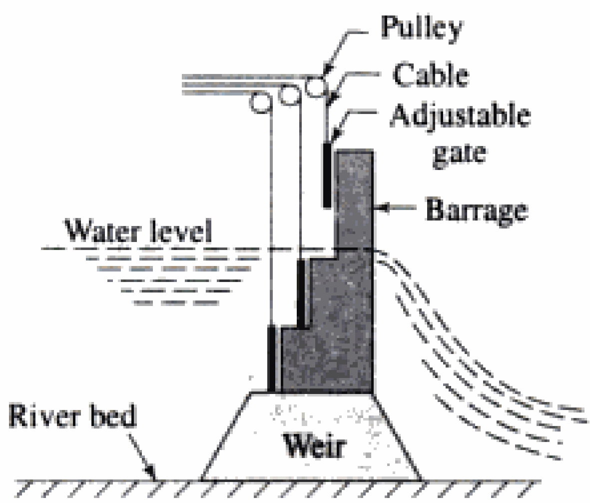


Fig. 1.5 Barrage

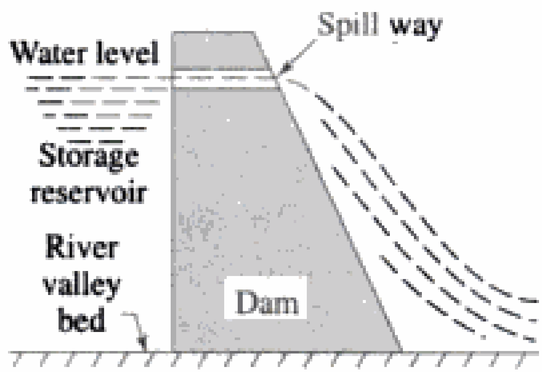
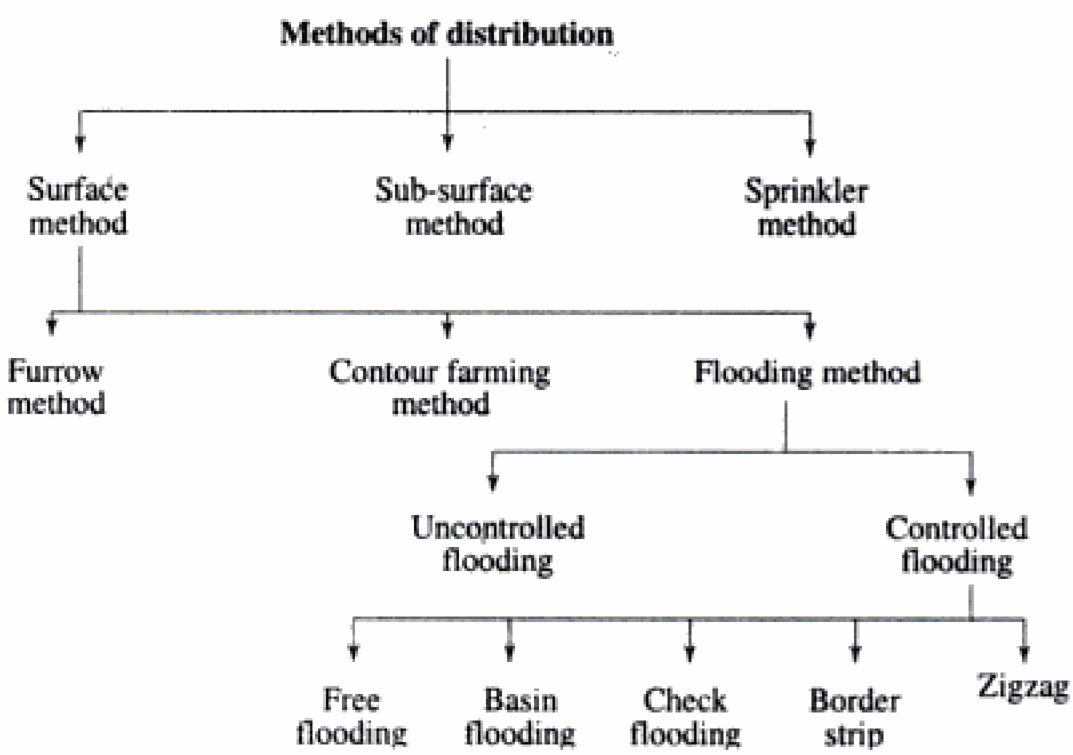


Fig. 1.6 Dam

1.7 METHODS OF DISTRIBUTION OF WATER



A. Surface Method

In this method, the irrigation water is distributed to the agricultural land through the small channels which flood the area up to the required depth. The surface method is again sub-divided into three categories:

(a) Furrow Method

In this method, the irrigation water is supplied to the land by digging narrow channels known as furrows at regular intervals. The water flows through the furrows and infiltrates into the soil and spreads laterally to saturate the root zone of the crops. This method is suitable for the crops which are sown in rows. The crops are potato, ground nut, tobacco, sugarcane, etc (Fig. 1.7).

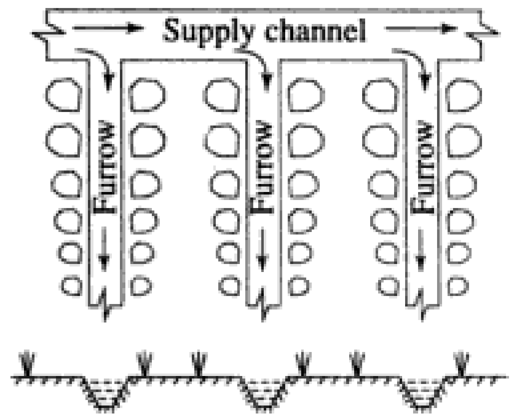


Fig. 1.7 Furrow method

(b) Contour Farming

This method is adopted in hilly areas where the land has steep slope. Here, the land is divided into series of horizontal strips which are known as terraces. Small bunds are provided at the end of each terrace to hold water upto the required depth.

This method serves also the purpose of flood control and soil erosion (Fig. 1.8).

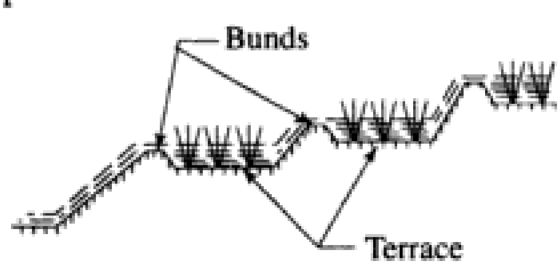


Fig.1.8 Contour farming

(c) Flooding Method

This method is suitable for the agricultural land which exists in flat topography. In this method, the field is flooded with water with the help of field channels. The flooding method may be of two types.

(1) Uncontrolled flooding This method is applicable in inundation irrigation system. Here, the land is flooded with water by inundation canal. As there is no controlling system in inundation canal, this type of distribution of water is known as uncontrolled flooding. This method results in wastage of water and over irrigation.

(2) Controlled Flooding This method is applicable in perennial irrigation system. In this method, the agricultural area is flooded with water through the canals which are provided with regulators. It is again sub-divided into following types-

(a) Free flooding
(b) Basin method
(c) Check flooding

(d) Border flooding
(e) Zig-zag method

(a) Free Flooding In this method, the agricultural land is divided into small strips by a series of field channels which are connected to the supply channel. The strips of land are flooded with water by opening the field regulators (F.R.) the surplus water flows through the waste water channel and is discharged into the river or drainage (Fig. 1.9).

(b) Basin Method This method is employed for watering orchards. In this method, each tree or a group of trees are enclosed by circular channel through which water flows. The circular channel is known as basin. Each basin is connected to field channel. The field channel is again connected to the supply channel. When all the basins are filled with water, the supply of water is stopped (Fig. 1.10).

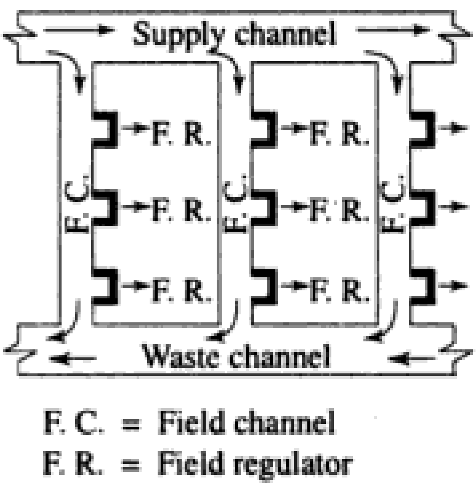


Fig. 1.9 Free flooding

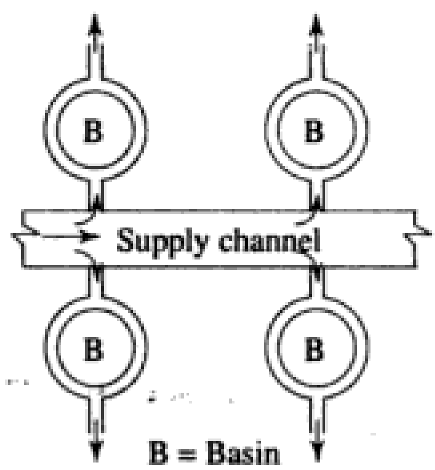


Fig. 1.10 Basin method

(c) Check Flooding In this method, the agricultural area is divided into small plots (known as check basins) by check bunds. The water is supplied to the check basins through the field channels which are connected with the supply channel. Each basin is flooded with water to the desired depth and the water is retained for some hours so that it can infiltrate into the soil (Fig. 1.11).

(d) Border Strips In this method, the agricultural area is divided into series of long narrow strips (known as Border strips) by levees, i.e. small bunds. The strips are aligned along the country slope so that the water can flow easily throughout the area. This method is suitable when the area is at level with gentle country slope.

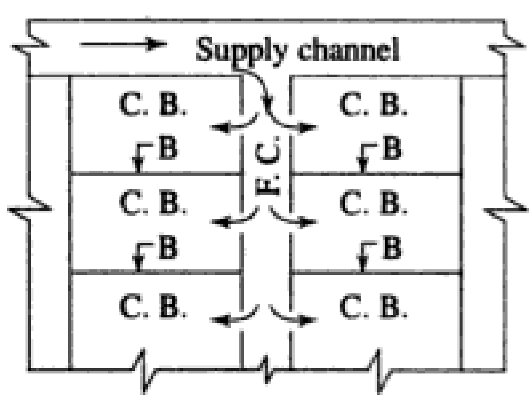


Fig. 1.11 Check Flooding

(e) Zig-zag Method In this method, the agricultural area is sub-divided into small plots by low bunds in a zig-zag manner. The water is supplied to the plots from the field channel through the openings. The water flows in a zig-zag way to cover the entire area. When the desired depth is attained, the openings are closed (Fig. 1.13).

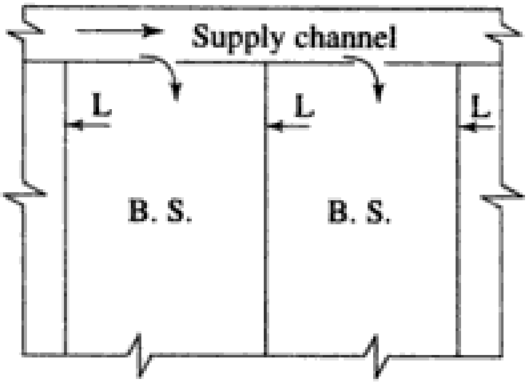


Fig. 1.12 Border strips

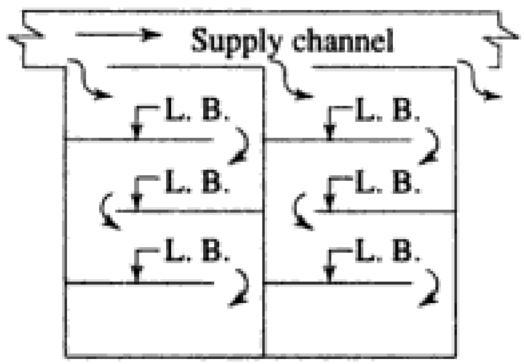


Fig. 1.13 Zig-zag method

B. Sub-Surface Method

In this method, the water is applied to the root zone of the crops by underground network of pipes. The network consists of main pipe, sub-main pipes, and lateral perforated pipes. The perforated pipes allow the water to drip out slowly and thus the soil below the root zone of the crops absorbs water continuously. This method is suitable for permeable soil like sandy soil. This method is also known as drip method or trickle method of irrigation (Fig. 1.14).

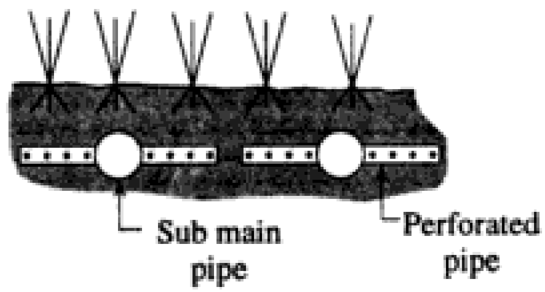


Fig. 1.14 Sub-surface method

C. Sprinkler Method

In this method, the water is applied to the land in the form of spray like rain. The spraying of water is achieved by the network of main pipe, sub-main pipes and lateral pipes.

The lateral pipes may be perforated at the top and sides through which the water comes out in the form of spray and spreads over the crop in a particular area. Again, the lateral pipes may contain series of nozzles through which the water comes out as fountain and spreads over in a particular area.

Now-a-days, the lateral pipes consists of riser pipes with rotating arms at the top. The arms are fitted with nozzles. So, the water gets distributed on a circular area when the arms are rotated on the vertical axis by electrical motor coupled with belt and Pulley System. The network of pipe lines are supported on pillars and the water is forced through the pipe lines by pumping unit. The following are different forms of sprinklers

(a) Perforation on lateral pipes
(b) Fixed nozzles on lateral pipes
(c) Rotating sprinklers.

(a) Perforation on Lateral Pipes

In this type, the lateral pipes are perforated along the top and sides. The water is sent under pressure by a pumping unit through the main pipe, sub-main pipes and lateral pipes. The water comes out through the perforations in all directions in the form of spray. The spacing of lateral pipes should be such that the whole area may be evenly sprayed with water. The lateral pipes are supported on pillars (Fig. 1.15).

(b) Fixed Nozzles on Lateral Pipes

In this type, a series of nozzles are fixed along the lateral pipes. The spacings of the nozzles are such that the water may cover the whole area evenly. The lateral pipes are supported on pillars. When the water is forced under pressure through the network of pipes, it comes out as fountain through the nozzles and spreads over the land (Fig. 1.16).

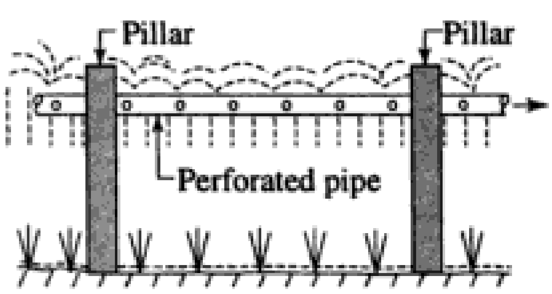


Fig. 1.15 Perforated lateral pipes

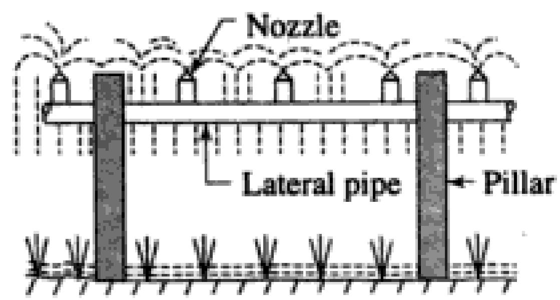


Fig. 1.16 Fixed nozzles

(c) Rotating Sprinkless

In this type, the riser pipes are fixed on the lateral pipes at a regular intervals. On the top of the riser pipe are two arms which can rotate about a vertical axis. The upper ends of the arms consists of nozzles. When the water is forced under pressure through the main, sub-main and lateral pipes, it rises up and comes out through the nozzles in the form of spray. As the arms rotate, a circular area is covered by each riser (Fig. 1.17).

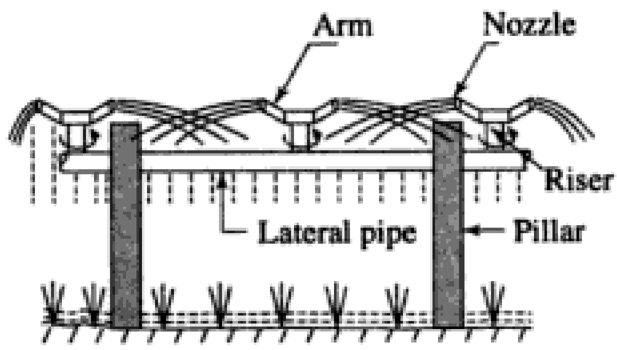


Fig. 1.17 Rotary sprinklers

1.8 IRRIGATION PROJECT SURVEYING

1. Availability of Irrigation Water

When it is found necessary to take up an irrigation project, the availability of required water should be investigated. The following points should be considered

(a) Whether any perennial river is available near the command area or not.
(b) If an inundation river is available, the maximum discharge of that river is to be ascertained from the highest flood level mark (as indicated by the villagers residing near the bank of the river).
(c) From various investigations (i.e. maximum discharge, rainfall etc.) it is necessary to ascertain whether the river will be able to meet the total water requirement or not.

2. Selection of Probable Site for Barrage or Dam

When the source of water is available, the suitable site for the barrage or dam should be found out considering the following points,

(a) The course of the river should be straight at least for a distance of about one kilometre both on the upstream and down-stream side of the site.
(b) The width of the river should be minimum and the section of the river should be well-defined.
(c) A suitable basin should be available for the storage reservoir.
(d) The elevation of the site should be higher than that of the culturable command area.
(e) The storage reservoir should not submerge much valuable land.
(f) The capacity of the reservoir should fulfil the total water requirement.

3. Discharge Observation for the River

The gauge and discharge observation station should be established at the proposed site to collect the following data,

(a) The daily discharge, maximum discharge and minimum discharge of the river throughout the year should be recorded.
(b) Silt analysis should be carried out in rainy season (when the river carries much silt) to determine the nature of sedimentation in river or reservoir.
(c) Discharge observations should also be carried out for all the rivers crossing the proposed canal. This is required for designing cross-drainage works.

4. Marking of Gross Command Area and Culturable Area

When it is decided to take up the project, the gross command area should be marked on the topographical map. The culturable areas should be defined on the map to find the culturable command area that is to be included in the project.

5. Marking of Alignment of Main Canal

The alignment should be marked on the topographical map of the concerned area. While marking the alignment the following points should be kept in mind

(a) The alignment of the main canal should be taken in such a way so that unnecessary cutting and banking is avoided.
(b) The alignment of the main canal should be such that the branch canals can be taken suitably to cover the whole culturable area.

(c) The alignment should cross the rivers, roads, railways lines etc. perpendicularly as much as possible.
(d) The alignment should not be taken through the valuable agricultural land.
(e) The alignment should not pass through the thickly populated areas, religious places, burial grounds, etc.

6. Preliminary Location Survey

The reconnaissance survey should be carried out along the alignment to record necessary data such as obstacles, road crossings, railway crossings, river crossings, etc. This survey involves the following procedures.

(i) The approximate distance along the alignment should be measured by pacing and the magnetic bearings of the traverse legs (open traverse) and it should be noted in the field book.
(ii) The objects and the nature of the ground on both sides of the alignment should also be noted in the field book.
(iii) The alignment may be diverted to avoid religious places, valuable structures, etc.
(iv) The alignment should be made to cross the rivers perpendicularly.
(v) An index map should be prepared for the alignment.

7. Final Survey

(a) Final Location of Barrage or Dam Site The final location of the barrage or dam site involves the following steps.

(i) The centre line of the barrage or dam site should be marked with pillars on both banks of the river.
(ii) The cross-section of the barrage site should be taken very precisely.
(iii) Cross-sections should be taken at regular interval on the upstream side of the barrage site to ascertain the storage capacity of the reservoir.
(iv) Boring test should be carried out along the centre line of the barrage site to determine the depth and nature of foundation.

(b) Route Survey A prismatic compass survey or planetable survey should be conducted along the alignment of the main canal to prepare a route survey map of the area covering a distance of about 30 m on both sides of the alignment.

(c) Longitudinal Levelling The longitudinal levelling should be done along the alignment of the main canal. Generally, the staff readings are taken at an interval of 20 m along the centre line of the main canal. The magnetic bearings of the lines (traverse legs) should also be noted in the level book. Longitudinal levelling for the branch canals should also be done.

(d) Cross-Sectional Levelling The cross-sectional levelling at regular intervals along the alignment of the main canal should be taken. The cross-sections for the branch canals also should be taken. These cross-sections are required for the computation of volume of earth work.

(e) Data for Cross-Drainage Work At the places of river crossings, road crossings, railway crossings etc. additional data should be collected for designing cross-drainage works. At the sites of river crossings the gauge and discharge observation stations should be established.

(f) Soil Survey The soil survey should be conducted along the alignment. It consists of collecting the sample of soil by boring up to the depth until impervious layer is obtained.

(g) Well Observation Well observation should be carried out along the alignment. This operation consists of measuring the water level of the wells existing on both sides (within 50 m) of the alignment. This is done to know the nature of water table along the course of the canal.

8. Preparation of Drawings

(i) Route survey map (to suitable scale).
(ii) Longitudinal sections for the main and branch canals with formation level (to suitable scale)
(iii) Cross-sections of main and branch canals with formation level (to suitable scale).
(iv) Contour map along the alignment.
(v) Design of curves with setting out table.

9. Office Works

(i) The sections of the canals should be designed.
(ii) The detailed estimate should be prepared to know the volume of earth work in cutting or banking along the main canal and branch canals.
(iii) The total land width required should be marked on the route survey map.
(iv) The design of the barrage or dam, cross-drainage works and other allied structures should be completed.
(v) The detailed report should be prepared for the compensation. It includes the names of owners, location, amount of properties, valuation of the land, etc.
(vi) The total cost of the project should be ascertained by considering all the aspects.

10. Justification of the Selection of Final Alignment

After preliminary survey, the estimates for the tentative alignments (if taken) are prepared. Then by comparing the total costs, working feasibility, etc. with the alignments the final alignment is selected.

11. Final Location Survey

The final location survey of the approved alignment of the canal should be carried out for the execution of the project works. It includes the following:

(i) The centre line of the main and branch canals should be marked with concrete pillars at an intervals of 30 m or 50 m.

(ii) The total land width required for the main and branch canals should be marked with pillars at suitable intervals.

1.9 IRRIGATION PROJECT REPORT

1. Introduction

The introduction of the project includes the following points:

(a) Aim of the project.
(b) Location of the project.
(c) Total area to be covered within the project.
(d) Total population to be benefitted by the project.
(e) Future prospect if irrigation is practised.
(f) Stages of future development.
(g) Total cost of the project.

2. Necessity and Economic Justification of the Project

To justify the necessity and economical development of the area, the following points should be clearly illustrated

(a) Amount of yearly rainfall.
(b) Nature of distribution of rainfall during the crop season.
(c) Types of major crops grown in the area.
(d) Total water requirement of the crops.
(e) Amount of water requirement by irrigation system.
(f) Expected increase in yield of crops, if irrigation is practised.
(g) Total revenue expected.

3. Report on Land Acquisition and Compensation

A detailed statement should be prepared showing the names of owners, types of properties, quantity, amount of compensation, etc. The procedure adopted for the land acquisition should be clearly mentioned.

4. Details of Design and Drawing of Hydraulic Structures

The detailed design procedure and drawing of hydraulic structures, canals and other allied structures should be incorporated.

5. Detailed Estimate

The detailed estimate for all the works of the project should be incorporated.

6. Specification

The specifications of the construction materials and different works should be clearly mentioned.

7. Availability of Materials and Labourers

The source of construction materials and places of recruitment of labourers should be mentioned.

8. Communication

The existing communication to the selected barrage or dam site should be pointed out. If new communication is required for inaccessible site, the possible route should be pointed out and the expenditure for the new route should be included in the project.

9. Maps to be Submitted

(a) Topographical map of the area showing the canal alignment and barrage or dam site.
(b) Route survey map.
(c) Longitudinal sections.
(d) Cross-sections.
(e) Contour map of alignment.
(f) Detailed drawing of barrage or dam, cross-drainage works, etc.

10. Conclusion and Recommendation

After furnishing all the aspects of the project, the proposal is forwarded to the higher authority with proper recommendation for the necessary approval.

1.10 IMPORTANT IRRIGATION PROJECTS IN INDIA

1. Bhakra Nangal Project

It is a multipurpose project for irrigation and hydro-electric power generation. It consists of Bhakra dam which is 518 m long and 226 m high. The dam is constructed across the river Sutlej in Bilaspur district of Himachal Pradesh. The storage reservoir is known as Govind Sagar reservoir. The area of the reservoir is about 150 sq. km. A barrage is constructed at Nangal which is 305 m long and 29 m high. The main canal is of length 174 km and the culturable command area is 15 lakh hectares in Punjab, Haryana and Rajasthan. The project consists of four hydro-electric power generation stations. Two at the toe of the dam and other two at Gangwal and Kotla. The total power generation capacity of the project is about 1200 MW.

2. Damodar Valley Project

This project is situated in West Bengal. It is a multipurpose project for irrigation, navigation, flood control and hydro-electric power generation. The overall development of the Damodar Valley is achieved by the components described here.

(a) Tilaiya Dam It is about 365 m long and 30 m high constructed across the river Barakar.

(b) Konar Dam It is about 3550 m long and 49 m high constructed across the river Konar.

(c) Maithan Dam It is also constructed across the river Barakar. The length of the dam is about 1005 m and the height is about 35 m.

(d) Panchet Dam It is constructed across the river Damodar. The height and the length of the dam is about 30 m and 2570 m respectively.

(e) Durgapur Barrage It is 672 m long and 12 m high constructed across the river Damodar. An irrigation cum navigation canal of length 137 km is taken off from the left bank. A canal of length 88 km is taken off from the right bank for irrigation only.

The power generation capacity of the project was initially 105 MW. Now, the capacity has been increased stage by stage. The total culturable command area is about 3.75 lakh hectares.

3. Farakka Barrage Project

This barrage is situated in West Bengal. It is constructed across the river Ganges at Farakka in Murshidabad district. It is a part of the main project "Ganga–Bramhaputra navigation cum irrigation project." A right bank feeder canal has been taken off from the barrage to connect the river Bhagirathi to keep it navigable throughout the year. This barrage serves an important role for roadway and railway communication with West Bengal, Bihar and Assam. The proposed link canal will be excavated from Mahananda barrage (at Sonapur) to Farakka barrage for the purpose of irrigation and Navigation.

4. Gandak Project

This is situated in Champaran district of Bihar just close to India-Nepal border. A barrage of length 740 m and height 10 m has been constructed across the river Gandak. It is a multipurpose project for irrigation and hydro-electric power generation. The total culturable area is about 15 lakh hectares. The hydro-electric power generation capacity is 15 MW.

5. Godavari Barrage Project

This project is situated in Andhra Pradesh. A barrage of length 3600 m with 174 number of bays has been constructed across the river Godavari. A feeder canal has been taken from the barrage to feed Godavari delta canal system to irrigate an area of about 5 lakh hectares.

6. Hirakud Project

The project is situated in Orissa. It consists of a dam of length 4800 m and height 60 m constructed across the river Mahanadi at Sambalpur district. The culturable area of the project is about 2.5 lakh hectare The hydro-electric power generation capacity is about 270 MW.

7. Mahanadi Delta Project

This project is situated in Orissa. It is executed in two stages.

First Stage It consists of the remodelling work of the following

(a) A barrage has been constructed across the river Mahanadi near Cuttack. This is a remodelling work of an old weir.

(b) Another barrage has been constructed across the river Birupa which is also a remodelling work of an old weir.
(c) A link canal has been taken off from the barrage to connect existing canal system to irrigate an area of about 3 lakh hectares.

Second Stage It is completely a new project. In this stage a weir is constructed across the river Mahanadi at Mundali to irrigate an area of about 5.50 lakh hectares.

8. Mayurakshi Project

This project is situated in West Bengal. It consists of a dam of length 640 m and height 47 m constructed across the river Mayurakshi. A barrage has been constructed across the same river a few kilometre down stream of the dam. The total culturable area of the project is about 3 lakh hectares.

9. Kangsabati Project

This project is situated in West Bengal. It consists of two dams. One has been constructed across the river Kangsabati and the other has been constructed across the river Kumari both in Bankura district. These two dams have been connected by a dyke to form a single reservoir. The total culturable area of this project is about 4 lakh hectares.

10. Kosi Project

The project is situated in Bihar. A barrage of length about 1144 m has been constructed across the river Kosi on India–Nepal border. Eastern Kosi canal system irrigates a vast area of India and Nepal. A hydro-electric power generation station of capacity 25 MW has been installed. Western Kosi canal system has been included in this project at a later stage.

11. Tungabhadra Project

It is situated in Karnataka. A masonry dam of length 2440 m and height 50 m has been constructed across the river Tungabhadra in Bellary district. It is a multipurpose project for irrigation and hydro-electric power generation. The total culturable area is about 3.5 lakh hectares. The total power generation capacity is about 100 MW.

12. Ukai Project

It is situated in Gujarat. A composite dam of length 4928 m and height 69 m has been constructed across the river Tapi near Ukai village in Surat district. It is a multipurpose project for irrigation. Flood control and hydro-electric power generation. The total culturable area is 2 lakh hectares. The hydro-electric power generation capacity is 75 MW.

REVIEW QUESTIONS

1. Fill up the blanks with appropriate word/words
   - (i) For the growth of the crops ___________ of water should always be required.
   - (ii) When the seasonal rainfall ___________ the minimum requirement for the growth of crops, the irrigation is necessary.
   - (iii) When the seasonal rainfall is ___________ distributed during the crop period, the irrigation is necessary.
   - (iv) In desert area the rainfall is very ___________.
   - (v) Vegetable, fruits, tobacco, etc. are known as ___________
   - (vi) The root zone of crops and the soil may get ___________ due to excessive seepage of water through canal banks.
   - (vii) Seepage and leakage of water from the irrigation canal may form ___________.
   - (viii) In lift irrigation ___________ areas can be irrigated.
   - (ix) When the water level of a river is to be raised to some required height, then a ___________ is constructed.
   - (x) When the water of the river is required to be fixed at different levels at different times, then a ___________ is constructed.
   - (xi) When a storage reservoir is to be formed, then a ___________ is constructed.
   - (xii) For the irrigation of potato, groundnut, sugarcane the ___________ of distribution of water is adopted.
   - (xiii) In hilly area, the ___________ method of distribution of water is adopted.
   - (xiv) For the irrigation of orchard, the ___________ is adopted.
   - (xv) The sub-surface method of distribution of water is suitable for the lands consisting of ___________.
2. Define irrigation, state the points to be considered while recommending the necessity of irrigation.
3. State the advantages and disadvantages of irrigation.
4. Differentiate between the lift irrigation and flow irrigation.
5. Name the methods of distribution of water adopted for the following crops and describe the methods with sketch
   - (a) Potato
   - (b) Orchard
   - (c) Paddy (in plane terrain)
6. What are the different types of sprinkler method of distribution of water? Describe briefly, with a neat sketch.
7. Write short notes on any four of the major irrigation projects in West Bengal.
8. Write short notes on any five of the major irrigation projects in India.

9. Enumerate the procedure of irrigation project survey.
10. State, briefly, the procedure of preparation of irrigation project report.

ANSWERS

1. (i) adequate quantities
(ii) is less than
(iii) not evenly
(iv) scanty
(v) cash crops
(vi) alkaline property
(vii) marshy lands
(viii) isolated small
(ix) weir
(x) barrage
(xi) dam
(xii) furrow method
(xiii) contour farming
(xiv) Basin method
(xv) sandy soil

2

WATER REQUIREMENT OF CROPS AND SOIL-WATER RELATIONSHIP

2.1 INTRODUCTION

For proper growth and maturity of the crops, water is of vital importance throughout the crop period. The water requirement may vary from crop to crop, from soil to soil and from period to period. Again, the total water requirement for a crop is not supplied at a time, but at a fixed interval so that the root zone of the crop may remain saturated throughout the crop period. Generally, the seasonal rainfall cannot meet the total water requirement. Hence, the additional requirement is fulfilled by the irrigation system.

The irrigation engineer should be acquainted with the type of soil, characteristics of soil moisture, quality of irrigation water, frequency of irrigation etc. for the proper implementation of irrigation system.

2.2 FACTORS AFFECTING THE WATER REQUIREMENT

The following are the factors that affect the water requirement of crops

(a) Water Table If the water table is nearer to the ground surface, the water requirement will be less. If it is much below the ground surface, the water requirement will be more.

(b) Climate In hot climate, the evaporation loss is more and hence the water requirement will be more and *vice versa.*

(c) Ground Slope If the slope of the ground is steep, the water flows down very quickly and the soil gets little time to absorb requisite moisture resulting in water loss. So, the water requirement will be more. But if the ground is flat, the water flows slowly and the soil gets sufficient time to absorb the requisite moisture. So, the water requirement is less.

(d) Intensity of Irrigation If the intensity of irrigation for a particular crop is high, then more area comes under the irrigation system and the water requirement is more and *vice versa.*

(e) Type of Soil In sandy soil water percolates very quickly and cannot be retained. So, water requirement is more. But the clayey soil can retain water near the root zone of crops. So, it requires less water.

(f) Method of Application of Water In surface method more water is required to meet up evaporation loss. In sub-surface method less water is required as the soil just absorbs the optimum moisture. In sprinkler method also less water is required as it just moists the soil like rainwater.

(g) Method of Ploughing In deep ploughing (by tractor) less water is required, because the soil can retain moisture for longer period. But in shallow ploughing (by bullocks) more water is required as the soil cannot retain moisture for a longer period due to evaporation.

2.3 DEFINITION OF IMPORTANT TERMS

1. Gross Command Area (G.C.A.)

The whole area enclosed between an imaginary boundary line which can be included in an irrigation project for supplying water to agricultural land by the network of canals is known as Gross Command Area (G.C.A.). It includes both the culturable and unculturable areas (Fig. 2.1).

2. Unculturable Area

The area where the agriculture cannot be done and crops cannot be grown is known as unculturable area. The marshy lands, barren lands, lakes, ponds, forests, villages, etc. are considered as unculturable area.

3. Culturable Area

The area where the agriculture can be done satisfactorily is known as culturable area.

4. Culturable Command Area (C.C.A.)

The total area within an irrigation project where the cultivation can be done and crops can be grown is known as Culturable Command Area (C.C.A). Again C.C.A. may be of two categories.

(a) Culturable Cultivated Area It is the area within C.C.A. where the cultivation has been actually done at present.

(b) Culturable Uncultivated Area It is the area within the C.C.A. where cultivation is possible but it is not being cultivated at present due to some reasons.

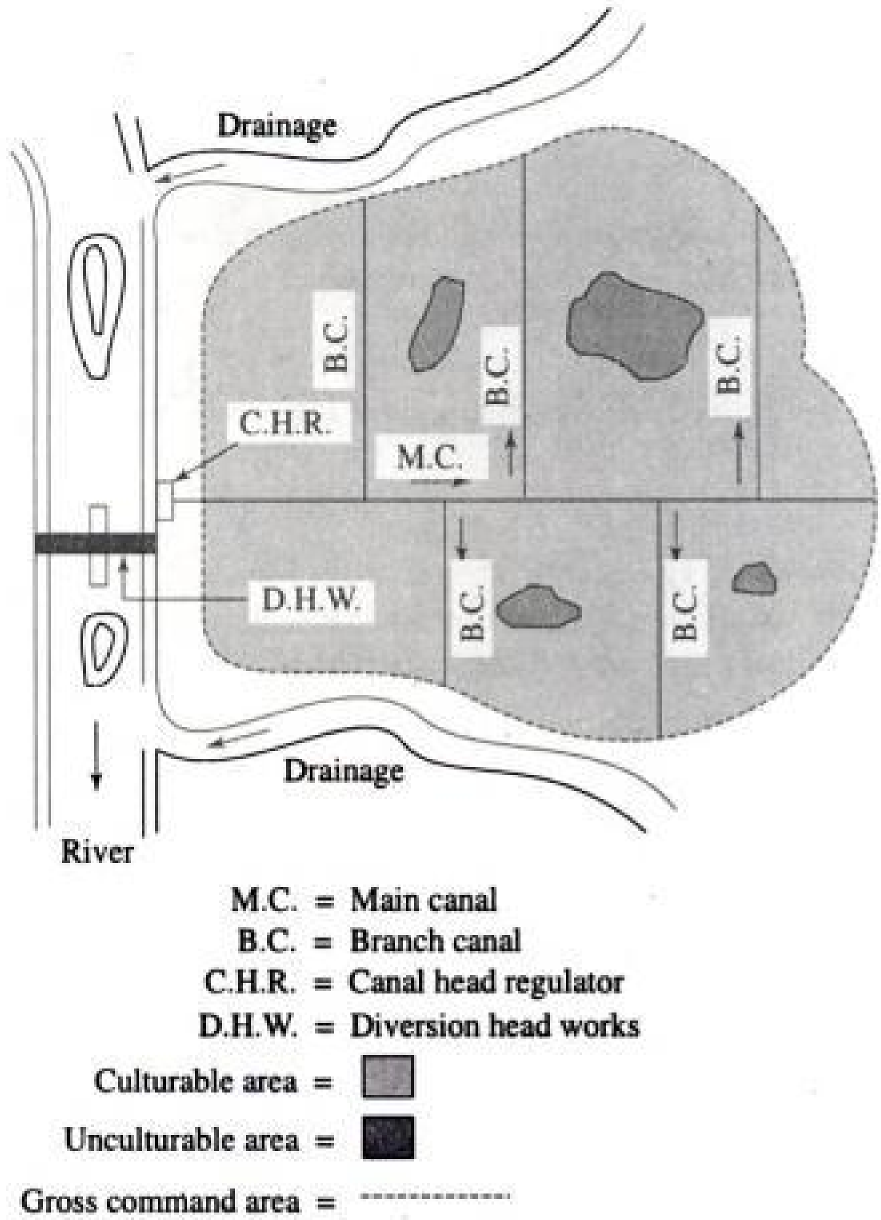


Fig. 2.1 Gross command area

5. Intensity of Irrigation

The total culturable command area may not be cultivated at the same time in a year due to various reasons. Some area may remain vacant every year. Again, various crops may be cultivated in the culturable command area. So, the intensity of irrigation may be defined as a ratio of cultivated land for a particular crop to the total culturable command area. it is expressed as a percentage of C.C.A. For example, if total culturable command area is 1000 hectares where wheat is cultivated in 250 hectares, then,

Intensity of irrigation for wheat = $\frac{250}{1000} \times 100 = 25\%$. So, area to be irrigated = C.C.A. × Intensity of irrigation.

6. Crop Ratio

It is defined as the ratio of the areas of the two main crop seasons, e.g. *Kharif* and *Rabi*.

For example, if the area under Kharif crop is 2500 hectares and the area under *Rabi* crop is 5000 hectares then, crop ratio of *Kharif* to *Rabi* is 1 : 2 (i.e. C.R. = $\frac{2500}{5000}$ = 1:2)

The crop ratio should be so selected that the discharge of the canal for supplying water to *Kharif* and *Rabi* may be nearly equal.

7. Crop Season

The period during which some particular types of crops can be grown every year on the same land is known as crop season. The following are the main crop seasons.

(a) Kharif Season This season ranges from June to October. The crops are sown in the very beginning of monsoon and harvested at the end of autumn.

The major kharif crops are—Rice, Millet, Maize, Jute, Groundnut, etc.

(b) Rabi Season This season ranges from October to March. The crops are sown in the very beginning of winter and harvested at the end of spring. The major *Rabi* crops are—Wheat, Gram, Mustard, Rapeseed, Linseed, Pulses, Onion, etc.

Again there are several crops which are not included in *Kharif* and *Rabi* as they require more time and they cover both the main seasons.

As for example, cotton requires eight months to mature and sugarcane requires about whole year to mature. Hence, they are designated as follows,

(i) Cotton—eight month's crop.
(ii) Sugarcane—perennial crop.

8. Cash Crop

The crops which are cultivated by the farmers to sell in the market to meet their current financial requirements are known as cash crops. The crops like vegetables, fruits, etc. are considered as cash crops.

9. Crop Rotation

The process of changing the type of crop for the cultivation on the same land is known as crop rotation. It is found that if same crop is cultivated on the same land every year, the fertility of the land gets reduced and the yield of crop also gradually reduces. This is so because the necessary salts required for the growth of a particular crop get exhausted. It is found by experiment that if the principle of crop rotation is practised, the fertility of the soil can be restored.

Few crop rotation possible are

(i) Rice—Gram.
(ii) Wheat—Millet—Gram.
(iii) Rice—Gram—Wheat.

10. Crop Period

The crop period is defined as the total period from the time of sowing a crop to the time of harvesting it. That means, it is the period in which the crop remains in the field.

11. Overlap Allowance

Sometimes a crop of one season may overlap the next crop season by a few days more which it requires to mature. During this period of overlapping the irrigation water is to be supplied simultaneously to the crops of both the seasons. Due to the extra demand of water during this period, the discharge of the canal has to be increased. So, for the purpose of canal design, a provision should be made for this extra demand. This provision is termed as *overlap allowance*. This is expressed in percentage.

12. Time Factor

The ratio of the number of days the canal has actually been kept open to the number of days the canal was designed to remain open during the base period is known as *Time factor.*

For example, a canal was designed to be kept open for 15 days, but it was practically kept open for 10 days for supplying water to the culturable area. Then the time factor is $\frac{10}{15}$.

So,

$$\text{Time factor} = \frac{\text{No. of days the canal practically kept open}}{\text{No. of days the canal was designed to keep open}}$$

$$= \frac{\text{Actual discharge}}{\text{designed discharge}}.$$

13. Capacity Factor

Generally, a canal is designed for a maximum discharge capacity. But, actually it is not required that the canal runs to that maximum capacity all the time of the base period. So, the ratio of the average discharge to the maximum discharge (designed discharge) is known as capacity factor.

For example, a canal was designed for the maximum discharge of 50 cumec, but the average discharge is 40 cumec.

So, $\text{Capacity factor} = \frac{40}{50} = 0.8.$

14. Number of Watering

The total depth of water required by a crop is not supplied at one time. But, it is supplied over the base period by stages depending upon the requirement.

The initial watering which is done on the land to provide moisture to the soil just before sowing any crop is known as paleo or paleva.

The first watering is done when the crop has grown to about three centimetres. This watering is known as *Kor* watering and the period is known as *Kor* period.

Subsequent watering is done at some regular intervals during the base period till the crop attains maturity.

The number of watering depends on the type of soil, base period, soil condition, climatic condition etc.

15. Cumec Day

The quantity of water flowing continuously for one day at the rate of one cumec is known as cumec-day.

$$
\begin{aligned}
1 \text{ cumec-day} &= \frac{1 \text{ m}^3}{\text{sec.}} \times 24 \times 60 \times 60 \text{ secs.} \\
&= 24 \times 60 \times 60 \text{ m}^3 \\
&= \frac{24 \times 60 \times 60}{10{,}000} \times 1 \text{ m } (1 \text{ hectare} = 10\,000 \text{ m}^2) \\
&= 8.64 \text{ hectare-metre.}
\end{aligned}
$$

16. Arid Region

The area where the rainfall is very scanty and occurs irregularly and where the agriculture is not at all possible is known as arid region.

2.4 BASE, DELTA AND DUTY

Base The base is defined as the period from the first to the last watering of the crop just before its maturity. It is also known as base period. It is denoted by 'B' and expressed in number of days. The base period for some common crops are given in Table 2.1.

Table 2.1

Crop	*Base in days*
Rice	120
Wheat	120
Maize	100
Cotton	200
Sugarcane	320

Delta Each crop requires certain amount of water per hectare for its maturity. If the total amount of water supplied to the crop (from first to last watering) is stored on the land without any loss, then there will be a thick layer of water standing on that land. This depth of water layer is known as Delta for the crop. It is denoted by 'Δ' and expressed in cm. Delta for some common crops is given in Table 2.2.

Table 2.2

Kharif crop	*Delta in cm*
Rice	125
Maize	45
Ground nut	30
Millet	30
Rabi crop	Delta in cm
Wheat	40
Mustard	45
Gram	30
Potato	75

Duty The duty of water is defined as number of hectares that can be irrigated by constant supply of water at the rate of one cumec throughout the base period. It is expressed in hectares/cumec. and is denoted by 'D'. The duty of water is not constant, but it varies with various factors like soil condition, method of ploughing, method of application of water, etc. The duties of some common crops are given in Table 2.3.

Table 2.3

Crop	*Duty in hectares/cumec*
Rice	900
Wheat	1800
Cotton	1400
Sugarcane	800

2.5 FACTORS AFFECTING DUTY

The factors that affect the duty are described below.

1. Soil Characteristics If the soil of the canal bed is porous and coarse grained, it leads to more seepage loss and consequently low duty. If the soil is compact and closed grained, the seepage loss will be less and the duty will be high.

If the agricultural land consists of sandy soil, the percolation loss will be high causing the duty to be low. If it consists of alluvial soil, the percolation loss will be less and the soil retains the moisture for longer period and consequently the duty will be high.

2. Climatic Condition When the atmospheric temperature of the command area becomes high, the evaporation loss is more and the duty becomes low and *vice versa*.

3. Rainfall If the rainfall is sufficient during the crop period, less quantity of irrigation water shall be required and therefore the duty will be more and *vice versa*.

4. Base Period When the base period is longer, the water requirement will be more and the duty will be low and *vice versa*.

5. Type of Crop The water requirement of various crops are different. So, the duty varies from crop to crop.

6. Topography of Agricultural Land If the agricultural land is uneven, the water requirement will be more and hence the duty will be low. If the land has slight slope, the duty will be high as water requirement is optimum. As the ground slope increases the duty decreases because there is wastage of water.

7. Method of Ploughing Proper deep ploughing which is done by tractors requires overall less quantity of water and hence the duty is high. But, shallow ploughing with bullocks requires overall more quantity of water, and hence the duty is low.

8. Methods of Irrigation The duty of water is high in case of perennial irrigation system as compared to that in inundation irrigation system. It is so because in Perennial system head regulator is used whereas in inundation system there is no regulator.

9. Water Tax If some tax is imposed on the basis of the volume of water consumption, the farmer will use the water economically, and thus the duty will be high.

2.6 METHODS OF IMPROVING DUTY

The following points should be remembered for improving the duty of water.

1. Proper Ploughing Ploughing should be done properly and deeply so that the moisture retaining capacity of the soil is increased.

2. Methods of Supplying Water The method of supplying water to the agricultural land should be decided according to the field and soil conditions. For example,
Furrow method—for crops sown in rows.
Contour method—for hilly areas.
Basin method—for orchards.
Flooding method—for plain lands.

3. Canal Lining To reduce percolation loss the canals should be lined according to site condition.

4. Transmission Loss To reduce transmission loss the canals should be taken close to the irrigable lands as far as possible.

5. Crop Rotation The principle of crop rotation should be adopted to increase the moisture retaining capacity and fertility of the soil.

6. Implementation of Tax The water tax should be imposed on the basis of volume of water consumption.

2.7 RELATION BETWEEN BASE, DELTA AND DUTY

Let,

D = Duty of water in hectares/cumee

B = Base in days,

Δ = Delta in m

From definition, one cumec of water flowing continuously for 'B' days gives a depth of water Δ over an area 'D' hectares. That is,

1 cumec for B days gives Δ over D hectares

or 1 cumec for 1 days gives Δ over $\frac{D}{B}$ hectares

or 1 cumec for 1 day $= \frac{D}{B} \times \Delta$ hectare-metre

So, 1 cumec-day $= \frac{D}{B} \times \Delta$ hectare-metre (1)

Again, 1 cumec-day $= 1 \times 24 \times 60 \times 60 = 86400 \text{ m}^3$

$= 8.64$ hectare-metre (2)

(1 hectare $= 10{,}000 \text{ m}^2$)

From (1) and (2)

$$\frac{D}{B} \times \Delta = 8.64$$

$$\therefore \quad \Delta = \frac{8.64 \times B}{D} = \text{in m.}$$

2.8 NUMERICAL PROBLEMS ON BASE, DELTA AND DUTY

Problem 1 A channel is to be designed for irrigating 5000 hectares in *Kharif* crop and 4000 hectares in *Rabi* crop. The water requirement for *Kharif* and *Rabi* are 60 cm and 25 cm, respectively. The *Kor* period for Kharif is 3 weeks and for *Rabi* is 4 weeks. Determine the discharge of the channel for which it is to be designed.

Solution Using the relation.

$$\Delta = \frac{8.64 \times B}{D}$$

Discharge for Kharif Crop

Here, $\Delta = 60 \text{ cm} = 0.60 \text{ m}$

$B = 3 \text{ weeks} = 21 \text{ days}$

$$\therefore \qquad \text{Duty} = \frac{8.64 \times 21}{0.60} = 302.4 \text{ hectares/cumec}$$

Area to be irrigated = 5000 hectares

$$\text{Required discharge of channel} = \frac{5000}{302.4} = 16.53 \text{ cumec}$$

Discharge for Rabi Crop

Here, $\Delta = 25 \text{ cm} = 0.25 \text{ m}$

$B = 4 \text{ weeks} = 28 \text{ days},$

$$\therefore \qquad \text{Duty} = \frac{8.64 \times 28}{0.25} = 967.68 \text{ hectares/cumec,}$$

Area to be irrigated = 4000 hectares.

$$\text{Required discharge of channel} = \frac{4000}{967.68} = 4.13 \text{ cumec}$$

So, the channel is to be designed for the maximum discharge of 16.53 cumec, because this discharge capacity of the channel will be able to supply water to both the seasons.

Problem 2 The gross command area of an irrigation project is 1.5 lakh hectares, where 7,500 hectare are unculturable.The area of kharif crop is 60,000 hectares and that of *Rabi* crop is 40,000 hectares. The duty of *Kharif* is 3000 hectares/cumec and the duty of *Rabi* is 4000 hectares/cumec.

Find (a) The design discharge of channel assuming 10% transmission loss.

(b) Intensity of irrigation for *Kharif* and *Rabi*.

Solution Culturable command area = 1,50,000 – 7500 = 142500 hectares.

Discharge for Kharif Crop,

Area of *Kharif* crop = 60,000 hectares.

Duty of *Kharif* crop = 3000 hectares/cumec

$$\text{Required discharge of channel} = \frac{60{,}000}{3000} = 20 \text{ cumec}$$

Considering 10% loss

$$\text{Design discharge} = 20 \times \frac{110}{100} = 22 \text{ cumec}$$

Discharge for Rabi Crop

Area of *Rabi* crop = 40,000 hectares

Duty of *Rabi* crop = 4000 hectares/cumec

$$\text{Required discharge of channel} = \frac{40000}{4000} = 10 \text{ cumec}$$

Considering 10% loss

$$\text{Design discharge} = 10 \times \frac{110}{100} = 11 \text{ cumec}$$

(a) So, the design discharge of the channel should be 22 cumec, as it is maximum.

(b) Intensity of irrigation for *Kharif* = $\frac{60{,}000}{142500} \times 100 = 42.11\%$

Intensity of irrigation for *Rabi* = $\frac{40{,}000}{142500} \times 100 = 28.07\%$

Problem 3 The gross command area of an irrigation project is 1 lakh hectares. The culturable command area is 75% of G.C.A. The intensities of irrigation for *Kharif* and *Rabi* are 50% and 55% respectively. If the duties for *Kharif* and *Rabi* are 1200 hectare/cumec and 1400 hectares/cumec respectively, determine the discharge at the head of the canal considering 20% provisions for transmission loss, overlap allowance, evaporation loss etc.

Solution Culturable command area = $100000 \times \frac{75}{100}$ = 75,000 hectares

For Kharif crop,

Area under *Kharif* = $75{,}000 \times \frac{50}{100}$ = 37500 hectares

Duty for *Kharif* = 1200 hectares/cumec

Required discharge for *Kharif* = $\frac{37500}{1200}$ = 31.25 cumec

For Rabi crop,

Area under *Rabi* = $75{,}000 \times \frac{55}{100}$ = 41250 hectares

Duty for *Rabi* = 1400 hectares/cumec

Required discharge for *Rabi* = $\frac{41250}{1400}$ = 29.46 cumec

So, to meet up the actual water requirement of the crops, the discharge of the canal at the head of the field should be 31.25 cumec (as it is maximum). Now considering 20% provision for losses,

Required discharge at the head of canal = $31.25 \times \frac{120}{100}$

= 37.5 cumec

Problem 4 Determine the head discharge of a canal from the following data. The value of time factor may be assumed as 0.75.

Crop	*Base period in days*	*Area in hectare*	*Duty in hec tares/cumec*
Rice	120	4000	1500
Wheat	120	3500	2000
Sugarcane	310	3000	1200

Solution Discharge of canal required

(a) For rice = $\frac{4000}{1500}$ = 2.667 cumec (*Kharif*)

(b) For wheat $= \frac{3500}{2000} = 1.750$ cumec (*Rabi*)

(c) For sugarcane $= \frac{3000}{1200} = 2.500$ cumec (perennial)

As, the base period of sugarcane is 310 days, it will require water both in *Kharif* and *Rabi* seasons.

Now, actual discharge required in *Kharif* season = 2.667 + 2.500 = 5.167 cumec.

Actual discharge required in *Rabi* season = 1.750 + 2.500 = 4.250 cumec.

So, the maximum discharge in *Kharif* season (i.e. 5.167 cumec) should be taken into consideration as it will be able to serve both the seasons.

$$\text{Time factor} = 0.75 = \frac{\text{Actual discharge}}{\text{Design discharge}} = \frac{5.167}{\text{Design discharge}}$$

$$\text{Design discharge} = \frac{5.167}{0.75} = 6.889 \text{ cumec}$$

Therefore, the required head discharge of the canal is 6.889 cumec.

Problem 5 Find out the capacity of a reservoir from the following data. The culturable command area is 80,000 hectares.

Crop	*Base in days*	*Duty in hect/cumec*	*Intensity of irrigation in percentage*
Rice	120	1800	25
Wheat	120	2000	30
Sugarcane	320	2500	20

Assume the canal and reservoir losses as 5% and 10% respectively.

Solution Using relation

$$\Delta = \frac{8.64 \times B}{D}$$

Calculation of delta for each crop

$$\textit{delta for } \text{rice} = \frac{8.64 \times 120}{1800} = 0.576 \text{ m}$$

$$\textit{delta} \text{ for wheat} = \frac{8.64 \times 120}{2000} = 0.518 \text{ m}$$

$$\textit{delta} \text{ for sugarcane} = \frac{8.64 \times 320}{2500} = 1.106 \text{ m}$$

Calculation of area for each crop

$$\textit{Area} \text{ for rice} = \frac{80{,}000 \times 25}{100} = 20{,}000 \text{ hectare}$$

$$Area \text{ for wheat } = \frac{80{,}000 \times 30}{100} = 24000 \text{ hectare}$$

$$Area \text{ for sugarcane } = \frac{80{,}000 \times 20}{100} = 16{,}000 \text{ hectare}$$

Volume of water required for each crop

We know, volume = area × delta

∴ Volume for rice = 20,000 × 0.576 = 11520.00 ha-m.

Volume for wheat = 24,000 × 0.518 = 12432.00 ha-m.

Volume for sugarcane = 16,000 × 1.106 = 17696.00 ha-m

Total volume of water = 41648.00 ha-m

Considering canal loss of 5%

$$\text{Water required at the head of canal} = 41648 \times \frac{105}{100} = 43730.40 \text{ ha-m}$$

Again considering, reservoir loss of 10%

$$\text{Capacity of reservoir} = 43730.40 \times \frac{110}{100} = 48103.44 \text{ ha-m}$$

Problem 6 The command area of a channel is 4000 hectares. The intensity of irrigation of a crop is 70%. The crop requires 60 cm of water in 15 days, when the effective rainfall is recorded as 15 cm during that period.

Find,(a) The duty at the head of field.

(b) The duty at the head of channel.

(c) The head discharge at the head of channel.

Assume total losses as 15%.

Solution

Depth of water required = 60 mm

Effective rainfall = 15 cm

Depth of irrigation water = 60–15 = 45 cm

∴ Delta = 45 cm = 0.45 m, B = 15 days

$$\text{From relation, } \Delta = \frac{8.64 \times B}{D}$$

$$\text{Duty } D = \frac{8.64 \times 15}{0.45} = 288 \text{ hectares/cumec}$$

(a) So, duty at the head of field = 288 ha/cumec. Due to the losses of water the duty at the head of the channel will be reduced.
Here, losses are 15%.

(b) So, the duty at the head of channel $= 288 \times \frac{85}{100} = 244.80$ hect/cumec

(Duty will be reduced due to loss)

$$\text{Total area under crop} = 4000 \times \frac{70}{100} = 2800 \text{ hectares}$$

(c) $$\text{The discharge at the head of channel} = \frac{2800}{244.8} = 11.438 \text{ cumec}$$

2.9 TYPES OF SOILS

The soils are divided into following four groups.

(a) Alluvial Soils

These soils are formed by the deposition of silt which is carried by the river water during floods. Again, the silt is formed due to the weathering action on the rocks by the heavy current of the river water in the hilly region. The alluvial soils are found in Indo-Gangetic plains, Bramhaputra plains and plains of other big rivers in India. The moisture absorbing capacity of these soils is very good and these are most suitable for agriculture as they possess good manure value.

(b) Black Soils

These soils originated from the weathering action on the rocks like granite, basalt, trap etc. These soils are mainly found in Andhra Pradesh, Madhya Pradesh, Gujarat, Tamil Nadu, etc. The characteristics of black soils are that these are sticky when wet and becomes very hard when dry. The black soil is suitable for the cultivation of cotton.

(c) Red Soils

These are formed by the weathering action on the rocks of igneous and metamorphic groups. These are also known as loam. The water absorbing capacity of red soil is very low. The red soils are found in Karnataka, Tamil Nadu, Maharashtra, Orissa, West Bengal, etc.

(d) Laterite Soils

These are formed by the weathering action on the laterite rocks. These are yellowish-red in colour and possess good drainage property. Laterite soils are found in Malabar, Kerala, Karnataka, Orissa, Assam, etc.

2.10 TYPES OF SOIL WATER

When water is spread over the soil either by irrigation or by rainfall, the water is absorbed by the pores of the soil. This water is termed as soil water or soil moisture. The following are the various forms of soil water.

(a) Gravitational Water

When it rains or the irrigation water is supplied to the soil, the water content of the soil goes on increasing until a saturation point is reached (Fig 2.2). At this stage the soil pores are completely saturated and no more water is absorbed by the soil. The surface water then starts flowing downwards due to the influence of gravity. The portion of water which flows down is known as gravitational water. The gravitational water is not useful for the plants as it flows out rapidly and it cannot be absorbed by the rootzone.

(b) Capillary Water

The portion of the water retained by the soil, after completely eliminating the gravitational water, is known as capillary water. This water gets absorbed by the root of the plants and is held in the capillary pores of the soil particles. The water content goes on reducing gradually due to evaporation and transpiration.

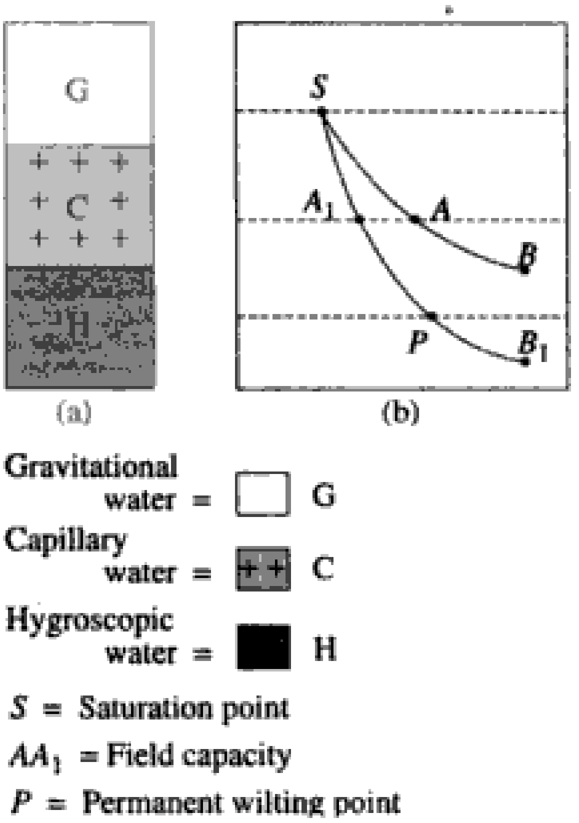


Fig. 2.2 Soil water

When there are no plants on the soil water content is reduced only by evaporation which is shown by the curve *SAB* (Fig. 2.2).

When there are plants on the soil, the water content gets reduced both by evaporation and transpiration which is shown by curve $SA_1\,PB_1$. Here, the point *P* is known as the permanent wilting point, and the water content corresponding to *A* or A_1 is known as field capacity.

(c) Hygroscopic Water

The water content below the permanent wilting point is known as hygroscopic water. This water is retained by the soil in the form of thin film on the surface of soil particles. This water cannot be extracted or absorbed by the root of the plants. So, at this stage, the growth of the plants is stopped and ultimately the plants are dead.

(d) Field Capacity

The field capacity is defined as the amount of maximum moisture that can be held by the soil against gravity. It is expressed as percentage.

(e) Permanent Wilting Point

The permanent wilting point is defined as the amount of moisture held by soil which cannot be extracted by the plant roots for transpiration. At this point the wilting of the plant occurs. It is also expressed in percentage.

2.11 CONSUMPTIVE USE OF WATER

The consumptive use of water is defined as the total quantity of water used for the growth of the plants by transpiration and the amount lost by evaporation (which is known as evapo-transpiration).

It is expressed in hectare-metre per hectare or as depth of water (in m) for a specified period. The consumptive use of water is ascertained to know the water requirement for each crop. The value of consumptive use of water varies from crop to crop, time to time and even from place to place.

2.12 METHODS OF DETERMINING CONSUMPTIVE USE

The consumptive use of water is determined by the following methods.

(a) Lysimeter Method

In this method, a water tight tank of cylindrical shape having diameter about 2 m and depth about 3 m is placed vertically on the ground. The tank is filled with sample of soil. The bottom of the tank consists of a sand layer and a pan for collecting the surplus water. The consumptive use of water is ascertained by measuring the amount of water required for the satisfactory growth of the plants within the tank. The consumptive use of water is given by

$$C_u = W_a - W_d$$

where C_u = Consumptive use of water,
W_a = Water applied,
W_d = Water drained off.

(b) Field Experimental Method

In this method, some fields are selected for experiment. The quantity of water is applied in such a way that it is sufficient for the satisfactory growth of the crops. There should be no run off or deep percolation. If there is any runoff it should be measured and deducted from the total quantity of water applied. Since, the phenomenon of deep percolation cannot be ascertained in the field to get consumptive use of water some correction for deep percolation may be applied by finding the same (i.e. correction) from field observations.

(c) Soil Moisture Study

In this method, several plots of land are selected where irrigation water is to be supplied. The soil samples are taken from different depths at the root zone of the plants just before and after the irrigation. Then the water contents of the soil samples are ascertained by laboratory tests. The depth of water removed from the soil is determined by relation,

$$D_r = \frac{pwd}{100}$$

where, D_r = Depth of water removed in m; p = Percentage of water content; w = Sp. gr. of soil; d = depth of soil in m.

The total quantity of water removed in 30 days period is calculated. Then a curve of water consumption versus time is prepared. From this curve the water consumption for any period can be ascertained.

2.13 FREQUENCY OF IRRIGATION

The irrigation water is applied to the field to raise the moisture content of the soil up to its field capacity. The application of water is then stopped. The water content also reduces gradually due to transpiration and evaporation. If the moisture content is dropped below the requisite amount, the growth of the plants gets disturbed. So the moisture content requires to be immediately replenished by irrigation and it should be raised to the field capacity. The frequency of irrigation should be worked out in advance so that it can be applied in proper intervals.

The frequency of irrigation may be ascertained by the following expressions,

(a) $$D_w = \frac{W_s \times d}{W_w} \times [F_c - M_o]$$

where, D_w = Depth of water to be applied in each watering; d = Depth of root zone; W_s = Unit wt. of soil; W_w = Unit wt. of water; F_c = Field capacity; M_o = Optimum moisture content.

(b) $$f_w = \frac{D_w}{C_u}$$

where, f_w = Frequency of watering; D_w = Depth of water to be applied in each watering; C_u = Daily consumptive use of water.

Example Determine the frequency of irrigation from the following data.

(i) Field capacity of soil = 35%
(ii) Permanent wilting point = 18%
(iii) Density of soil = 1.5 g/cm^3
(iv) Depth of root zone = 70 cm
(v) Daily consumptive use of water = 17 mm

Solution

Available moisture = Field capacity – Permanent wilting point = 35 – 18 = 17%.

Let, readily available moisture is 75% of the available moisture,

Readily available moisture = 17 × 0.75 = 12.75%

Optimum moisture content = 35 – 12.75 = 22.25%

Now, by applying irrigation water the moisture content is to be raised from 22.25% to 35%

From, $$D_w = \frac{W_s \times d}{W_w} \times [F_c - M_o]$$

Here, $W_s = 1.5$ g/cm^3, $W_w = 1$ g/cm^3, $d = 70$ cm $= 0.70$ m, $F_c = 35\% = 0.35$, $M_o = 22.25\% = 0.2225$.

$$D_w = \frac{1.5 \times 0.70}{1} \times [0.35 - 0.2225]$$

$$= 1.05 \times 0.1275$$

$$= 0.133875 \text{ m}$$

$$= 13.39 \text{ cm}$$

Daily consumptive use of water (C_u) = 17 mm = 1.7 cm

From, $$f_w = \frac{D_w}{C_u}$$

Frequency of irrigation, $f_w = \frac{13.39}{1.7} = 7.87 = 8$ days (say)

Hence, water should be applied in the field at an interval of 8 days.

2.14 STANDARD OF IRRIGATION WATER

Absolutely pure water cannot be expected for irrigation. However, there should be some acceptable limits to the impurities so that the yield of crop is not hampered. The following are the impurities that may exist in water.

(a) Sediment Concentration

The sediment of fine silt improve the fertility of land. Whereas other types of sedimentation decreases the fertility. Again, excessive sedimentation creates trouble in canals and reservoirs. So, water should not contain excessive suspended sediment.

(b) Concentration of Soluble Salt

The presence of salts of Calcium, Magnesium, Sodium and Potassium may be injurious to crops if it exceeds the permissible limit. The salt concentration may be measured electrically or it may be computed by laboratory test. The concentration is generally expressed in the following ways,

(i) When expressed in P.P.M. the amount in excess of 700 P.P.M. is harmful to plants.

(ii) Electrical conductivity of saline water is expressed in micro mhos/cm the value up to 250 micro mhos/cm is helpful for all crops. But the value from 250 – 750 micro mhos/cm is very injurious for crops.

(c) Proportion of Sodium Ions

Generally, all soils co' tain Calcium, Magnesium and Sodium ions. The percentage of Sodium ions should be less than 5. If the percentage increases to more than 10, it is injurious to crops. The proportion of sodium ions is designated by a factor which is known as Sodium Absorption Ratio (S.A.R.). The water having the value of S.A.R. between '0' and '10' is helpful for all crops. The value more than 10 is unsuitable for irrigation.

(d) Concentration of Potentially Toxic Elements

Some elements are toxic to the crops. Initially, Boron is useful for the growth of crops, but its concentration above 0.3 P.P.M may be toxic to the plants. The concentration of selenium is also toxic to the plants

(e) Bi-Carbonate Concentration

The concentration of Calcium and Magnesium bicarbonates increases the proportion of sodium ions which is hazardous for the plant's life.

(f) Bacterial Contamination

The bacterial contamination of irrigation water is not directly harmful to the plants, but the food grains or fruits which are grown by using bacterial contaminated water may be harmful to human beings.

2.15 IRRIGATION EFFICIENCY

The amount of irrigation water supplied to the land is not fully utilised for the growth of the crops. This is due to various losses. Now, the ratio of the amount of water available (output) to the amount of water supplied (input) is known as Irrigation Efficiency. It is expressed in percentage. The following are the various types of irrigation efficiencies.

(a) Water Conveyance Efficiency (η_c)

It is the ratio of the amount of water applied to the land to the amount of water supplied from the reservoir. It is obtained by the expression,

$$\eta_c = \frac{W_l}{W_r} \times 100$$

where, η_c = Water conveyance efficiency; W_l = Amount of water applied to land, W_r = Amount of water supplied from reservoir.

(b) Water Application Efficiency (η_a)

It is the ratio of the water stored in root zone of plants to the water applied to the land.

It is obtained by the expression, $\eta_a = \frac{W_z}{W_l} \times 100$

where, η_a = Water application efficiency, W_z = Amount of water stored in root zone, W_l = Amount of water applied to land.

(c) Water Use Efficiency (η_u)

It is the ratio of the amount of water used to the amount of water applied.

It is obtained from the relation,

$$\eta_u = \frac{W_u}{W_l} \times 100$$

where, η_u = Water use efficiency; W_u = Water used; W_l = Water applied.

(d) Consumptive use Efficiency (η_{cu})

It is the ratio of the consumptive use of water to the amount of water depleted from the root zone. It is obtained from the expression,

$$\eta_{cu} = \frac{C_u}{W_p} \times 100$$

where, η_{cu} = Consumptive use efficiency; C_u = Consumptive use of water; W_p = Amount of water depleted from root zone.

REVIEW QUESTIONS

1. Fill up the blanks with appropriate word/words:
   - (i) If the ________ is nearer to the ground surface, the water requirement of the crop will be less.
   - (ii) In hot climate the ________ loss is more and hence the ________ will be more.
   - (iii) If the slope of the ground is ________, the water requirement will be more.
   - (iv) In deep ploughing, the soil can ________ moisture for longer period.
   - (v) The total area enclosed between the boundary line of an irrigation project is known as ________.
   - (vi) The area where the agriculture can be done is known as ________.
   - (vii) The season ranges from June to October is known as ________.
   - (viii) The season ranges from October to March is known as ________.
   - (ix) Cotton is known as ________ crop.
   - (x) Sugarcane is known as ________ crop.
   - (xi) The period from sowing a crop to the time of harvesting is known as ________.
   - (xii) The initial watering which is done on the land just before sowing any crop is known as ________.
   - (xiii) The first watering which is done when the crop has grown to few centimetres is known as ________.
   - (xiv) One cumec day is equal to ________ hectare-metre.
   - (xv) The duty of water is expressed in ________.
2. What are the points to be remembered while ascertaining the water requirement of crops?
3. What is the difference between the gross command area and culturable Command area?

4. What is duty? What points do you consider while determining the duty of water?
5. Define base, delta and duty and establish a relation between them.
6. Distinguish between paleva and *kor* watering.
7. Distinguish between gravitational water, capillary water and hygroscopic water.
8. What is meant by the frequency of irrigation? Why is the frequency of irrigation ascertained for?
9. What should be the standard of the irrigation water?
10. What are the different types of irrigation efficiency?

ANSWERS

1. (i) Water table
(ii) evaporation, water requirement
(iii) steep
(iv) retain
(v) gross command area
(vi) culturable command area
(vii) *kharif* season
(viii) *rabi* season
(ix) eight month's
(x) perennial
(xi) crop period
(xii) paleva
(xiii) *kor* watering
(xiv) 8.64
(xv) hectares/cumec

3

HYDROLOGY

3.1 DEFINITION

The science of studying the different forms of water available above the earth surface or below the earth surface is known as hydrology. It includes the following points.

1. The measurement of precipitation, (i.e. rainfall).
2. The study of water losses due to transpiration, evaporation, absorption and infiltration.
3. Estimation of run-off and peak flow.
4. The procedure of river gauging.
5. Preparation of hydrograph to predict maximum food discharge.
6. The procedure of river training works.
7. The procedure of flood forecasting and flood control works.
8. Availability of underground water.

3.2 IMPORTANCE OF HYDROLOGY

The knowledge of hydrology is very essential for the applications:

(a) Determination of the capacity of a reservoir from the rainfall records and the yearly discharge observation of a river.
(b) Determination of peak flow of a river.
(c) Determination of suitable site for hydro-electric power generation.
(d) Sources of watersupply in a town or city.
(e) Methods to be adopted for the flood control.

3.3 SOME TERMS RELATED TO HYDROLOGY

Catchment Area The catchment area of a river means the area from where the surface run off flows to that river through the tributaries, streams, springs etc. The area is bounded by watershed line.

Run-off When it rains, some portion of rain water infiltrates into the soil, some is intercepted by vegetation, some evaporates and the remaining portion flows over the ground surface to join the rivers, streams, lakes etc. This portion of water which flows over the ground surface is known as surface run off or run-off.

The surface run off is also designated by rainfall excess or effective rainfall.

3.4 HYDROLOGIC CYCLE

The water of the universe always changes from one state to other under the effect of the sun. The water from the surface sources like lakes, rivers, ocean, etc. converts to vapour by evaporation due to solar heat. The vapour goes on accumulating continuously in the atmosphere. This vapour is again condensed due to the sudden fall of temperature and pressure. Thus clouds are formed. These clouds again causes the precipitation (i.e. rainfall). Some of the vapour is converted to ice at the peak of the mountains. The ice again melts is summer and flows as rivers to meet the sea or ocean. These processes of evaporation, precipitation and melting of ice go on continuously like an endless chain and thus a balance is maintained in the atmosphere. This phenomenon is known as hydrologic cycle (Fig. 3.1).

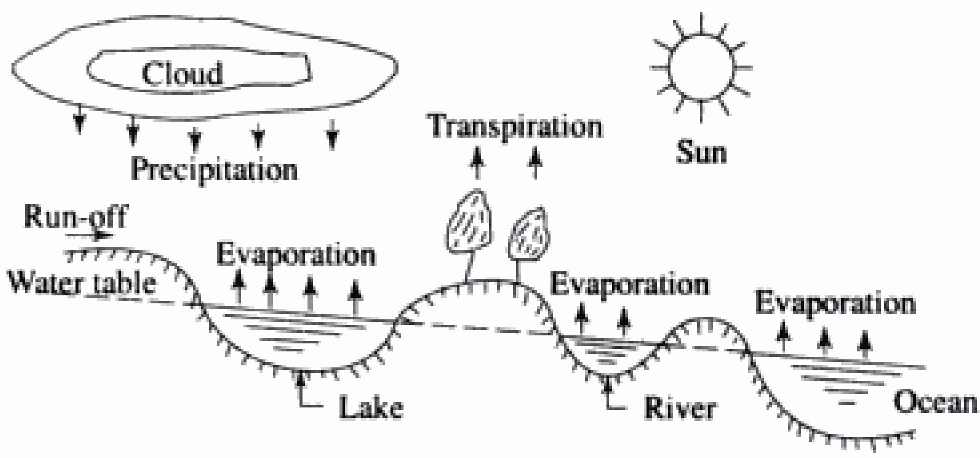


Fig. 3.1 Hydrologic cycle

3.5 HYDROGRAPH

The hydrograph is a graphical representation of the discharge of a river (in cumec) against the time (in hr or days). The discharge is plotted as ordinate (y-axis) and the time is plotted as abscissa (x-axis) (Fig. 3.2).

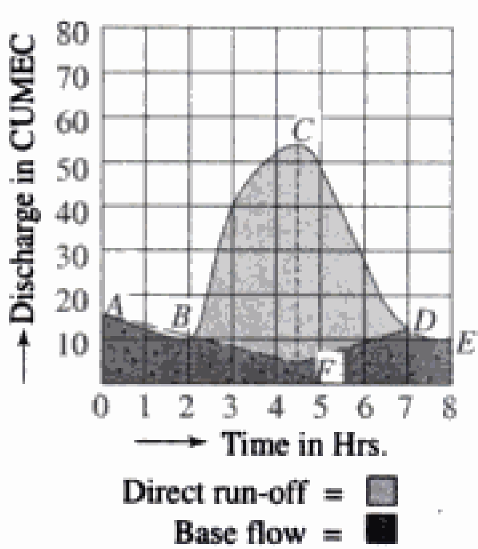


Fig. 3.2 Hydrograph

During the dry season, there is only base flow (i.e. ground water flow) but no surface run off. This may be shown by a line which is approximately straight (not shown in the figure).

In rainy season, at the beginning of the rainfall there is only base flow (shown by the line AB). After some period, when the initial losses (like interception, evaporation and infiltration) are fulfilled, the surface runoff starts and hence the discharge of the river goes on increasing. Hence the limb of the curve rises which is called rising limb (shown by the line BC). This line reaches to the peak value at 'C'. Again when the rain stops, the flow in the river decreases and the limb of the curve declines. This limb is known as recession limb) (as shown by the line CD). The discharge at the point C indicates the maximum discharge (i.e. peak discharge or flood discharge). The total area under the curve $ABCDE$ indicates the total run off. But this run off includes the base flow and the direct runoff. So, to get the actual run off the base flow is to be deducted by separating it from total area.

The method of separation of base flow is discussed in Sec. 3.8.

3.6 HYETOGRAPH

The graphical representation of rainfall and run-off is known as hyetograph (Fig. 3.3). The graph is prepared with intensity of rainfall (in cm/hr) as ordinate and time (in hrs) as abscissa. The infiltration capacity curve is drawn on this graph to show the amount of infiltration loss (shown by dotted portion). The upper portion indicates the effective rainfall (shown by hatched lines). The centroid of the effective rainfall is ascertained on the graph for the determination of total run-off at any specified period.

3.7 UNIT HYDROGRAPH

A unit hydrograph may be defined as a hydrograph which is obtained from one cm of effective rainfall (i.e. run-off) for unit duration. Here, effective rainfall means the rainfall excess (i.e. run-off) which directly flows to the river or stream. The unit duration is the period during which the effective rainfall is assumed to be uniformly distributed. The unit duration may be considered as 1 hr, 2 hr, 3 hr, 4 hr ..., etc. As for example, if a hydrograph is prepared for an effective rainfall of one cm lasting for 2 hrs, then it is known as 2 hr. unit hydrograph, for the duration of 3 hrs it is known as 3 hr unit hydrograph and so on (Fig. 3.4).

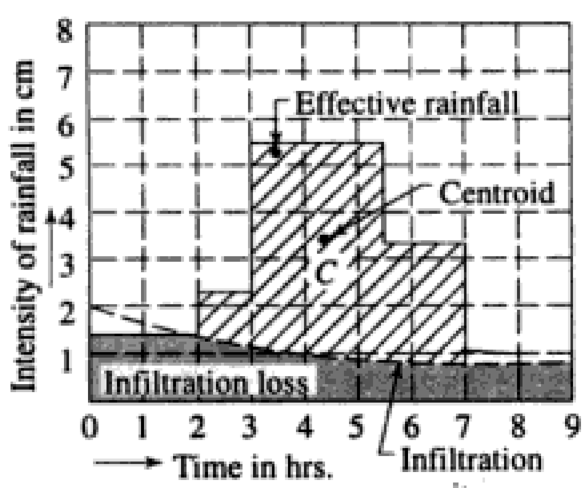


Fig. 3.3 Hyetograph

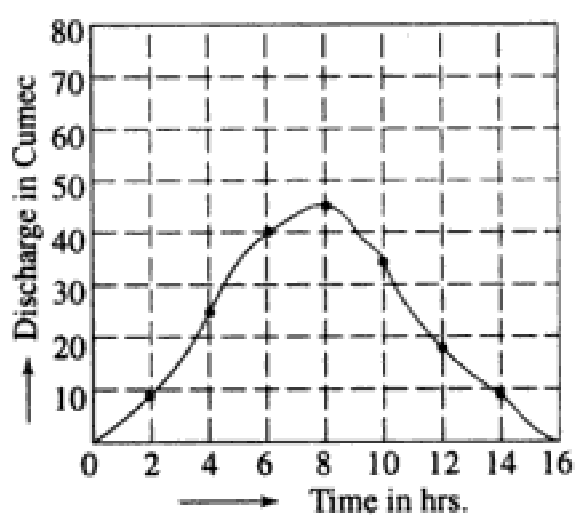


Fig. 3.4 Unit hydrograph

Concept of Unit Hydrograph

The unit hydrograph theory is based on the conception that if two identical storms occur on a drainage basin with identical conditions, then the unit hydrographs of runoff from the two storms may be expected as same. This conception of unit hydrograph was first given by L.K. Sherman in 1932.

Assumptions in Unit Hydrograph Theory The unit hydrograph theory is based on the following assumptions.

(i) The effective rainfall is evenly distributed during the specified period of time.
(ii) The effective rainfall is evenly distributed over the whole drainage basin.
(iii) For a drainage basin, the base period of direct runoff corresponding to effective rainfalls of different intensities is constant, provided unit duration is same.
(iv) The ordinates of all the hydrographs of a common base period are directly proportional to the total amount of direct run-off. This is also known as principle of linearity.
(v) The hydrograph of direct run off prepared from a given pattern of effective rainfall remains invariable irrespective of time of occurrence. This is known as principle of time invariance.

Limitations of Unit Hydrograph Theory The following are the limitations of unit hydrograph.

(i) This theory is not applicable to large areas because uniformly distributed effective rainfall cannot be expected in large area.
(ii) This theory is not applicable in places where precipitation is composed of snowfall.
(iii) The principle of time invariance is valid only for specified time and condition of drainage basin.
(iv) Practically no two storms have the same nature in space and time period. So, it is not possible to construct unit hydrograph for each pattern.
(v) The principle of linearity is not practically valid for smaller and larger storms.

Advantages of Unit Hydrograph Theory Inspite of some limitation of the unit hydrgraph theory, the following are the advantages it has

(i) Flood hydrograph can be prepared quickly for a given basin.
(ii) It can be utilised for the calculation of ordinates of hydrographs.
(iii) From the unit hydrograph the expected volume of run-off from a basin can be computed.

Construction of Unit Hydrograph The rainfall records for a specified period are collected from the raingauge stations of the catchment area. The discharge from the catchment area is also observed for the same period. A hyetograph is prepared from the rainfall records. A discharge hydrograph is also prepared from the recorded discharge. These two data are the basic requirement for the construction of unit hydrograph. The volume of direct run-off (in cumec) obtained from the hydrograph is converted to cm/sec according to the following method.

The discharge (in cumec) is divided by the catchment area (in m^2) to get run-off in cm/sec.

$$\text{i.e. run-off} = \frac{m^3/sec}{m^2} = m/sec = m \times 100 \text{ cm/sec}$$

Thus the ordinates of the storm hydrograph are obtained and it is prepared accordingly. From the storm hydrograph, the unit hydrograph is constructed.

Construction of Unit Hydrograph from Isolated Storm At first a discharge hydrograph is prepared from the discharge records of the catchment area. Here, the discharge (i.e. run-off) is expressed in cumec. This run-off is converted to cm/sec (as explained earlier) to get the ordinates of storm hydrograph and then storm hydrograph is prepared. From the storm hydrograph the average depth of run off is calculated, suppose it is D. Now, to get the ordinates of unit hydrograph the respective ordinates of storm hydrograph is multiplied by a factor $1/D$. The reduced ordinates are plotted at respective points to get the shape of the unit hydrograph.

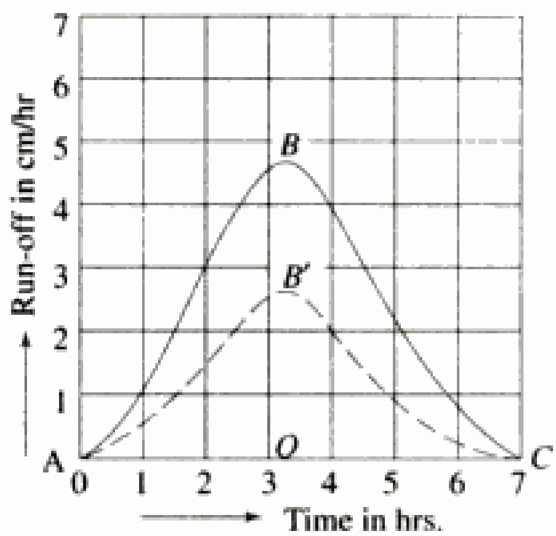


Fig. 3.5 Storm hydrograph

Example In a typical 4 hr storm producing 5 cm of run-off from a basin, the flows in the stream area as follows:

Time in hr	*Flow in cumec*
0	0.0
2	1.25
4	4.25
6	6.75
8	5.60
12	3.50
16	1.35
20	1.0

Plot the unit hydrograph for this storm.

Solution The storm produces 5 cm run-off. The hydrograph is drawn with the given data (Fig. 3.6). Here, the multiplying factor is 1/5.

The ordinates of the given hydrograph is to be multiplied by 1/5 to obtain the respective ordinates of unit hydrograph.

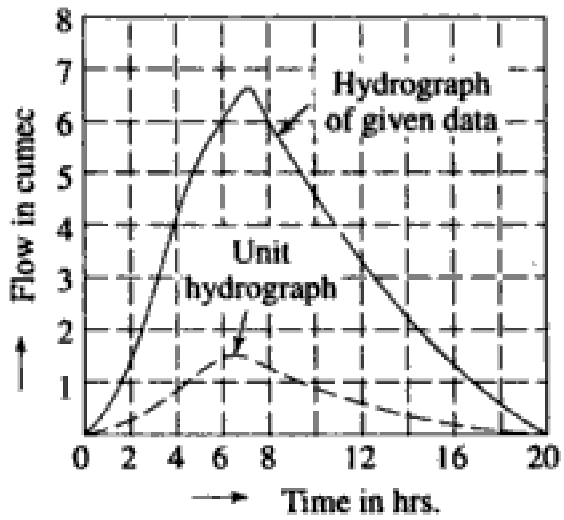


Fig. 3.6 Unit hydrograph

Now, ordinate at 0 = 1/5 × 0.00 = 0.00
ordinate at 2 = 1/5 × 1.25 = 0.25
ordinate at 4 = 1/5 × 4.25 = 0.85
ordinate at 6 = 1/5 × 6.75 = 1.35
ordinate at 8 = 1/5 × 5.60 = 1.12
ordinate at 12 = 1/5 × 3.50 = 0.70
ordinate at 16 = 1/5 × 1.35 = 0.27
ordinate at 20 = 1/5 × 0.00 = 0.00

The calculated ordinates are plotted at respective points. The points are joined to get the shape of unit hydrograph (as shown by dotted line in Fig. 3.6).

Construction of Unit Hydrograph for Other Durations The unit hydrograph for different duration may be constructed by the principle of superposition. Suppose, it is required to construct a 9 hr unit hydrograph from 3 hr unit hydrograph. Then 3 hr unit hydrograph is plotted thrice with a time lag of 3 hr. The ordinates of those three overlapping unit hydrograph are summed up to obtain a summation hydrograph of 9 hr. The ordinates of 9 hr summation hydrograph are divided by 3 to obtain the ordinates of 9 hr unit hydrograph.

Example The ordinates of 3 hr unit hydrograph is given in the table. Construct a 9 hr unit hydrograph from it.

Time (hr)	0	3	6	9	12	15	18	21	24	27	30
Discharge (cumec)	0	9	20	25	18	10	8	4	2	1	0

Solution The 3 hr unit hydrograph is arranged thrice with time lag of 3 hr then the ordinates are summed up to get the ordinates of 9 hr summation hydrograph. The ordinates of 9 hr summation hydrograph are divided by 3 to obtain the ordinates of 9-hr unit hydrograph (Table 3.1).

Now, with the ordinates shown in the Column 5 a graph is drawn which represents the summation hydrograph of 9 hr [shown in Fig. 3.7(a)].

With the ordinates shown in the Column 6 a graph is drawn which resprents 9 hr unit hydrograph [shown in Fig. 3.7(b)].

Table 3.1

Time (hrs)	*3 hr unit hydrograph ordinate*	*3 hr unit hydrograph with 3 hr offset*	*3 hr unit hydrograph with 6 hr offset*	*Summation hydrograph*	*9 hr unit hydrograph ordintes*
1	*2*	*3*	*4*	*5 = (2 + 3 + 4)*	*6*
0	0	–	–	0	0
3	9	0	–	9	3
6	20	9	0	29	9.67
9	25	20	9	54	18
12	18	25	20	63	21
15	10	18	25	53	17.67
18	8	10	18	36	12
21	4	8	10	22	7.67
24	2	4	8	14	7.33
27	1	2	4	7	2.33
30	0	1	2	3	1
33	–	0	1	1	0.33
36	–	–	0	0	0
	97.0	97.0	97.0	291.0	97.0

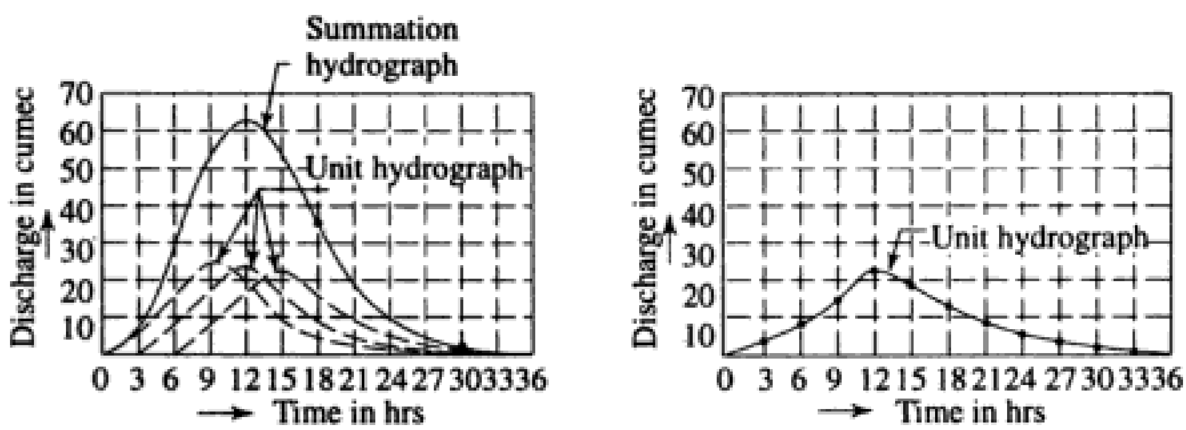
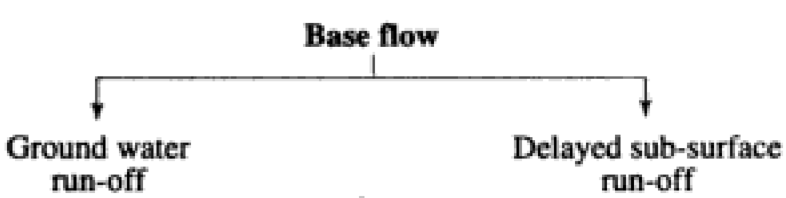


Fig. 3.7 (a) Summation hydrograph (b) hour unit hydrograph

3.8 BASE FLOW



The ground water contribution to the stream is known as base flow. It consists of two portions, (i) the ground water directly flows to the stream (ii) the rain water first infiltrates into the ground and then flows laterally to the stream even after the precipitation has stopped (Fig. 3.8).

Negative Base Flow When the water level of the stream is lower than the water table before the commencement of heavy rainfall, the ground water flows to the stream. But when the water level of the stream rises above the water table during the heavy rainfall, the stream water flow towards the ground water. This is known as negative base flow (Fig. 3.9).

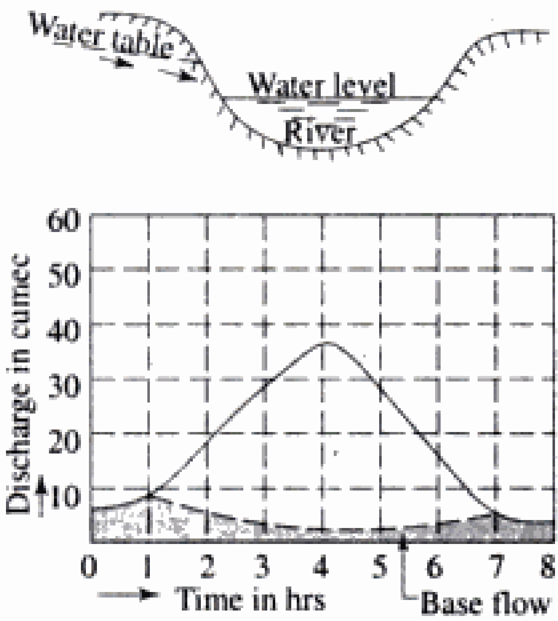


Fig. 3.8 Base flow

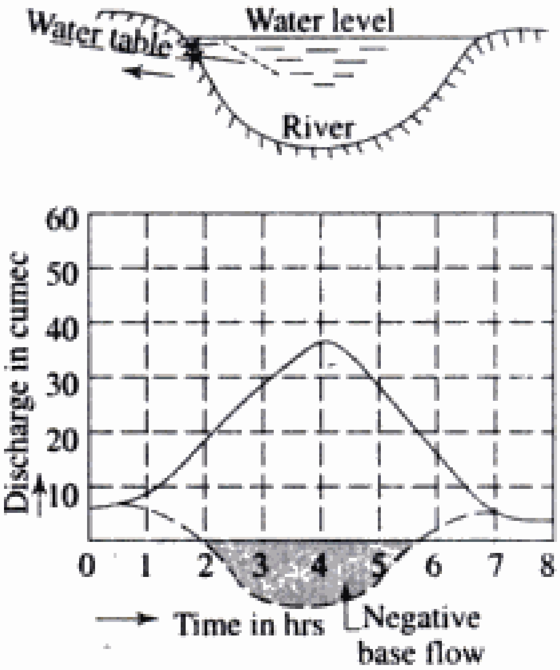


Fig. 3.9 Negative base flow

Separation of Base Flow

From the given data (discharge and time) the hydrograph is drawn which represents the total run-off which comprises direct run-off (see Sec. 3.5). The base flow is to be separated to get the actual run off. The base flow separation is described the following text.

(i) The curve *AB* is extended to meet the vertical line (passing through the point '*C*' at peak flow) at point *F*. The line *FD* is joined. The area of the dotted portion indicates the base flow (Fig. 3.10).

(ii) Again the point '*D*' is obtained on the recession limb of the hydrograph at *N* days after the peak flow, the value of *N* is calculated as,

$$N = 0.83\, A^{0.2}$$

where, A = Area of drainage basin in sq. km.

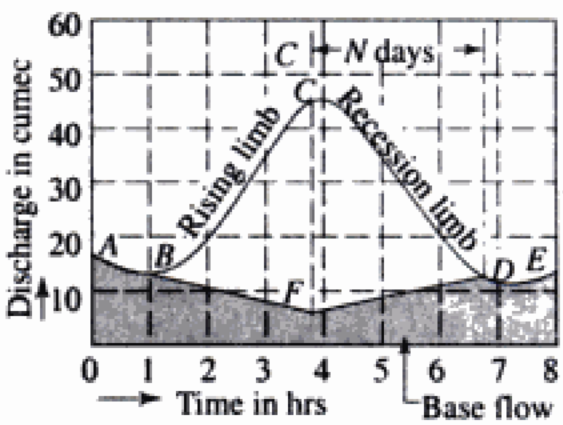


Fig. 3.10 Separation of base flow

3.9 PRECIPITATION AND ITS MEASUREMENT

Definition From the principle of hydrologic cycle we have seen that water goes on evaporation continuously from the water surfaces on earth, (e.g. river, lake, sea, ocean, etc), by the effect of sun. The water vapour goes on collecting in the atmosphere up to a certain limit. When this limit exceeds and the temperature and pressure fall to a certain value, the water vapour will get condensed and thereby cloud is formed. Ultimately droplets are formed and returned to earth in the form of rain, snowfall, hail, etc. This is known as precipitation.

Types of Precipitation or Rainfall

Depending upon the various atmospheric conditions the precipitation may be of the following types.

1. Cyclonic Precipitation This type of precipitation is caused by the difference of pressure within the air mass on the surface of the earth. If low pressure is generated at some place the warm moist air from the surrounding area rushes to the zone of low pressure with violent force. The warm moist air rises up with whirling motion and get condensed at higher altitude and ultimately heavy rainfall occurs. This may be of two types.

(a) Frontal Precipitation When the moving warm moist air mass is obstructed by the zone of cold air mass, the warm moist air rises up (as it is lighter than cold air mass) to higher altitude where it gets condensed and heavy rainfall occurs (Fig. 3.11). This is known as frontal precipitation.

(b) Non Frontal Precipitation When the warm moist air mass rushes to the zone of low pressure from the surrounding area, a pocket is formed and the warm moist air rises up like a chimney towards higher altitude. At higher altitude this air mass gets condensed and heavy rainfall occurs (Fig. 3.12). This is known as non frontal precipitation.

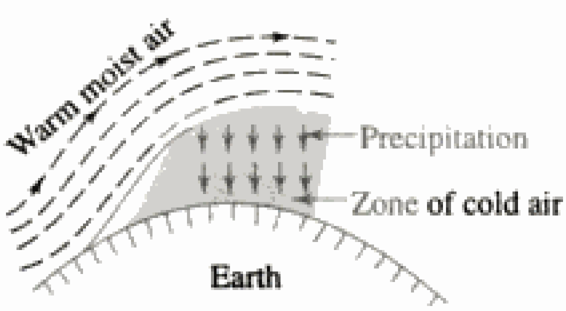


Fig. 3.11 Frontal precipitation

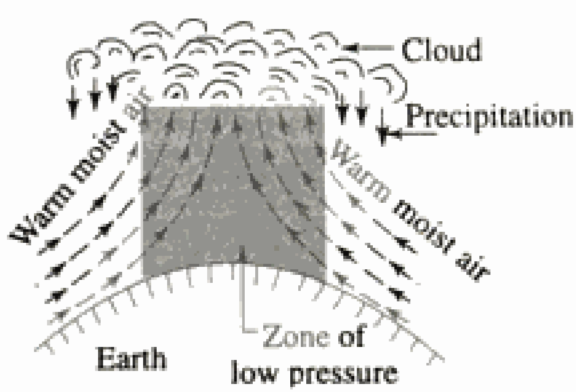


Fig. 3.12 Non frontal precipitation

2. Convective Precipitation In tropical countries when on a particular hot day the ground surface gets heated unequally, the warm air is lifted to high altitude and the cooler air takes its place with high velocity. Thus, the warm moist air mass is condensed at the high altitude causing heavy rainfall. This is known as convective precipitation (Fig. 3.13).

3. Orographic Precipitation The moving warm moist air when obstructed by some mountain rises up to a high altitude. It then gets condensed and precipitation occurs. This is known as orographic precipitation (Fig. 3.14).

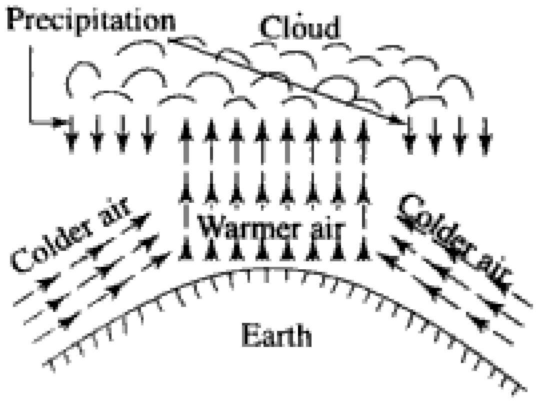


Fig. 3.13 Convective precipitation

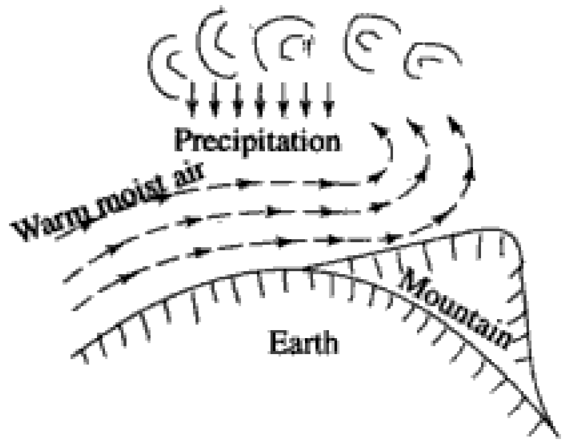


Fig. 3.14 Orographic precipitation

Measurement of Rainfall (i.e. Precipitation)

The instrument which is used to measure the amount of rainfall is known as raingauge. The principle of raingauge is that the amount of rainfall in a small area will represent the amount of rainfall in a large area provided the meteorological characteristics of both small and large area are similar. The raingauges are of the following types.

1. Non-Recording Type Raingauge Simon's raingauge is a non-recording type of raingauge which is most commonly used. It consists of metal casing of diameter 127 mm which is set on a concrete foundation *A* glass bottle of capacity about 100 mm of rainfall is placed within the casing. A funnel with brass rim is placed on the top of the bottle. The arrangement is shown in Fig. 3.15.

The rainfall is recorded at every 24 hours. Generally, the measurement is taken at 8.30 a.m. everyday. In case of heavy rainfall the measurement should be taken 2 or 3 times daily so that the bottle does not overflows. To measure the amount of rainfall the glass bottle is taken off and the collected water is measured in a measuring glass, and recorded in the raingauge record book. When the glass bottle is taken off it is immediately replaced with a new bottle of same capacity.

2. Recording Type Raingauge In this type of raingauge, the amount of rainfall is automatically recorded on a graph paper by some mechanical device (Fig. 3.16). Here, no person is required for measuring the amount of rainfall from the

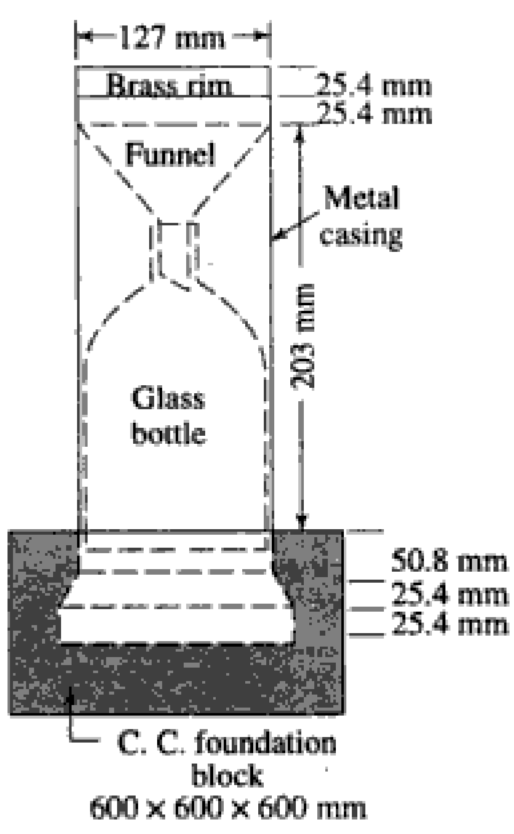


Fig. 3.15 Simon's raingauge

container in which the rain water is collected. The recording type raingauge may be of three types.

(a) Weighing Bucket Raingauge This type of raingauge consists of a receiving bucket which is placed on pan. The pan is again fitted with some weighing mechanism. A pencil arm is pivoted with the weighing mechanism in such a way that the movement of the bucket can be traced by a pencil on the moving recording drum. So, when the water is collected in the bucket the increasing weight of water is transmitted through the pencil which traces a curve on the recording drum (Fig. 3.17). The raingauge produces a graph of cumulative rainfall versus time and hence it is some times called Integrating raingauge. The graph is known as the mass curve of rainfall (Fig. 3.16).

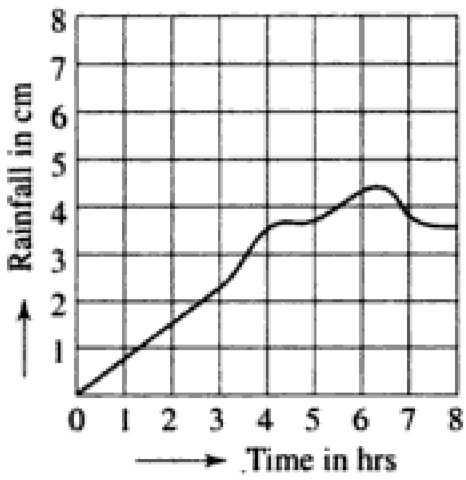


Fig. 3.16 Rain recording graph

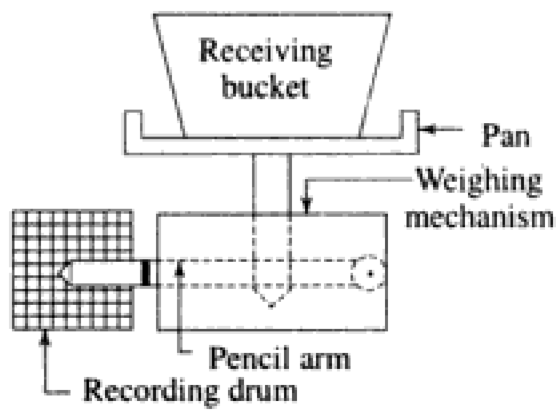


Fig. 3.17 Weighing bucket rain gauge

(b) Tipping Bucket Raingauge It consists of a circular collector of diameter 30 cm in which the rain water is initially collected. The rain water then passes through a funnel fitted to the circular collector and gets collected in two-compartment tipping buckets pivoted below the funnel (Fig. 3.18).

When 0.25 mm rain water is collected in one bucket then it tips and discharges the water in a reservoir kept below the buckets. At the same time the other bucket comes below the funnel and the rain-water goes on collecting in it. When the requisite amount of rainwater is collected, it also tips and discharges the water in the reservoir. In this way, a circular motion is generated by the buckets. This circular motion is transmitted to a pen or pencil which traces a wave like curve on the sheet mounted on a revolving drum. The total rainfall may be ascertained from the graph. There is an opening with stopcock at the bottom of the reservoir for

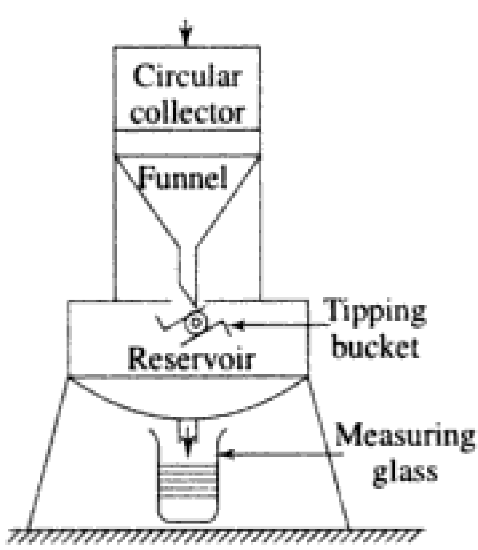


Fig. 3.18 Tipping Bucket raingauge

discharging the collected rainwater. Sometimes a measuring glass is provided to verify the results shown by the graph.

(c) Float Type Raingauge In this type of rain gauge, a funnel is provided at one end of a rectangular container and a rotating recording drum is provided at the other end. The rain water enters the container through the funnel. A float is provided within the container which rises up as the rain water gets collected there. The float consists of a rod which contains a pen arm for recording the amount of rainfall on the graph paper wrapped on the recording drum. It consists of a syphon which starts functioning when the float rises to some definite height and the container goes on emptying gradually (Fig. 3.19).

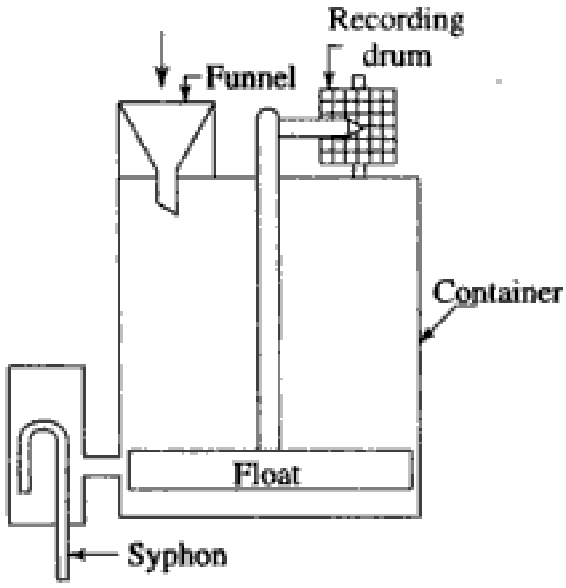


Fig. 3.19 Float type raingauge

Selection of Site for Raingauge Stations The following points should be considered while selecting a site for rain gauge station.

(a) The site should be on level ground and on open space. It should never be on sloping ground.
(b) The site should be such that the distance between the gauge station and the objects (like tree, building etc) should be at least twice the height of the objects.
(c) In hilly area, where absolutely level ground is not available, the site should be so selected that the station may be well shielded from high wind.
(d) The site should be easily accessible to the observer.
(e) The site should be well protected from cattles by wire fencing.

Network of Raingauges The network of raingauge stations should be properly designed to cover the whole area of the basin. A guide line has been set up by the World Meteorological Organisation (WMO) for the network of raingauge stations.

(a) For plain region—one station for every 600–900 sq. km.
(b) For mountainous region—one station for every 100–250 sq. km.
(c) For arid region—one station for every 1500–10000 sq. km.

3.10 AVERAGE DEPTH OF PRECIPITATION

One raingauge station cannot represent a large basin. So, a basin is always composed of many rainguage stations which are evenly distributed throughout the whole basin. Again, the amount of rainfall may not be equal in all raingauge stations. Since, the average rainfall of the basin is required for estimating the run-off from the basin, it is customary to apply any suitable method to determine the average depth of precipitation. The following three methods are generally adopted to calculate the average depth of precipitation.

1. Arithmetic Mean Method This method is very simple. In this method the rainfall values obtained from all the raingauge stations are added and divided by the number of stations to get the average value. Suppose, N is the number of stations and R_1, R_2, R_3 ... etc. are the rainfall values obtained from the stations.

Then, average depth of precipitation = $\dfrac{R_1 + R_2 + R_3 + \ldots}{N}$

This method is reliable to get the accurate value of average precipitation, when the raingauge stations are uniformly distributed over the basin.

2. Thiessen Polygon Method This method is highly suitable for large areas. It is based on the assumption that each raingauge station has its own domain within the basin area. That domain may be defined by geometrical construction as follows (Fig. 3.20).

(i) Suppose, A, B, C, D, E and F are the raingauge stations. All the stations are joined by dotted lines to form a number of triangles.

(ii) The perpendicular bisectors are drawn to each sides of the triangles. Thus a closed polygon *abcde* is formed which indicates the domain of the raingauge station A.

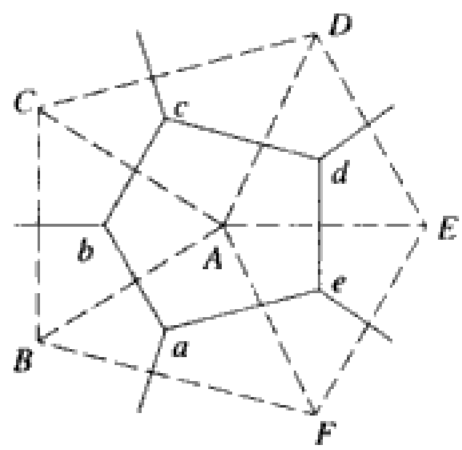


Fig. 3.20 Thiessen polygon method

(iii) Similarly, all the other raingauge stations within the basin are joined to form triangles. The perpendicular bisectors are drawn as above. Thus, a number of polygons are formed.

(iv) Geometrically, it can be proved that each polygon represents the domain of each raingauge station.

(v) The area of each polygon is measured by graph paper or by planimetre.

(vi) The result is then tabulated as follows:

Station	*Rainfall in cm*	*Area of polygon in sq. km*	*Product of rainfall and area*
(1)	(2)	(3)	(4) = (2) × (3)
A	10.5	120	1260.0
B	12.0	95	1140.0
C	9.5	105	997.5
D	8.0	125	1000.0
E	11.5	98	1127.0
F	7.5	100	750.0
G	6.0	105	630.0
		Σ 748.0	Σ 6904.5

Average depth precipitation = $\frac{6904.5}{748.0}$ = 9.23 cm.

3. Iso-hyetal Method A Iso-hyet line represents a line joining the points of equal depth of precipitation. So, it is just like a contour line. In this method, all the raingauge stations are located within the map of the required basin. Then depth of precipitation of all the stations are noted at the respective station point. Now, the Iso-hyet lines are drawn at 1 cm interval by the method of interpolation (as done during the plotting of contour lines), (Fig. 3.21). The area enclosed between the two successive Iso-hyet lines is found out by graph paper or by planimeter. The result is tabulated as shown to get the average depth of precipitation.

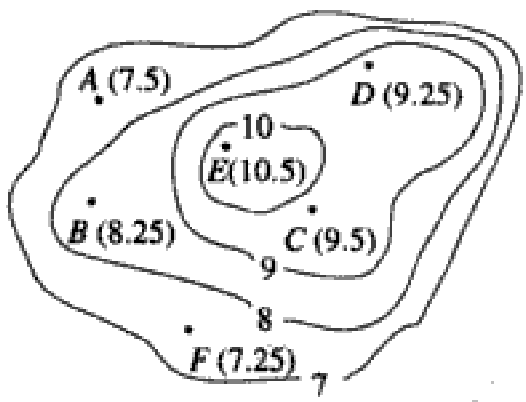


Fig. 3.21 Iso-hyet lines

Iso-hyetal interval	*Average depth*	*Area between two successive iso-hyet lines*	*Average depth × area*
(1)	(2)	(3)	(4) = (2) × (3)
10–9	9.5	*a*	9.5 × a
9–8	8.5	*b*	8.5 × b
8–7	7.5	*c*	7.5 × c
7–6	6.5	*d*	6.5 × d
		ΣA (say)	ΣB (say)

Average depth of precipitation = $\frac{\Sigma B}{\Sigma A}$ = x cm (say)

3.11 WATER LOSSES

Matter or energy cannot be lost, it only changes from one state to other. Similarly, water also cannot be lost, but it changes from one state to other, as we have seen in hydrological cycle. Here, the point 'Water losses' indicate the portion of water which cannot be observed or obtained as surface run-off directly during the period of precipitation that means,

Water losses = Precipitation – Surface run-off

Now, we are to study the causes of these losses. Under different observation it is found that the following are the main causes of water losses.

1. Interception Due to solar heat leaves, branches, trunks of trees and vegetative covers may get dried up and gain capacity of absorbing water. So, when precipitation starts some portion of water is directly absorbed by these absorbing agents. This phenomenon is known as interception. The interception continues till the leaves, branches, etc. get completely saturated. After saturation the water comes down as droplets from leaves and branches and flows down through the stem of the trees. In forest area the amount of interception is more than that of in open area. In open area the water is mainly absorbed by grass, crops, vegetables etc. The rate of interception is high at the starting of precipitation and reduces to a negligible amount if precipitation continues for longer period.

The following are the factors affecting the process of interception.

(a) Type of Vegetative Cover The tall trees in dense forest intercept quite a substantial amount of water. But in low-lying vegetative cover the interception is very low.

(b) Wind Velocity If the precipitation is accompanied with high wind, then the leaves, branches, etc. are not capable of holding much water.

(c) Duration of Precipitation if precipitation occurs for short duration with longer interval, then interception will be more. Again, if precipitation continues for longer period and the weather remains cloudy, then the interception will be less.

(d) Intensity of Precipitation If the intensity of precipitation is low, the interception will be more and *vice versa*.

(e) Season In summer or dry season the interception is more where as in winter season the interception is low.

(f) Climatic Condition In hot climate and arid region the interception is high. But in cold climate and humid region the interception is low.

2. Evaporation

It is the process of change in the state of water from liquid or solid to vapour due to the transfer of heat energy. The evaporation may be caused due to the following reasons.

(a) Evaporation from Free Water Surfaces The water from the surface of reservoirs, lakes, ponds, streams etc. is constantly changing into vapour through the process of evaporation.

The water consists of large number of molecules. These molecules are constantly moving with some velocities in different directions. Again, the molecules are attracted to each other by some forces. It is found that the molecules near the surface are easily detached from the water surface due to temperature and can easily escape into air as vapour. Thus, the molecules from the water surface constantly leave their positions and the other molecules take their place. This process of evaporation is clearly explained in Dalton's law of evaporation. The law is expressed by the expression,

$$\text{Evaporation} \propto (P_{sv} - P_{av})$$

i.e.

$$E = K\,(P_{sv} - P_{av})$$

where, E = Evaporation rate in cm/day,
K = A constant, whose value depends on wind velocity, humidity, barometric pressure, temperature etc,
P_{sv} = Saturation vapour pressure,
P_{av} = Actual vapour pressure.

Factors Affecting Evaporation from Free Water Surface The following are the factors that affect the evaporation from free water surface.

(a) ***Area of Water Surface*** If the area of water surface is large, the evaporation will be more and *vice versa.*

(b) ***Depth of Water*** If the depth of water is less, the evaporation will be more and *vice versa.*

(c) ***Humidity*** If the humidity of the atmosphere is more, the evaporation will be less and *vice versa.*

(d) ***Temperature*** If the temperature is more, the saturation vapour naturally increases and the rate of evaporation also increases and *vice versa.*

(e) ***Wind Velocity*** If the wind velocity is more, the process of vaporisation becomes easier, and loss of evaporation becomes more and *vice versa.*

(b) Evaporation from Soil Surfaces The principle of evaporation from soil surfaces is similar to that of from the free water surface the rate of evaporation from saturated soil is identical to that of from free water surface. But when the moisture content of the soil decreases, the rate of evaporation also decreases. The evaporation from the soil surface will continue as long as the soil layer (about 20 cm) remains moist.

(c) Evaporation from Vegetation (i.e. transpiration) Transpiration is practically a process of evaporation through the leaves of living plants. This process involves continuous circulation of water from the soil to the roots and then passes through the stem and finally evaporates through the leaves. This is also known as evapotranspiration. This is the principal requirement for the growth of the plants.

3. Infiltration

When it rains in a particular area, some portion of water moves downwards through the soil pores under the force of gravity to join the water table. This phenomenon is known as infiltration. At the beginning of the rain the loss of infiltration is high and then it reduces gradually as the soil pores get saturated.

Infiltration Capacity The process of infiltration depends on the number of voids present in the soil, shape and degree of compaction. So, different soils have different number of voids, different degree of compaction and hence different capacities of absorbing water. The maximum rate at which a soil is capable of absorbing water is known infiltration capacity.

Factors Affecting Infiltration Capacity The factors that affect the infiltration capacity are described in the following text.

(a) Texture of soil In coarse grained texture of soil (i.e. sand) the water infiltrates very quickly. But in fine grained texture of soil (i.e. clayey soil), the water infiltrates very slowly.

(b) Condition of soil surface If the pores of surface soil are sealed due to sedimentation of silt or due to water logging, then the water will infiltrate very negligibly.

(c) Content of soil moisture When the surface soil is dry, the rate of infiltration is high. But, if the surface soil contains moisture, the rate of infiltration is low.

(d) Type of vegetative surface cover If the surface soil is grassless, the rate of infiltration will be more. But, if the surface soil is covered with grass, then the rate of infiltration will be low.

(e) Soil temperature If the temperature of saturated soil is very low (nearly 0°C or below) the soil mass becomes impermeable. So, the rate of infiltration will be low.

(f) Agriculture Intensive agriculture on the surface soil makes it previous and hence the rate of infiltration increases.

3.12 INFILTRATION INDICES

For the assessment of the water lost by infiltration some indices are considered which are known as infiltration indices. The following are the two indices that are generally adopted for the determination of infiltration losses.

(a) ϕ-index It is defined as the average rate of rainfall, during any storm, above which the volume of rainfall is equal to the volume of direct run-off (Fig. 3.22).

(b) W-index It is defined as the average rate of infiltration which is calculated by the expression,

$$W\text{-index} = \frac{R - Q}{T_r}$$

where, R = Total rainfall,

Q = Total direct run-off,

T_r = Duration of rainfall in hrs.

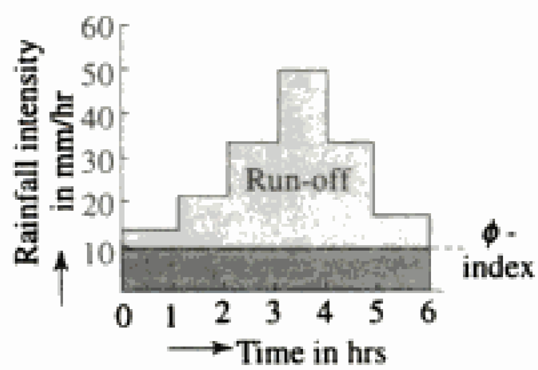


Fig. 3.22 ϕ-index

Note

(1) For uniform rainfall the values of ϕ-index and W-index will be equal.

(2) For non-uniform rainfall the values of ϕ-index and W-index will not be equal.

Examples on Infiltration Indices

Example 1 The following are the rates of rainfall for successive 30 minutes period for a storm duration of 210 minutes.

5.5., 6.0, 12.5, 8.0, 3.25, 3.25, 6.5 cm/hr

Take ϕ-index as 4.50 cm/hr calculate

(a) The run-off in cm.
(b) Total rainfall.
(c) The value of W-index.

Solution The given intensity of rainfall is plotted in a graph. Then the value of ϕ-index is superimposed. The area shown by hatched lines represents the value of run-off (Fig. S-1)

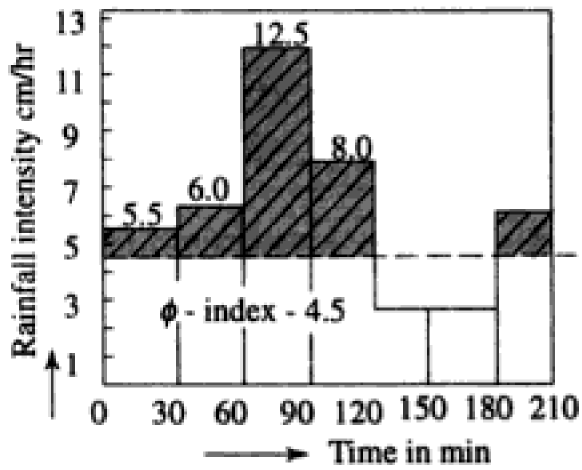


Fig. S-1

(a) Run-off $= (5.5 - 4.5) \times \dfrac{30}{60} + (6 - 4.5) \times \dfrac{30}{60} + (12.5 - 4.5)$

$$\times \frac{30}{60} + (8.0 - 4.5) \times \frac{30}{60} + (6.5 - 4.5) \times \frac{30}{60}$$

$$= 0.5 + 0.75 + 4.0 + 1.75 + 1.0$$

$$= 8 \text{ cm}$$

(b) Total rainfall =

$$= 5.5 \times \frac{30}{60} + 6 \times \frac{30}{60} + 12.5 \times \frac{30}{60} + 8 \times \frac{30}{60} + 3.25$$

$$\times \frac{30}{60} + 6.5 \times \frac{30}{60}$$

$$= 2.75 + 3.0 + 6.25 + 4.0 + 1.625 + 3.25$$

$$= 20.875 \text{ cm}$$

(c) W-index $= \dfrac{P - Q}{Tr} = \dfrac{20.875 - 8}{\dfrac{210}{60}} = \dfrac{12.875 \times 60}{210} = 3.679$ cm/hr

Example 2 The rainfall intensities for the successive one hour period for 8 hrs storm are given as,

20, 24, 30, 15, 35, 20, 10, 12 mm/hr

If the total surface run-off is 80 mm determine the value of ϕ-index.

Solution Total rainfall in 8 hrs = 2.0 + 2.4 + 3.0 + 1.5 + 3.5 + 2.0 + 1.0 + 1.2 = 16.6 cm.

Total run off in 8 hrs = 8.0 cm.

Total infiltration in 8 hrs = 16.6 – 8.0 = 8.6 cm

$$\text{Average infiltration} = \frac{8.6}{8} = 1.075 \text{ cm/hr.}$$

The ϕ–index for the 8 hrs is obtained by

$$8 \times \phi = 8.6$$

$$\therefore \quad \phi = \frac{8.6}{8} = 1.075 \text{ cm/hr}$$

Example 3 In a catchment area of 5 sq. km, the intensities of rainfall per hour for a five hour duration storm are 10, 15, 20, 22, 5 mm the volume of direct run-off is measured as 0.50 cumec-day determine the ϕ-index for the catchment area.

Solution Total rainfall in 5 hrs = 10 + 15 + 20 + 22 + 5 = 72 mm

$$\text{Average rate of rainfall} = \frac{72}{5} = 14.4 \text{ mm/hr.}$$

Total volume of rainfall over the catchment area

$$= \frac{72}{1000} \times 5 \times 10^6 = 360{,}000 \text{ m}^3$$

Total volume of direct run-off

$$= 0.50 \times 60 \times 60 \times 24 = 43200 \text{ m}^3$$

$$\text{Volume of water lost} = 360{,}000 - 43200 = 316800 \text{ m}^3$$

So, depth of water over the catchment

$$= \frac{316800}{5 \times 10^6} = 0.063360 \text{ m}$$

$$\phi\text{-index} = 63 \text{ mm}$$

3.13 FACTORS AFFECTING RUN-OFF

The following are the factors affecting the run-off.

(a) Intensity of Rainfall If the intensity of rainfall is more, the corresponding run-off will be more. Again if the intensity of rainfall is low, the corresponding run-off will also be low.

(b) Soil Characteristics of Catchment In the catchment area consisting of rocky or clayey soil, the runoff will be more. Again, if the soil characteristic of the catchment is sandy, the runoff will be low as the loss of infiltration is more.

(c) Topography of the Catchment If the ground slope of the catchment is steep, the runoff will be more. If the ground slope is flat and consists of depressions, the run-off will be low.

(d) Shape and Size of Catchment If the catchment area is large and fan shaped, the runoff will be more. If the catchment area is small and fern shaped, the runoff will be low (Fig. 3.23).

(e) Geological Condition of Catchment If the catchment area consists of fissures, cracks, etc, the water losses will be more and the run-off will be low.

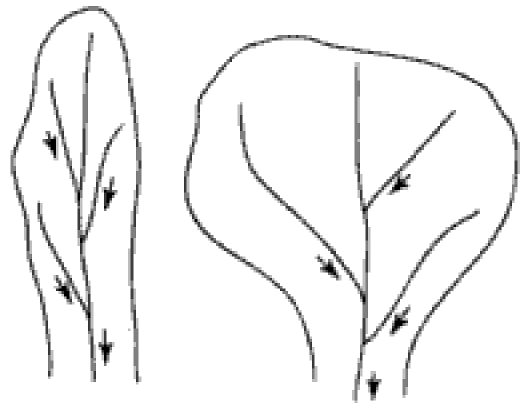


Fig. 3.23 Shape of catchment

(f) Cultivation and Vegetative Cover in Catchment Area If the catchment area consists of more cultivated area and forest areas, the runoff will be low.

(g) Weather Condition If the temperature in the catchment area is high, the evaporation loss will be more and hence run-off will be less and *vice versa*.

3.14 ESTIMATION OF RUN-OFF

Run-off may be estimated or computed by the following mathods.

A. Rational Method

By this method the runoff is computed by the expression,

$$Q = \frac{KiA}{36}$$

where, Q = Runoff in cumec
K = Coefficient of run-off,
i = Rainfall intensity in cm/hr,
A = Catchment area in hectares.

Area	*Value of K*
(1) Urban area	0.40–0.80
(2) Cultivated ares	0.30–0.70
(3) Pasture	0.10–0.40
(4) Forest area	0.10–0.40

B. Run-off Formulae

The following empirical formulae may be adopted to compute the runoff

(a) Inglis's Formula *For ghat areas*

$$R = 0.85\,P - 30.5$$

where, R = runoff in cm; P = rainfall in cm.

For non-ghat areas

$$R = \frac{(P - 17.8)}{254} \times P$$

where, R = run-off in cm,

P = rainfall in cm.

(b) Lacey's Formula

where, $$R = \frac{P}{1 + \dfrac{304.8F}{P \times S}}$$

R = Run-off in cm,

P = Rainfall in cm,

F = Monsoon duration factor varies from 0.50 to 1.50,

S = Catchment factor varies from 0.25 to 1.70.

(c) Khosla's Formula

$$R = P - \frac{T - 32}{3.74}$$

Where,

R = run-off in cm,

P = rainfall in cm,

T = Mean temperature in °F on the catchment area.

C. Infiltration Method

The rainfall records from the different raingauge stations in the catchment area are collected and then the average rate rainfall is determined by suitable method. From the average rate of rainfall a hyetograph is prepared for the catchment area. The infiltration capacity of the soil is also determined by any suitable method (by infiltrometer and rain simulator). Then infiltration capacity curve is prepared. On the hyetograph the infiltration capacity curve is supreimposed to get the quantity of run off (as shown in Fig. 3.24).

Again, ϕ–index for the catchment area is ascertained by various experiments. On the prepared hyetograph from the available rainfall records, the ϕ-index is superimposed to get the probable amount of runoff from the catchment area (Fig. 3.25).

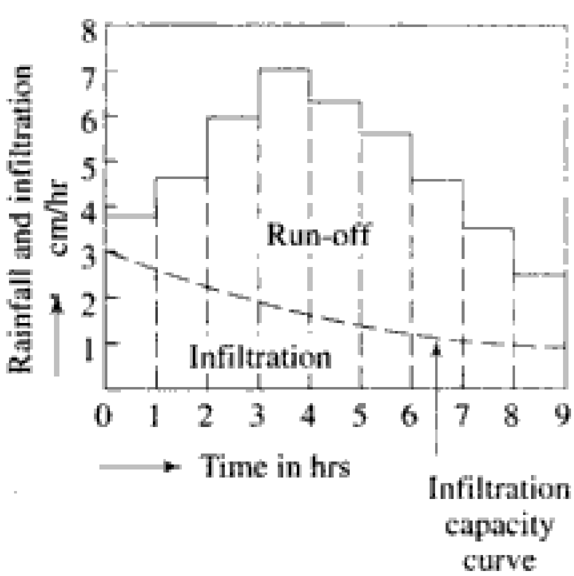


Fig. 3.24 Run-off and infiltration capacity

D. Unit Hydrograph Method

The unit hydrograph is practically a kind of unit measurement of run-off occurring in some specific unit duration (i.e. 2 hr, 3 hr ..., etc.). Again a unit hydrograph is a graph of 1 cm of direct run-off during some unit time. So, the unit hydrograph of different duration are prepared and kept ready for the instant calculation of direct run-off by noting the amount of run-off for that unit period. As for example, if a run-off of 2 cm is noted for a unit duration of two hours, then the ordinates of the 2 hrs unit hydrograph will doubled to get the run-off hydrograph for that specific period. Thus, with the help of unit hydrograph, the total run-off from any catchment area can be ascertained (Fig. 3.26).

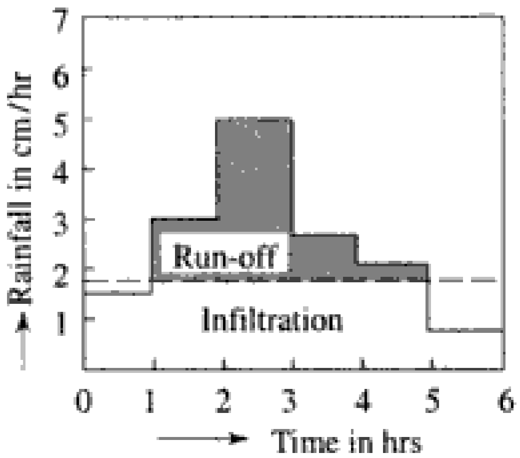


Fig. 3.25 Run-off and ϕ-index

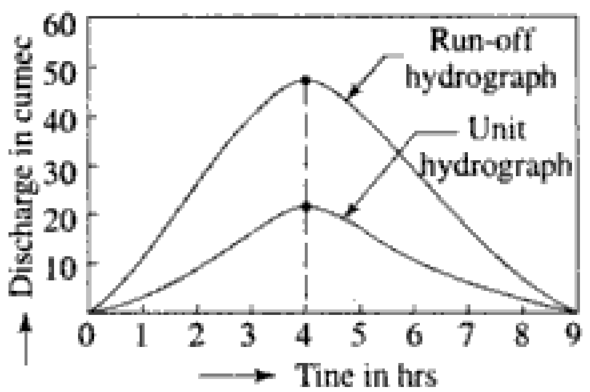


Fig. 3.26 Unit hydrograph method

3.15 S-HYDROGRAPH

S-hydrograph may be defined as a hydrograph which is constructed by summing up the ordinates of a series of unit hydrographs of same unit duration. So, it is also known as 'summation hydrograph'. This hydrograph represents the direct surface discharge resulting from successive storms of 1 cm in unit duration (i.e. 2 hr, 3 hr, 4 hr ..., etc.), for longer period. This hydrograph has continuously rising limb in the form of letter *S* until a constant value of discharge is reached. Let the base time (i.e. duration) of unit hydrograph be *T* and unit duration be t_r, the discharge will be constant at a time equal to $(T - t_r)$ hours (Fig. 3.27). Again the constant discharge is given by,

$$Q = \frac{2.778\,A}{t_r} \qquad (1)$$

where,

Q = Constant discharge in cumec,
A = Area of catchment in sq. km,
t_r = Unit duration in hrs.

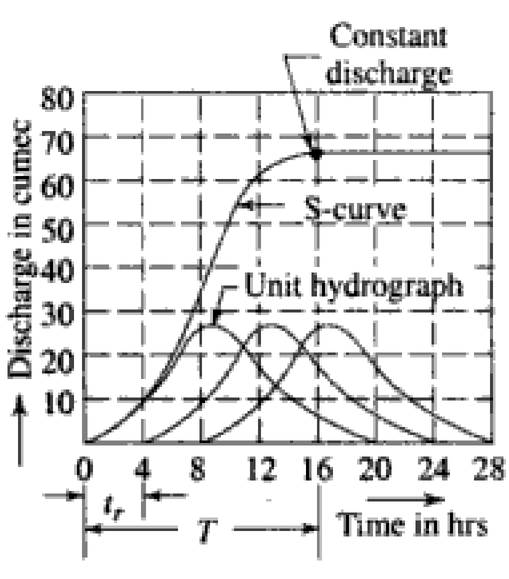


Fig. 3.27 S-hydrograph

$$Q = \frac{A}{36t_r} \qquad (2)$$

where,

Q = Constant discharge in cumec,
A = Area in hectares,
t_r = Unit duration in hrs.

Illustration The data for 2 hr unit hydrograph are given in the table below.

0	2	4	6	8	10	12	14	16	18	20
0	4.0	10.0	30.0	35.0	20.0	18.45	10.0	6.0	5.45	0

Let us draw a *S*-hydrograph for 4 hr unit duration where catchment area is 100 sq. km.

$$\text{Now, constant discharge, } Q = \frac{2.778 \times 100}{4} = 69.45 \text{ cm}$$

This constant discharge will reach at (20 – 4) = 16 hrs.

Here, the data of the 2 hr unit hydrograph are arranged as shown in Table 3.3.

Now, 2 hr unit hydrograph is drawn three times with time lag of 4 hr. Then *S*-hydrograph is drawn with ordinates available in Column 8 (Fig. 3.27).

Table 3.3 Computation of Data for *S*-Hydrograph

Time in hrs	*Ordinates of 2 hr unit hydrograph (cumec)*	*Ordinates of successive 2 hr unit hydrograph with time lag 4 hr (cumec)*					*Ordinates of S-hydrograph (cumec)*
1	*2*	*3*	*4*	*5*	*6*	*7*	*8*
0	0	–	–	–	–		0
2	4.0	–	–	–	–		4.0
4	10.0	0	–	–	–		10.0
6	30.0	4.0	–	–	–		34.0
8	35.0	10.0	0	–	–		45.0
10	20.0	30.0	4.0	–	–		54.0
12	18.45	35.0	10.0	0	–		63.45
14	10.0	20.0	30.0	4.0	–		64.0
16	6.0	18.45	35.0	10.0	0		69.45
18	5.45	10.0	20.0	30.0	4.0		69.45
20	0	6.0	18.45	35.0	10.0		69.45

3.16 ESTIMATION OF PEAK FLOW (FLOOD DISCHARGE)

The peak flow of flood discharge may be estimated by the methods described.

1. Empirical Formulae

(a) Involving Drainage Area Only

(i) Dicken's Formula

$$Q = C \times A^{3/4}$$

where, Q = Discharge in cumec; A = Catchment area in sq.km; C = A constant depending upon the factors affecting the flood discharge. An average value of C is considered as 11.5.

(ii) Ryve's Formula

$$Q = C \times A^{2/3}$$

where, Q = Discharge in cumec; A = Catchment area in sq.km; C = A constant. The average value of C is considered as 6.8.

(iii) Inglis' Formula

$$Q = \frac{123\,A}{\sqrt{A + 10.4}},$$

where, Q = Discharge in cumec; A = Catchment area in sq. km.

(iv) Nawab Jung Bahadur Formula

$$Q = C\ A'^{\left(0.92 - \frac{1}{14}\log A\right)}$$

where, Q = Discharge in cumec; C = A constant varies from 48 to 60; A = Area in sq. km.

$$A' = 0.39\,A$$

(v) Jarvis Formula

$$Q = C \times \sqrt{A}$$

where, Q = Discharge in cumec; A = Catchment area in sq.km; C = A constant maximum value is 177.

(vi) Modified Myer's Formula

$$Q = 177 \times p \times \sqrt{A}$$

where, Q = Discharge in cumec; A = Catchment area in cumec; p = A factor varies from .002 to 1. Usually p is taken as 1.

(b) Involving Drainage Area and its Shape

Dredge or Burge Formula

$$Q = 19.6 \times \frac{A}{L^{2/3}}$$

where, Q = Discharge in cumec; A = Catchment area in sq.km; L = Length of basin in km.

If B is the average width of the basin in km.

then, $A = B \times L$

or, $Q = 19.6 \times \frac{B \times L}{L^{2/3}}$

$= 19.6 \times B \times L^{1/3}$

(c) Involving Rainfall and Drainage Area Variables

$$Q = C\,(P \cdot B)^{5/4}$$

where, Q = Discharge in cumec,
P = Probable 100 years maximum one day rainfall in cm,
B = Average width of the basin in km,
C = A constant, generally taken as 1.5.

2. Boston Society of Civil Engineers Formula

$$q = 0.0056 \times \frac{D}{t}$$

where, q = Peak flow in cumec/sq.km; D = Total depth of flood run-off on the basin in cm; t = Total flood period in hrs.

Note This formula was established by considering the flood hydrograph as triangle.

where, flood hydrograph is not available, the committee suggested the following formula

$$q = \frac{C_F \times D}{A} \qquad (1)$$

or

$$Q = C_F \times D \times \sqrt{A} \qquad (2)$$

where, C_F = Coefficient of flood characteristics, generally it varies from 0.7 to 3.5; q = as before; D = as before; A = Catchment area in sq.km; Q = Peak flow in cumec.

3. Formula Involving Rainfall Intensity and Drainage Area

Rational Method

$$Q = \frac{K \times i \times A}{36}$$

where, Q = peak flow in cumec; i = Rainfall intensity in cm/hr; A = Catchment area in hectares, K = runoff coefficient.

Characteristics of catchment	*Value of K*
Sandy oil	0.10 to 0.20
Clayey soil	0.90
Loamy soil	0.40
Gravel paving	0.15 to 0.30
Asphalt paving	0.50
Concrete paving	0.80

3.17 FLOOD ROUTING

Flood routing may be defined as a technique of determining the flood discharge and arresting the same in a reservoir for some period so that the sudden flood discharge may not create devastation in the down stream areas. It is hence a process of flood control works. By flood routing the maximum water level (i.e. full reservoir level) in the reservoir that may attain can be determined by only studying the rainfall records and previous flood hydrographs. Again, by flood routing the peak flow at the up stream side of the reservoir is controlled in such a way that the flow at the downstream side is reduced to safe discharge. (Fig. 3.28).

It is seen from the hydrograph that the upstream flood hydrograph has higher peak flow with shorter base (*AB*) but at the downstream hydrograph has lower peak flow with broader base (*AC*). It is achieved by detaining the flood water for some longer period by closing the spillways and then the excess flood water is released gradually by opening the spill ways.

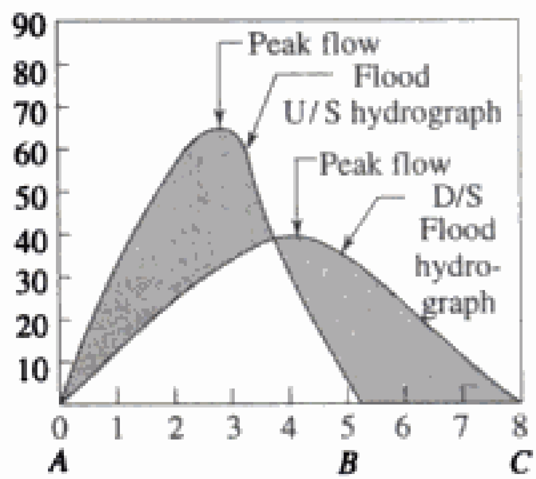


Fig. 3.28 Flood routing

Procedure of Flood Routing

There are various methods of flood routing:

(a) Flood Routing through Reservoir In normal cases, the rain water from the catchment area enters the reservoir and the water level reaches up to normal pool level, the excess water flows through the spillways. But due to the excessive rainfall during any period the discharge is highly increased and the water level exceeds the normal pool level and reaches up to maximum reservoir level. This volume of water is absorbed temporarily for some period and then allowed to flow to the down stream through the spillways (Fig. 3.29). So, the flood routing involves,

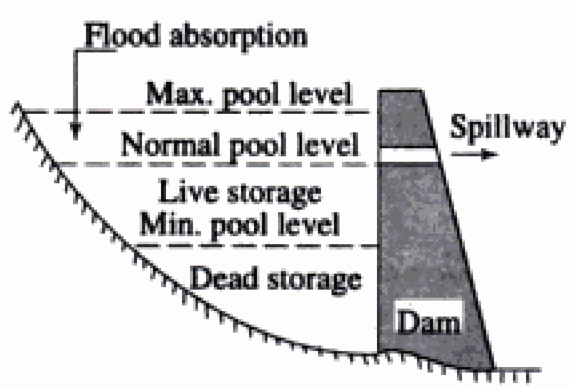


Fig. 3.29 Flood routing through reservoir

(1) The fixation of maximum reservoir level up to which the structure is completely safe.

(2) Implementation of outflow pattern from the reservoir so that it may not create any danger in the down stream side.

Thus, mathematically, Inflow = outflow + change in storage volume, i.e. $I = 0 + \Delta s$.

(b) Trial and Error Method of Flood Routing In this method, the inflow hydrograph is divided into some convenient intervals. Then for the first interval the following data are computed

(1) Total inflow during this interval
(2) Total outflow during this interval
(3) Reservoir storage S_1 and S_2 at the beginning and end of interval corresponding to any assumed normal pool level.
(4) Then $S_2 - S_1 = \Delta_S$. Here Δ_S represents the volume of flood absorption.
If the equation, $I = 0 + \Delta_S$ is satisfied with the computed results, then the assumed N.P.L. is correct, otherwise, the whole process is repeated until the equation is satisfied.

REVIEW QUESTIONS

1. Fill up the blanks with appropriate word/words
   (i) ______________ is the science of studying the different forms of water available on the earth.
   (ii) ______________ is the continuous process of evaporation and precipitation like an endless chain.
   (iii) The graphical representation of discharge of a river against time is known as ______________.
   (iv) The graphical representation of rainfall and run-off is known as ______________.
   (v) The hydrograph obtained from one centimetre run-off for unit duration is known as ______________.
   (vi) The ground water contribution to the stream is known as ______________.
   (vii) When stream water flows towards the ground water, then this flow is known as ______________.
   (viii) When the precipitation is caused by the difference of pressure within the air mass on the surface of the earth, then it is known as ______________.
   (ix) ______________ rain gauge is a non-recording type.
   (x) At the starting of precipetation some portion of water is absorbed by leaves and branches of trees this phenomenon is known as ______________.
   (xi) ϕ-index and W-index are known as ______________.
   (xii) The run-off will be more from ______________ shaped catchment area.
   (xiii) Flood discharge may be estimated from Dicken's formula by the expression ______________.
   (xiv) Flood discharge may be estimated from Ryve's formula by the expression ______________.

(xv) The technique of determining the flood discharge and arresting that discharge in a reservoir is known as ______________.

2. Define hydrology and enumerate the importance of it.
3. What is meant by hydrologic cycle? Explain with a neat sketch.
4. Distinguish between hydrograph and hyetrograph.
5. What is unit hydrograph? What are the advantages of unit hydrograph?
6. Distinguish between base flow and negative base flow.
7. What are the different types of precipitation? Describe with neat sketches.
8. What are the different types of raingauges? Describe any one with a neat sketch.
9. What are the different methods of computing average depth of precipitation. Describe the procedure of any one.
10. (a) What are the causes of water losses?
    (b) State the factors that affect the evaporation from free water surface.
    (c) State the factors that affect infiltration capacity.
    (d) State the factor that affect the run-off.

ANSWERS

1. (i) Hydrology
   (ii) Hydrologic cyclec
   (iii) Hydrograph
   (iv) Hyetograph
   (v) Unit hydrograph
   (vi) Base flow
   (vii) Negative base flow
   (viii) Cyclonic precipitation
   (ix) Simon's
   (x) interception
   (xi) Infiltration indices
   (xii) fan
   (xiiii) $Q = C \times A^{3/4}$
   (xiv) $Q = C \times A^{2/3}$
   (xv) Flood routing

4

LIFT IRRIGATION

4.1 INTRODUCTION

In early days, agriculture was completely dependent on the mercy of nature. In the year of sufficient rainfall, the yield of crop was satisfactory, whereas in the year of draught, the crop failed and the country had to face the curse of famine. So, for the sake of survival, man invented some techniques of lifting water from the surface sources for the irrigation of agricultural lands. These techniques were dependent on man or animal power. Those systems were suitable for small plots of lands. As the water was taken from the surface sources, the systems failed in summer.

With passage of time man invented more advanced techniques based on mechanical or electrical power. The water is drawn from the underground sources and is available throughout the year. This system also cannot cater the needs of vast area.

In village areas where the electrical power is not available or diesel pump set becomes costly, the cultivators are bound to follow the systems of lift irrigation by man or animal power.

Lift irrigation

- By man or animal power
- By mechanical or electrical power

4.2 LIFT IRRIGATION (BY MAN OR ANIMAL POWER)

Sources of Water The sources of water for the lift irrigation (by man or animal power) generally include ditches, ponds, lakes, streams, dugwells, unlined open wells, etc. In rainy season, the water from these surface sources may be available. But in dry season these sources may get dried up and water may be available from unlined open well or lined open well. Again these wells may get dried if the water table goes below the botton level.

Methods of Lifting Water Various methods can be applied for lifting the water by using man of animal power. The description of the common methods ϕ for lifting water follows:

(a) Doon It consists of a trough made of wooden or galvanised tin sheet. The trough is open at one end and closed at other. It is anchored over a fulcrum. The closed end is tied to a level with a rope. The other end of the lever carries a counter weight. The lever is pivoted at the middle point. Standing on a platform, the operator pushes the trough downwards until it is immerged in water. When the trough is filled with water, the pressure is released and the trough rises upwards. At some position the water of the trough starts getting discharged into the field channel. The operator now presses the trough again to immerge it in water. This process is repeated until the requirement is fulfilled (Fig. 4.1).

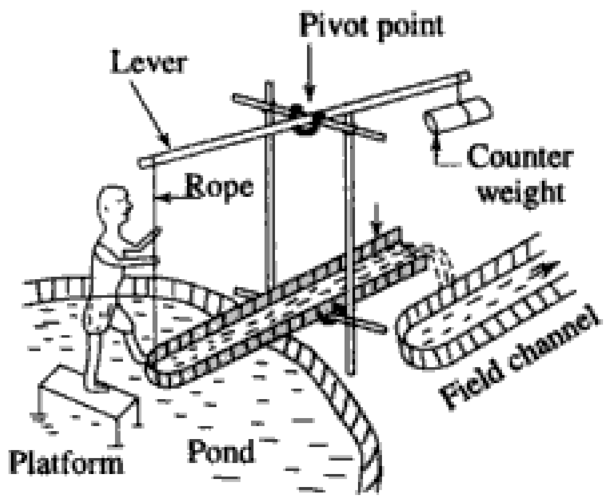


Fig. 4.1 Doon

(b) Archimedean Screw It consists of wooden or metal cylinder with a helical coil fitted within. The cylinder is placed in an inclined position and fitted with two supports with bearings so that it can be rotated by a handle. The lower end of the cylinder is kept immerged in water. When the cylinder is rotated by a man with the handle, the water gets sucked through the helical coil and follows the path. The water is then discharged into the field channel through the upper end. (Fig. 4.2).

(c) Swinging Basket In this method, the water is lifted by swinging a bucket with the help of pair of ropes by two persons standing on opposite sides. This method is suitable for lifting water to a small height. In the downward motion, the bucket is immerged in water by loosening the ropes. In the upward motion, the two persons exert maximum pressure on the ropes to raise the bucket above the water surface and then the bucket is tilted to discharge the water in to the field channel. The process is continued until the requirement is fulfilled (Fig. 4.3).

(d) Denkli This process is adopted for lifting water from the dug well. It consists of a lever pivoted on a vertical post. One end of the lever carries a counter weight and the other end carries a bucket suspended by a string. On the

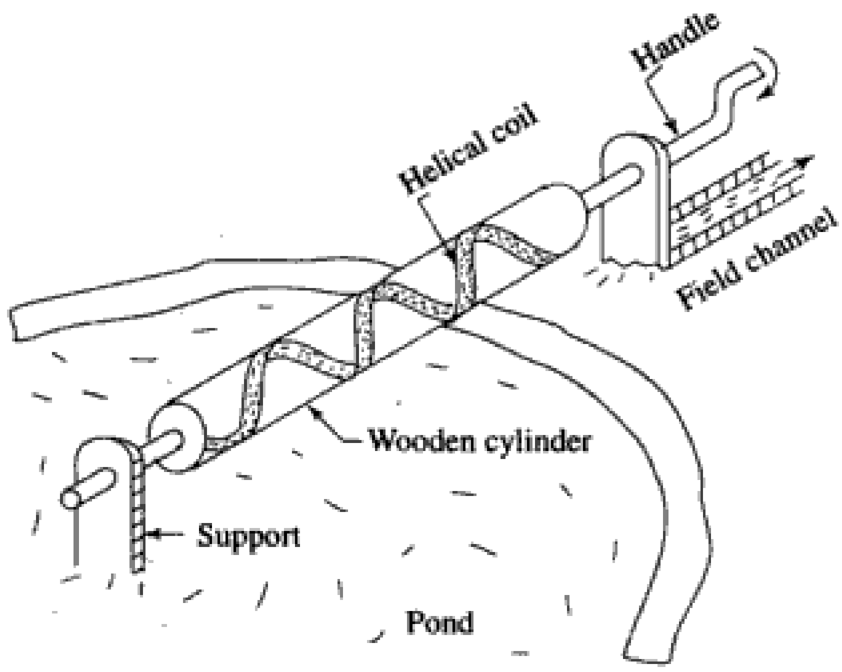


Fig. 4.2 Archimedean Screw

downward motion the operator exerts a downward thrust on the rope to immerse the bucket in the water of the well. In the upward motion, the operator releases the pressure and the bucket is automatically raised by the action of counter weight. When the bucket comes above the ground level, the operator delivers the water in the field channel. The operation is repeated till the requirement is fulfilled (Fig. 4.4).

(e) Ratti or Pulley This process is adopted for lifting water from a deep lined open well. It consists of a inclined post carrying a pulley. Over the pulley a long rope passes which carries a bucket at one end. The other end of the rope is held by the operator who can pull the bucket up and down. For the downward motion, the operator releases the rope slowly for immersing the bucket in well. Then, the operator pulls the bucket upwards to the top. The water is then discharged to the field channel merely by inverting the bucket (Fig. 4.5).

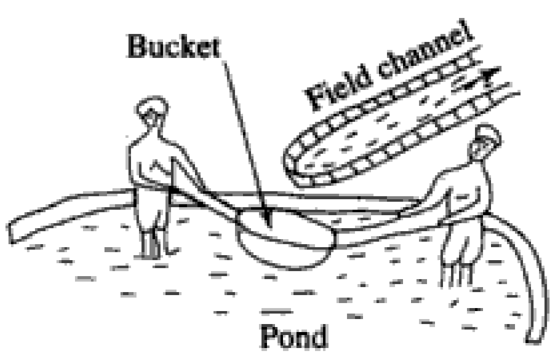


Fig. 4.3 Swinging basket

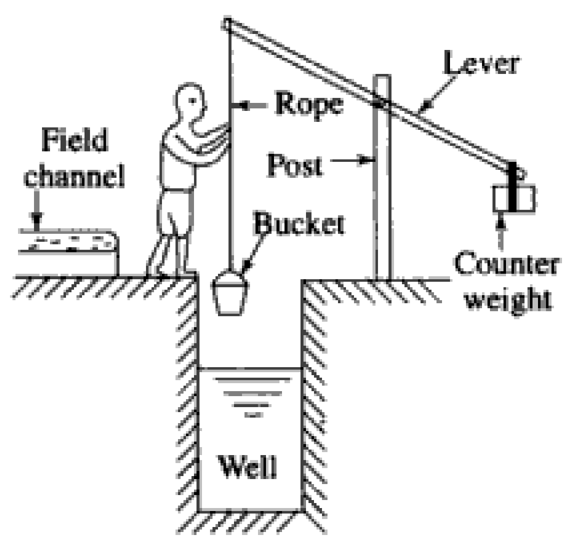


Fig. 4.4 Denkli

(f) Wind Lass This process is suitable for lifting water from deep open wells. In this process, a cylindrical wooden frame is supported in vertical posts with proper bearings. The wooden cylinder consists of two wheels at the two ends. Again, the wheels consists of four projecting arms with which the operator can rotate the wheel clockwise or anticlockwise. A rope is coiled with the cylinder. The free end of the rope carries a bucket. In anticlockwise rotation the rope is recoiled and the bucket gets lowered and gets immersed in water. In clockwise rotation of the wheel, the rope is coiled with the cylinder and the bucket get raised. When the bucket comes to the top of the well, the operator discharges the water into the field channel by inverting the bucket (Fig. 4.6).

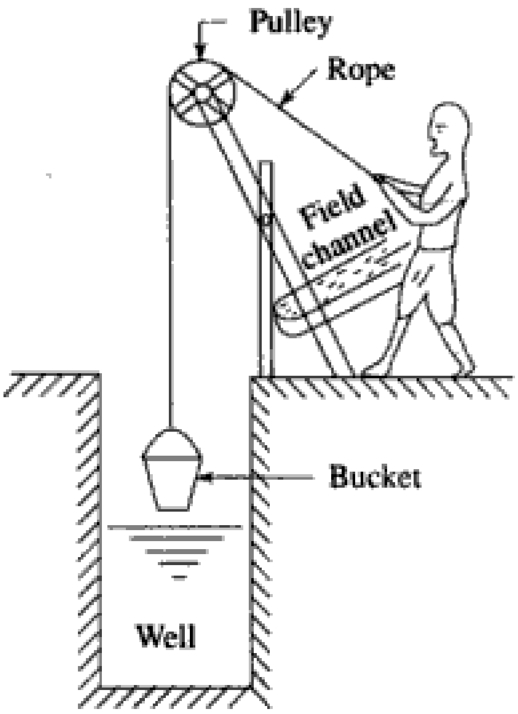


Fig. 4.5 Ratti or pulley

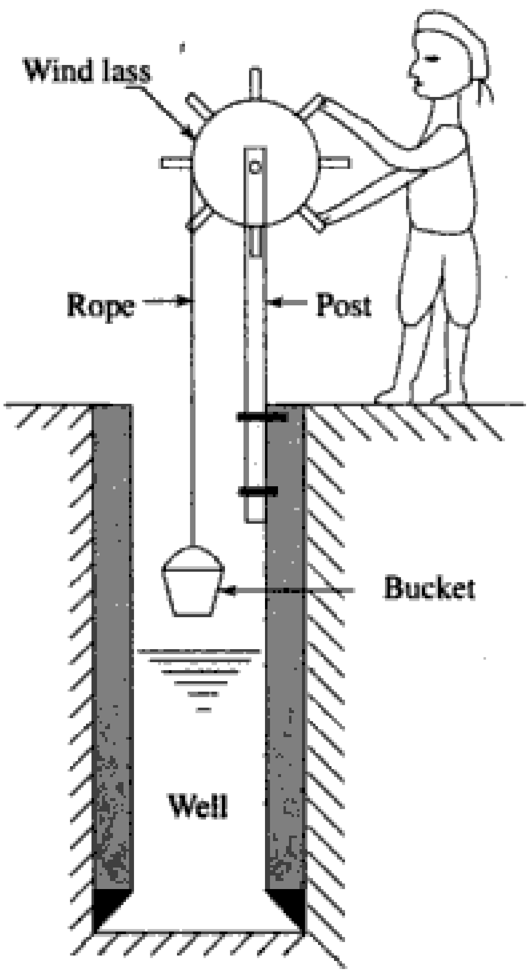


Fig. 4.6 Wind lass

(g) Mote This method is suitable for lifting water from lined deep open wells. It consists of two pulleys, where the upper pulley is large and fitted with a bracket and the lower pulley is small and fitted on the top of a strut. A lather container known as mote (in the shape of horn) is tied with ropes at both ends. One rope passes over the larger pulley and the other rope passes over the smaller pulley. Again, both the ropes are tied with a yoke which is pulled by bullocks. The bullocks move on sloping platform (*AB*) which is known as ramp. When the bullock are at position *A*, the mote gets immersed in water and remains horizontal and when the bullocks are at the position *B*, the mote becomes inclined and the water is discharged in a trough which carries the water to the field channel. The bullocks move in forward and backward directions along the ramp for lifting and discharging water. The process is continued till requirement is fulfilled (Fig. 4.7).

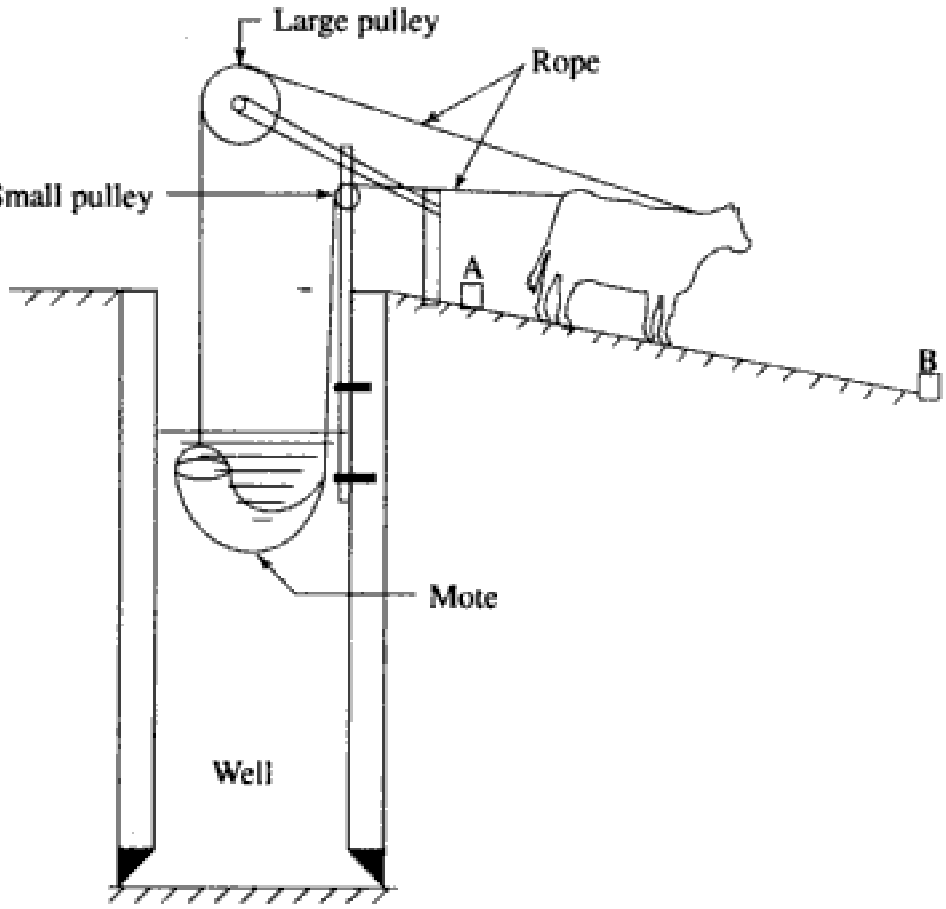


Fig. 4.7 Mote

(h) Persian Wheel This method is suitable for lifting water from lined deep open wells. It consists of a vertical axle which carries a toothed wheel. This wheel is geared with a horizontal shaft which carries a drum. An endless chain passes over the drum. A number of G.I. buckets are properly fixed on the chain. The vertical axle carries a yoke at the top on which two bullocks are yoked. As the bullocks move in a circular path, the toothed wheel rotates and the horizontal shaft also rotates due to the gear. Thus the drum rotates and the buckets move upward and downward with the chain. During downward motion the buckets are in inverted position and during the upward motion the buckets become erect and are raised full of water. During the movement of the chain, the buckets discharge water in a trough near the top of drum. the process goes on until the requirement is fulfilled (Fig. 4.8).

4.3 LIFT IRRIGATION (BY MECHANICAL OR ELECTRICAL POWER)

Sources of Water The ground water storage is the source for lift irrigation. The ground water storage is formed due to the infiltration of water through the soil voids during the process of precipitation. This storage consists of a number of layers of water bearing strata (known as aquifer) sandwiched between the impervious layers. In this system the ground water is tapped through the tube wells (shallow or deep) by diesel pump set or electrical motor pump set. This irrigation system is also known as well irrigation.

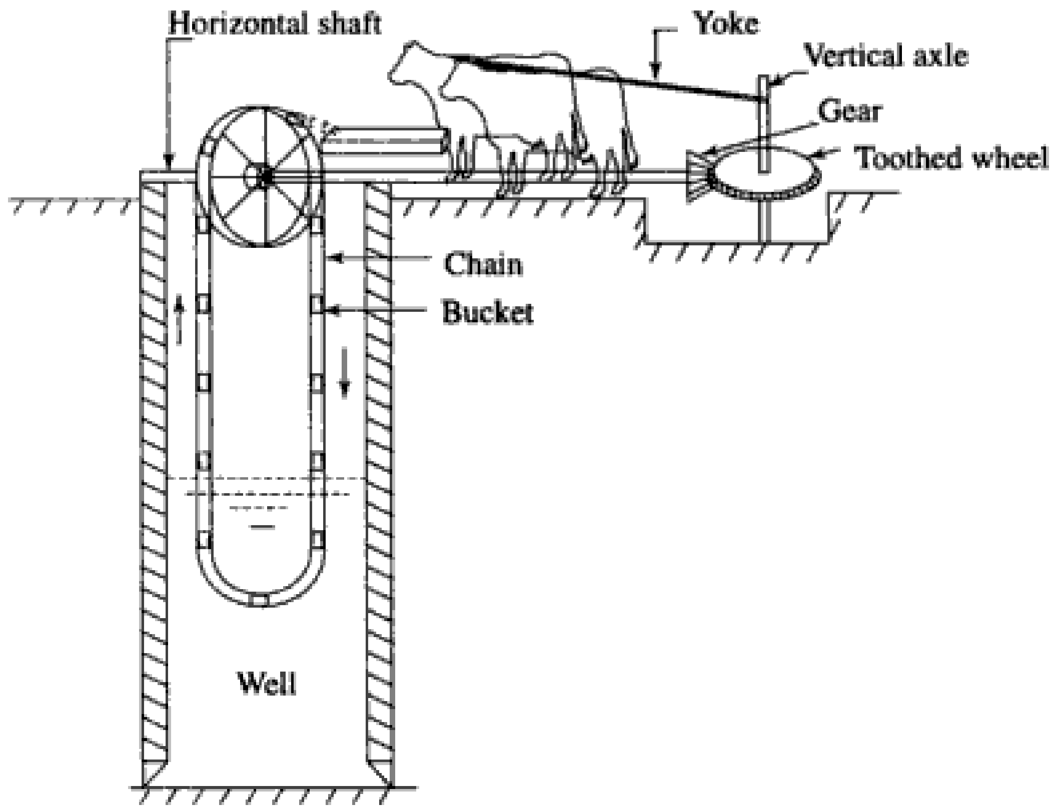


Fig. 4.8 Persian wheel

4.4 EXPLANATION OF DIFFERENT TERMS

1. Porosity

It is defined as the ratio of the volume of voids present in a soil mass to the total volume of that soil. It is expressed as percentage.

Thus, $$n = \frac{V}{V_t} \times 100$$

where, n = porosity; v = Volume of voids; v_t = Total volume of soil mass.

Porosity depends on the fineness and sharpness of the soil. Porosity will be high when particles are of uniform size. It will be low when the particles are well-graded. So, porosity indicates the capacity of a soil to contain water.

2. Coefficient of Permeability

It is defined as the rate of flow of water through the aquifer per unit cross-sectional area per unit hydraulic gradient. So, it indicates the velocity of water through the soil and expressed in cm/sec. It is denoted by K.

3. Water Table

It is the surface or a line on the soil below which the soil mass is saturated with water. This surface varies with the change of seasons. In rainy season the water surface is near the ground surface. But in dry season this surface goes down appreciably towards the impervious layer.

4. Ground Water Yield

It is defined as the quantity of water that can be extracted from the saturated aquifer by the force of gravity (i.e. by digging wells) (Fig. 4.9).

5. Ground Water Velocity

The ground water velocity through the permeable soil (i.e. aquifer) depends upon the type of soil, viscosity of water, porosity of soil, etc. The velocity of ground water can be determined by some empirical formulae,

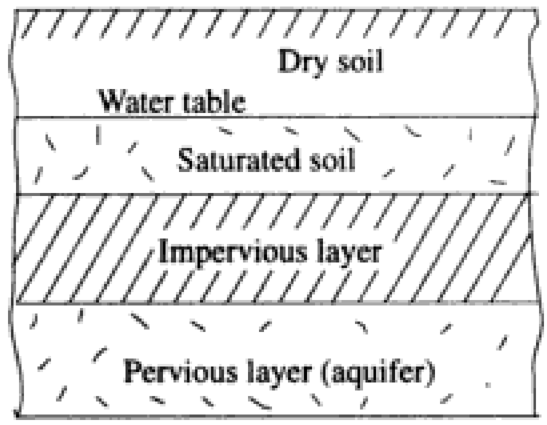


Fig. 4.9 Water table

(a) Slichter's formula

$$V = K' \cdot S \cdot \frac{D^2}{\mu}$$

where, V = Ground water velocity in m/day.

(b) Hazen's formula

$$V = \frac{K'' \cdot S \cdot D^2}{60} \times (1.8\,T + 42)$$

where, K' = A constant, approximate value 400,
K'' = A constant, approximate value 1000
S = hydraulic gradient
D = Effective particle size,
μ = Dynamic viscosity of water
T = temperature in °C

6. Aquifers or Water Bearing Strata

The permeable formation of the soil of the earth's crust is known as aquifer. It is also known as water bearing strata as the ground water occurs in this strata (Fig. 4.10). The aquifers may be of following types,

(a) Unconfined Aquifer It is the topmost aquifer in which the water table lies on the surface of saturation. So, it is also known as water table aquifer. The surface water enters this aquifer through the surface soil.

(b) Confined Aquifer The aquifer which is sandwiched between two impervious strata is known as confined aquifer.

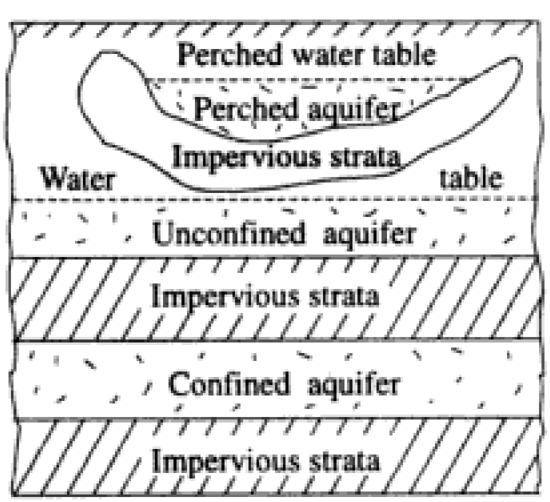


Fig. 4.10 Aquifer

(c) Perched Aquifer This too is a unconfined aquifer which is separated from the main water table by a shallow and curved impervious strata. The water table in this aquifer is known as perched water table which may be above the main water table.

7. Aquicludes

These are impermeable formations which may contain water but these are not capable of transmitting water through them. (e.g. clay).

8. Aquifuges

These are impervious formations which neither contain nor can transmit water (e.g. rocks like granite).

9. Aquitards

These are poor permeable formations which contain water but obstruct ground water movement, (e.g. sandy clay).

10. Hydraulic Conductivity

It is defined as the rate of flow of ground water with respect to a given hydraulic gradient. According to Darcy's law,

$$V = \frac{K_c\,(h_1 - h_2)}{L}$$

where, V = Velocity of ground water in m/day, K_c = Hydraulic conductivity, $(h_1 - h_2)$ = Difference of head in m, L = distance between points h_1 and h_2.

11. Specific Yield

It is defined as the ratio of the volume of water that can be drained off by gravity to the total volume of soil after attaining the point of saturation. It is also expressed as percentage.

Thus $$S_y = \frac{W_d}{V_t} \times 100$$

where, S_y = Specific yield, W_d = Volume of water drained, V_t = Total volume of soil after saturation.

12. Specific Retention

It is defined as the ratio of the volume of water that may be retained in soil, after draining due to the force of gravity, to the total volume of soil.

thus, $$S_r = \frac{W_r}{V_t} \times 100$$

where, S_r = Specific retention; W_r = Volume of water retained; V_t = Total volume of soil.

Note:
Since W_d and W_r form the total volume of water in saturated soil, porosity $n = S_y + S_r$.

13. Specific Capacity

It is defined as the discharge per unit draw down when the water is pumped out from the well. It is expressed in cumec.

Thus, specific capacity $= \dfrac{Q}{S}$

where, Q = Total discharge, in cumec; S = Total drawdown in m.

14. Efficiency of a Well

It is defined as the ratio of the actual specific capacity of a well to the theoretical specific capacity. It is expressed as percentage.

i.e. $$E_w = \frac{S_a}{S_t} \times 100$$

where, E_w = Efficiency of well; S_a = Actual specific capacity; S_t = Theoretical specific capacity.

15. Yield of a Well

The yield of a well is defined as the rate of pumping of water from the well without causing the failure of the well.

16. Spacing of Well

The spacing of wells should be such that the circle of influence of the adjacent well may not coincide. Because, due to the mutual interference, the yield of well will be reduced considerably (Fig. 4.11).

17. Cone of Depression

During pumping from an well the original water table forms a curved surface in the form of an inverted cone. This cone is known as cone of depression. The curved surface is known as drawdown curve (Fig. 4.12).

18. Circle of Influence

The base of cone of depression is known as the circle of influence. The radius of this circle is known as radius of the circle of influence (Fig. 4.12).

19. Draw Down

When water is pumped out from a well, then the original head H is reduced to

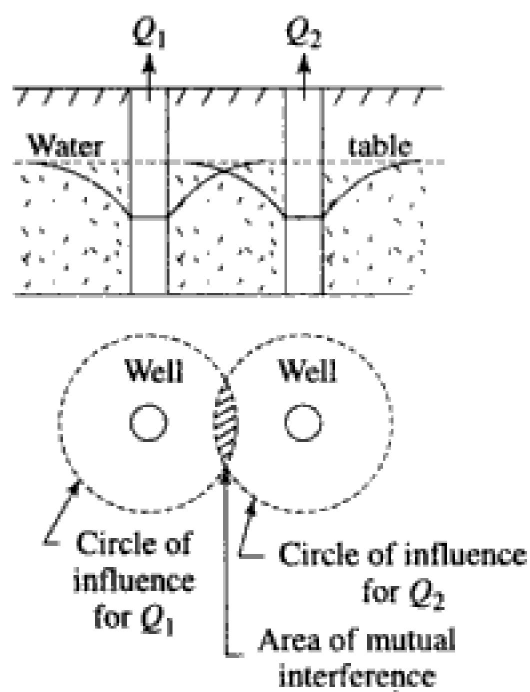


Fig. 4.11 Spacing of wells

the depressed head h and the water around the well enters into the well under the influence of the head $(H - h)$. This head is known as depression head or percolation head or drawdown (Fig. 4.12).

20. Shallow Well and Deep Well

Shallow Well The well which draws the water from the unconfined aquifer is known as shallow well. The yield of the well is dependant on the water table. In dry season when the water table goes below the bed of the well, the discharge of the well is stopped and the well is dried up (Fig. 4.13).

Deep Well The well which draws water from the confined aquifer is known as deep well. The yield of the well is not dependant on the water table. The discharge is reliable throughout the year (Fig. 4.13). In other words deep well goes below the mota layer.

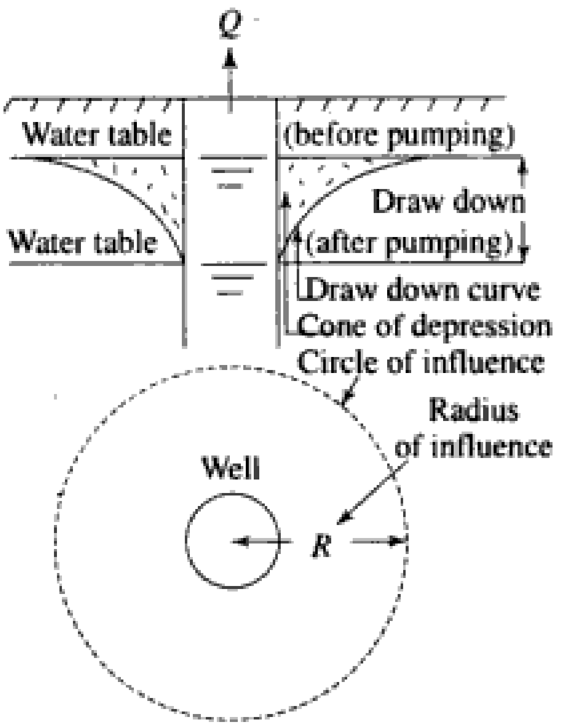


Fig. 4.12 Cone of depression

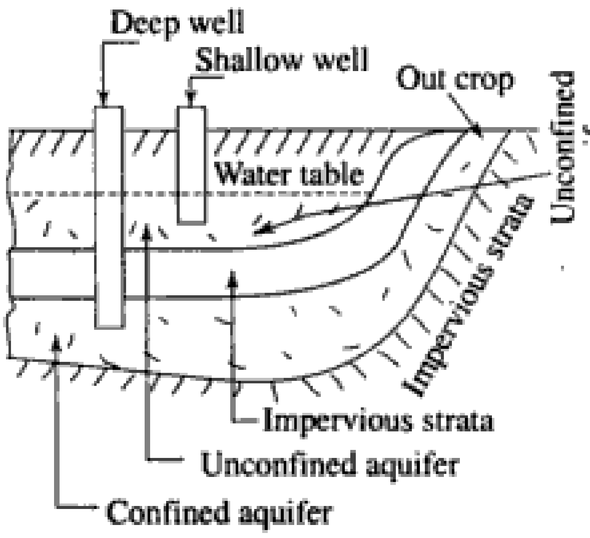


Fig. 4.13 Shallow and deep well

21. Coefficient of Transmissibility

It is defined as the rate of flow of water through the aquifer per unit width per unit hydraulic gradient covering the full saturated height of the aquifer. It is denoted by T.

Thus, $T = b\,K$. Where, b = Thickness of aquifer; K = Coefficient of permeability.

22. Interference Among Wells

When two wells are located close to each other, then their draw down curve will intersect and the discharge of one well will interfere with the discharge of the other. This is known as interference among the wells. Due to interference, the discharge of each well will be decreased, but the total discharge will be increased. The discharge of each individual well is given by,

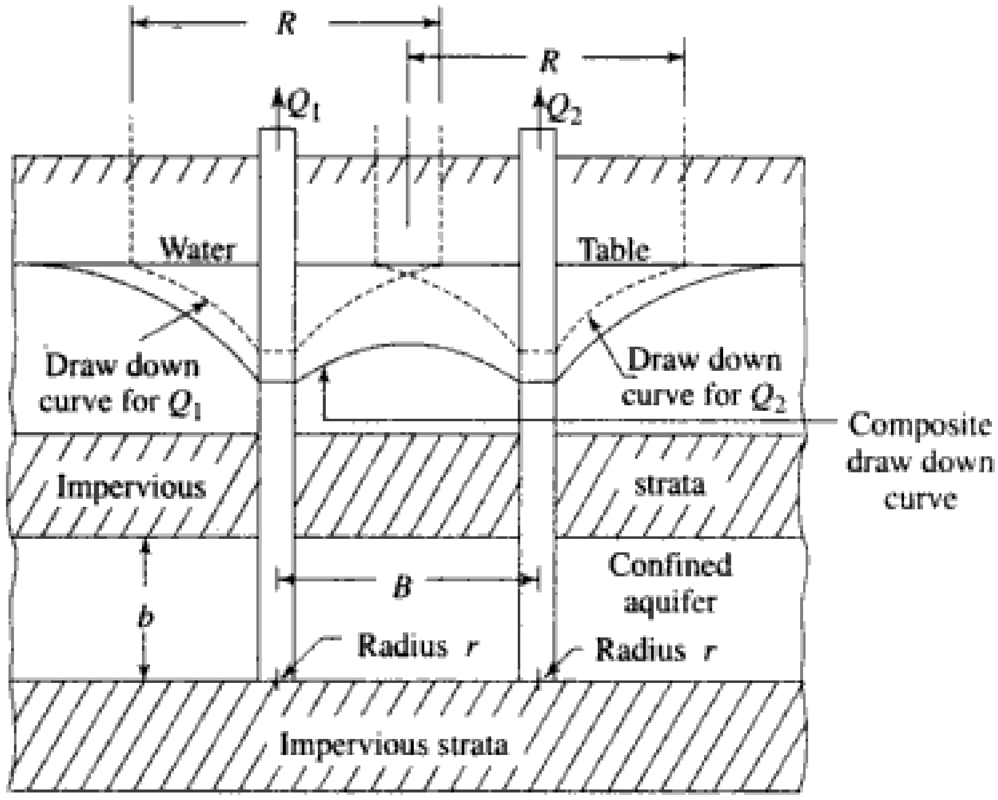


Fig. 4.14 Interference among wells

$$Q_1 = Q_2 = \frac{2\pi\, Kb\,(H - h)}{\log_e\left(\dfrac{R^2}{rB}\right)}$$

(Here $R > B$)

where, K = Coeff. of permeability; b = Thickness of aquifer; R = Radius of influence of each well; r = Radius of each well, B = Distance between wells.

4.5 CLASSIFICATION OF WELL

Well

- A. open well
- B. Tube well
  - Strainer type
  - Gavity type
  - Slotted type

A. Open Well An open well is constructed by digging the earth. Its diameter varies from 1 m to 2 m and depth varies from 10 m to 20 m. This well may be unlined or lined. The unlined open well is constructed in hard soil and in hilly areas. But the lined open well is constructed in loose soil. The lining is provided by brick masonry, stone masonry or reinforced cement concrete according to the site condition. An open well may be shallow or deep.

The shallow open well is constructed in the top permeable strata. So, it draws the surface water which infilterates into the ground through surface soil. This well is likely to get dried up in summer. But the deep well draws water from deeper permeable strata and hence the discharge is reliable [Fig. 4.16(a) and (b)].

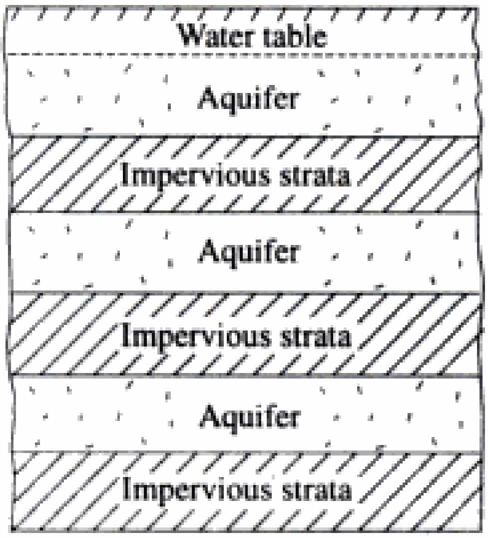


Fig. 4.15 Ground water source

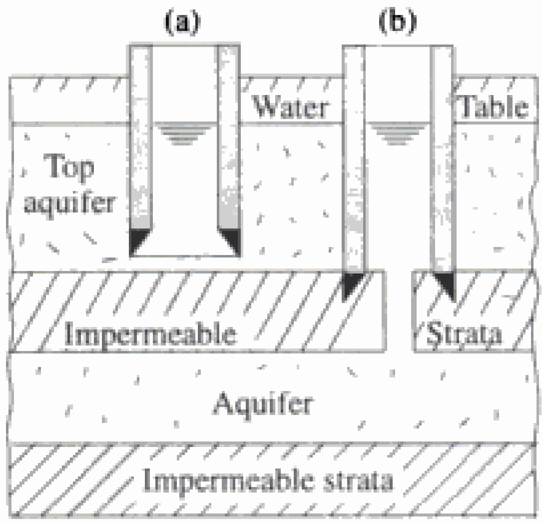


Fig. 4.16 (a) Shallow open well (b) Deep open well

B. Tube Wells A tube well consists of G.I. pipes of diameter varying from 3.75 cm to 15 cm and of length varying from 7 m to 8 m. The tube well is sunk into the ground by boring. The joints of the pipes are socket joints. It penetrates a number of water bearing strata which are sandwiched between the impervious layers.

Again, the tube well may be shallow or deep. The diameter of shallow tube well varies from 3.75 cm to 5 cm and the depth varies from 30 m to 40 m.

The discharge of this well is dependent on the water table. If the water table goes below the suction head in summer, the tube well is likely to be defunct.

The diameter of deep tube well varies from 10 cm to 15 cm and the depth from 200 m to 300 m. It draws water from the deeper most aquifers. Hence, the water is available throughout the year (Fig. 4.17).

The tube well may be of the following types:

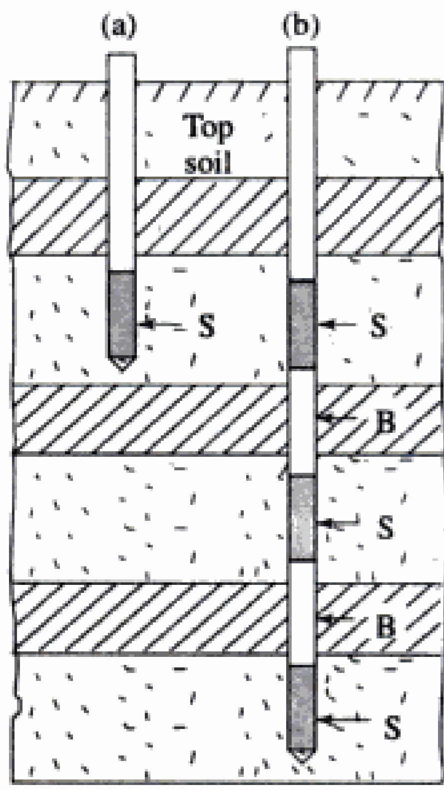


Fig. 4.17 (a) Shallow tube well (b) Deep tube well

(a) Strainer Type Tube Well

In this type of tube well, the G.I. pipes of length 6 m and of required diameter are assembled with socket joints and driven into the ground upto the required depth. A conical shoe is provided at the bottom of the pipe and the strainers are fitted in such a way so that they exist within the aquifers. The portion of the plain pipe which exists within the impermeable strata is known as blind pipe.

A strainer is a perforated pipe around which fine wire mesh is wrapped allowing an annular space between the two. The water enters are pipe through the wire mesh and the fine sand particles are prevented from entering the tube well (Fig. 4.18).

(b) Cavity Type Tube Well In this type of tube well, the G.I. pipe of required diameter is driven into the ground up to the depth so that it just penetrates clay layer and finds a water bearing strata. No strainer is provided in this tube well. So, at the beginning, when the pumping is started fine sand comes out with water and consequently a cavity is formed. As the pumping is continued, the cavity finally takes the shape of a hemisphere and no more sand particles enter the well. The cavity thus formed behaves like a clear water pool (Fig. 4.19).

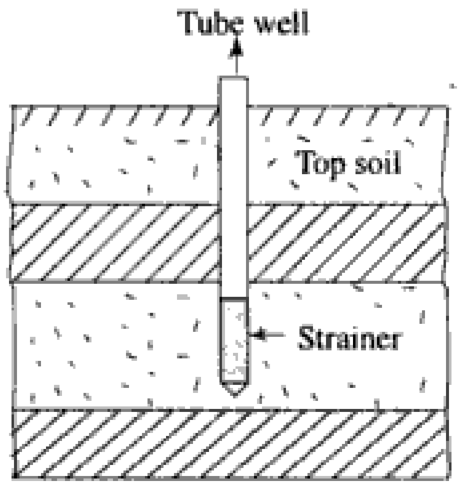


Fig. 4.18 Strainer type tube well

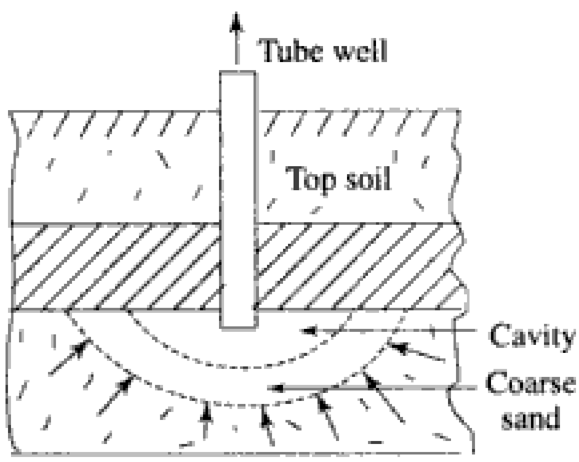


Fig. 4.19 Cavity type tube well

(c) Slotted Type Tube Well In this type of tube well, the G.I. casing pipe of bigger diameter (about 30 cm) is first driven into the ground up to the required depth. Then the G.I. well pipe of lesser diameter (about 15 cm) is inserted into the casing pipe. The lower portion of the well pipe is slotted up to a length of about 6 m. The casing pipe is then gradually withdrawn and the annular space between well pipe and casing pipe is filled up by a mixture of gravel and coarse sand. This is known as shrouding. The shrouding should be such so as to cover the slotted portion completely. When pumping is started the sandy water comes out through the well initially and finally the fine sand is eliminated and clear water comes out (Fig. 4.20).

4.6 CONSTRUCTION OF OPEN WELL

Open wells may be unlined or lined.

1. Construction of Unlined Open Well This type of open well is constructed in hard soil. In this case, the excavation of the earth is started according to the required diameter of the well. The excavation is continued until the water rushes out from the bottom. Generally, the diameter varies from 1.5 m to 2 m and the depth varies from 10 m to 12 m. As lining is not provided in this well, it is very cheap. This type of well is very common in hilly area (Fig. 4.21).

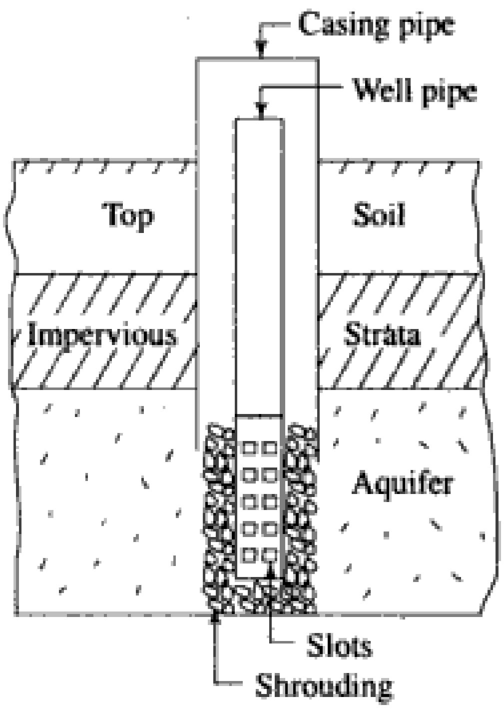


Fig. 4.20 Slotted type tube well

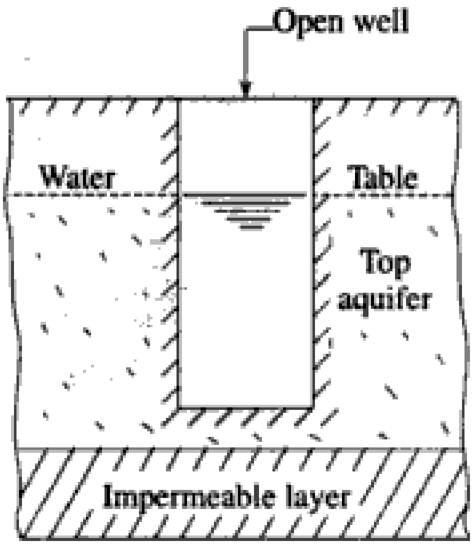


Fig. 4.21 Unlined open well

2. Construction of Lined Open Well

The lining of the well may be of two types,

(a) By pre-cast R.C.C. Rings In this case, a pit is first excavated up to a depth of 30 cm according to the diameter of the rings and one ring is placed in position perfectly. Then two more rings are set on it. Now, excavation is started and the rings go on sinking gradually due to their weight. More and more rings are set over the previous rings as the excavation proceeds. Finally, the water rushes out from the bottom. In village area, the earthen ware rings are generally used. The diameter of this type of well varies from 1 m to 1.5 m and depth varies from 15 m to 18 m. Care should be taken for the vertical sinking of well (Fig. 4.22).

(b) By Masonry Wall In this case, a pit is first excavated up to a depth of 30 cm and a well curb (made of steel, wood or R.C.C) having cutting edge at

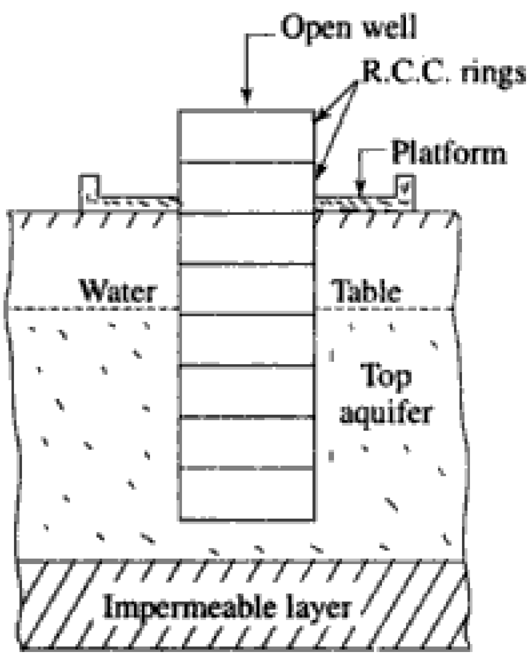


Fig. 4.22 Lining of R.C.C. rings

the bottom is placed in position. The top width of the circular curb is equal to the thickness of the well wall. On the top of the well curb the masonry wall is constructed up to a height of 1.5 m. After proper curing of the wall, the excavation is started inside the well and below the curb. The masonry wall goes on sinking due to its weight. The masonry wall is constructed step-by-step (1.5 m height at a time) as the excavation proceeds. Thus the well is sunk up to the required depth (Fig. 4.23). The verticality of the well should be maintained with Plumb bob.

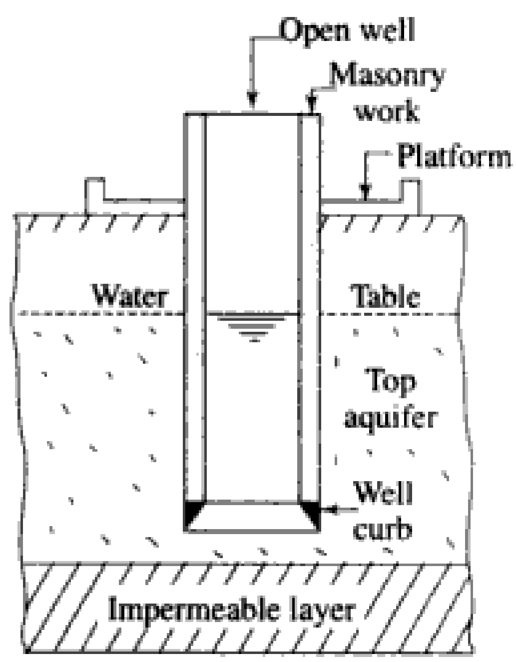


Fig. 4.23 Lining of masonry work

4.7 DETERMINATION OF YIELD OF OPEN WELL

The yield of open well can be determined by the following tests.

(a) Constant Level Test or Pumping Test In this test, the water from the open well is pumped for some period so that the water level is depressed by a head H which is known as depression head or draw down. Then the rate of pumping is adjusted in such a way so that the water level remains constant in the well. At this time, the rate of pumping is equal to the rate of yield from the well. Now, from the rate of pumping the discharge can be computed by suitable measuring device (Fig. 4.24). from Darcy's Law,

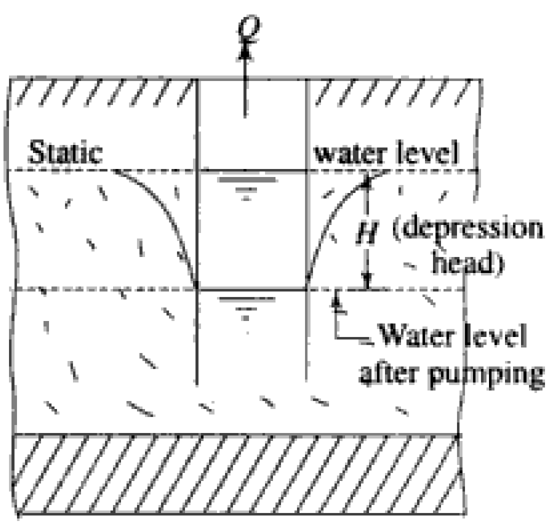


Fig. 4.24 Pumping test

$$Q = K A i$$

$$Q = K A \times \frac{H}{L} \quad \text{where, } i = \frac{H}{L}$$

$$Q = C A H \qquad c = \frac{K}{L}$$

where, Q = Discharge; K = Coeff. of permeability; A = c/s area of permeable layer through which water enters the well; i = hydraulic gradient.
H = Depression head; L = Length of flow path; C = percolation intensity coeff.

(b) Recuperation Test As it is very difficult to adjust the rate of pumping to maintain the constant level in the well, the recuperation test is adopted to determine the yield of open well.

In this test, the water from the well is pumped to a depression hed H_1 and the pumping is stopped. The water level rises due to ground water flow. The total time taken by the water to reach the static level is noted. Now the rate of yield may be calculated from the expression deduced below (Fig. 4.25).

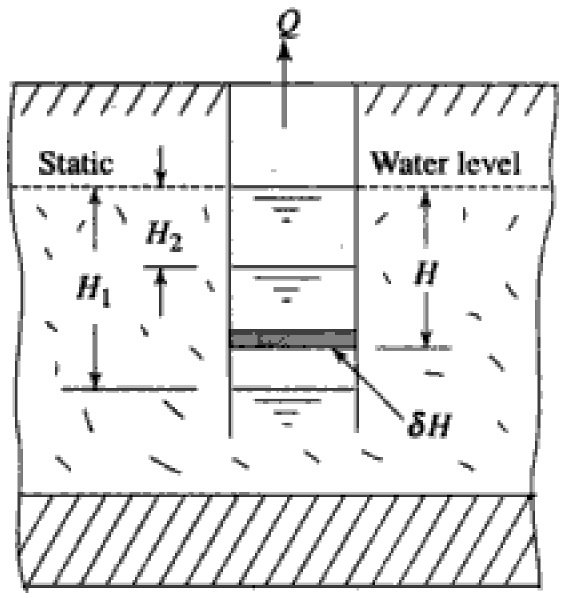


Fig. 4.25 Recuperation test

Let, H_1 = Depression head when pumping was stopped, H_2 = Depression head after certain period, T = Time taken by the water level to rise from H_1 to H_2, H = Depression head at time 't', δH = Decrease in depression head in time δt.

In time t the water level will recuperate by $(H_1 - H)$ and it will recuperate by δH in time δt after the time t.

So, volume of water, $\delta V = A\ \delta H$ (a)

where, A = c/s area of the bottom of well.

Again,

$$\delta V = Q \cdot \delta t$$

and $$Q \propto H$$

or $$Q = K \cdot H$$

where, Q = Rate of recharge at time t under head H.

$$\therefore \quad \delta V = K\, H\, \delta t \qquad \text{(b)}$$

$$K = A \text{ constant}$$

from (a) and (b)

$$K\, H\, \delta t = A\ \delta H$$

$$\delta t = -\frac{A}{K} \cdot \frac{\delta H}{H} \qquad \text{(c)}$$

Negative sign, because when the time t increases the depression head H decreases].

Integrating both sides of the equation (c) within limits,

$$t = O \text{ and } T$$

$$H = H_1 \text{ and } H_2$$

$$\int_0^T \delta t = -\frac{A}{K} \int_{H_1}^{H_2} \frac{\delta H}{H}$$

or $$T = -\frac{A}{K} \log_e \left[\frac{H_2}{H_1}\right]$$

or $$T = \frac{A}{K} \log_e \left[\frac{H_1}{H_2}\right]$$

or $$TK = A \log_e \left[\frac{H_1}{H_2}\right]$$

or $$\frac{K}{A} = \frac{1}{T} \log_e \left[\frac{H_1}{H_2}\right]$$

$$\frac{K}{A} = \frac{2.303}{T} \log_{10} \left[\frac{H_1}{H_2}\right] \quad 5.5(a)$$

Here, $\frac{K}{A}$ is known as specific yield of well per unit area. It is expressed as $m^3/hr/m^2$.

Again, $$K = \frac{2.303 \times A}{T} \log_{10} \left[\frac{H_1}{H_2}\right] \quad 5.5(b)$$

where, K = Specific capacity of well in m^3/hr/unit head.
The discharge of the well, $Q = K \cdot H$. where, H = constant depression head.

4.8 PROBLEMS ON YIELD OF OPEN WELL

Problem 1 Calculate the specific capacity of an open well from the following data,

Initial depression head = 5 m
Final depression head = 2 m
Time of recuperation = 2 hrs
Diameter of well = 3 m

Calculate also the specific yield and yield of the well under head of 3 m.

Solution

Here, $$A = \frac{\pi}{4} \times 3^2 = 7.07 \text{ m}^2$$
$$T = 2 \text{ hrs}$$
$$H_1 = 5 \text{ m}$$
$$H_2 = 2 \text{ m}$$
$$H = 3 \text{ m}$$

From the relation. (Eq. 5.5b)

$$K = \frac{2.303 \times A}{T} \cdot \log_{10} \left[\frac{H_1}{H_2}\right]$$

$$K = \frac{2.303 \times 7.07}{2} \log_{10} \left[\frac{5}{2}\right] = 3.24 \text{ m}^3/\text{hr/unit head}$$

So, the specific capacity is 3.24 m^3/hr/unit head

Specific yield = $\frac{K}{A} = \frac{3.24}{7.07} = 0.458 \text{ m}^3/\text{hr/m}^2$

Yield of well = $Q = K \times H = 3.24 \times 3 = 9.72$ m^3/hr.

Problem 2 Find the diameter of an open well to give the discharge of 3 lits/sec. The depression head is 3 m and specific yield is 1 m^3/hr/m^2.

Solution

We know, $$Q = K\,H \quad \text{or} \quad Q = \frac{K}{A} \cdot A \cdot H \qquad (1)$$

here, $$\frac{K}{A} = 1\text{m}^3/\text{hr/m}^2$$

$$A = \frac{\pi}{4} d^2 \text{ (assuming the diameter of well as } d \text{ m)}$$

$$H = 3 \text{ m}$$

$$Q = 3 \text{ lits/sec} = \frac{3 \times 60 \times 60}{1000} = 10.8 \text{ m}^3/\text{hr}$$

From equation (1)

$$10.8 = 1 \times \frac{\pi}{4} d^2 \times 3; \qquad 10.8 = 2.356\, d^2$$

$$d^2 = \frac{10.8}{2.356} \quad \therefore \quad d = 2.14 \text{ m}$$

So, the diameter of open well is 2.14 m.

4.9 SINKING OF TUBE WELLS

1. Shallow Tube Wells The shallow tube wells are generally sunk by two methods.

(a) Wash Boring Method In this method, a pit of depth about 1 m and diameter 10 cm is first excavated at the position where the tube well is proposed to be sunk. A G.I. pipe of diameter 3.75 cm and length 6 m is held vertically in the pit by the lever. The pipe is completely filled with water and an operator standing on the bamboo frame covers the upper end of the pipe with palm of his hand. Sufficient quantity of water is poured continuously on the pit. Now, the pipe is moved up and down with the lever which is operated by a group of labourers. During upward stroke, the pipe's end palm is kept tightly closed with palm and during downward stroke it is kept open. Thus, in upward motion the suction takes place and in downward motion the slurry rushes out through the upper end of the pipe. In this way, the pipe sinks gradually. When the first pipe reaches the ground level, another pipe is fixed with it by socket joint. The procedure is continued until the required depth is reached. The washing of the pipe is done until clear water comes out. This method is suitable for sandy and clayey soil (Fig. 4.26).

(b) Water Jet Method In this method, a pit is first excavated to a depth of about 1 m and diameter slightly larger than the diameter of the casing pipe. The casing pipe is held vertically in the pit. Then a jet pipe with a nozzle at the bottom is lowered inside the casing pipe. Water is forced under high pressure through the jet pipe and it comes out of the nozzle with tremendous force, the soil at the bottom of casing pipe is loosened and slurry is formed. This slurry comes out through the annular space between the jet pipe and casing pipe. The casing pipe is rotated slowly and it sinks gradually. When the casing pipe comes to the ground level, another pipe is fitted to it by socket joint. This procedure is continued until the required depth is attained. This method is suitable for sandy and clayey soil (Fig. 4.27). Finally, tube well pipe with strainers is inserted and casing pipe is withdrawn.

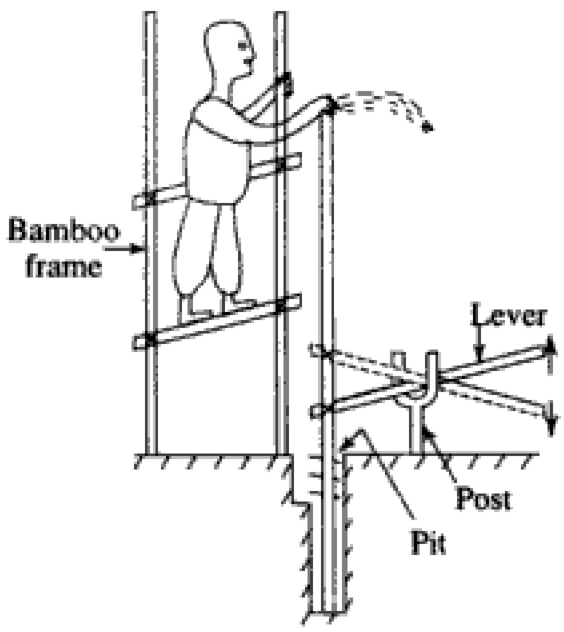


Fig. 4.26 Wash boring method

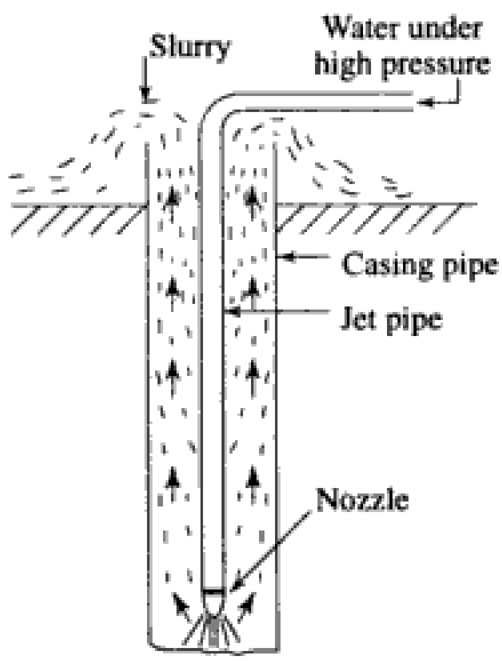


Fig. 4.27 Water jet method

2. Deep Tube Wells The deep tube wells are generally sunk by Percussion method and Rotary method.

(a) Percussion Method In this method a pit is first excavated to a depth of about 1 m and it is filled up with water. A casing pipe of required diameter having cutting edge at the bottom is placed vertically in the pit. The casing pipe consists of a platform for loading sand bags. A plunger of hollow metal tube is suspended by a rope which passes through a pulley fixed with the wooden tripod. The plunger consists of cutting edge and a ball valve. Now, repeated blows are given with the plunger by stretching the rope up and down. During each down stroke, the ball valve opens and the slurry enters the plunger. Again, during upstroke, the ball valve closes and the slurry is prevented from coming out. When the plunger is filled up with slurry, it is taken out and the slurry is removed. The plunger is again lowered and the same procedure is repeated. Thus the casing pipe goes on sinking. When the upper end of the casing pipe reaches the ground level, an additional pipe is fixed to it by socket joint. When the desired depth is attained the platform is removed and a tube well pipe with strainers (at predetermined aquifers) is inserted within the casing pipe. The casing pipe is then gradually withdraw step-by-step and the annular space between casing pipe and tube well pipe is shrouded with gravel and coarse sand (Fig. 4.28).

(b) Rotary Method This method consists of the following components

(a) A derrick
(b) A pump set (with motor)
(c) Drill pipe with drill bit
(d) Rotating device
(e) A platform carrying the derric and pump set.

The whole arrangement is known as drill rig.

The rig is anchored at the site properly. A suitable tank is excavated near the site, and it is filled with muddy water. The rotating device and drill pipe is placed in position. When the rotating device is started, the drill pipe rotates and slides downwards cutting the soil with drill bit. The water is continuously pumped into the hole through the drill pipe and thus slurry is formed. This slurry comes out through the annular space between the drill pipe and wall of the hole. This slurry is taken again to the mud water tank for recharging. A portion of the slurry is taken to setting basin to determine the type of formation at various layers. When the required depth is attained, the drill pipe is withdrawn. The tube well pipe with strainers at appropriate layers is inserted immediately into the hole. Then washing of the well is done properly to get clear water and more yield (Fig. 4.29).

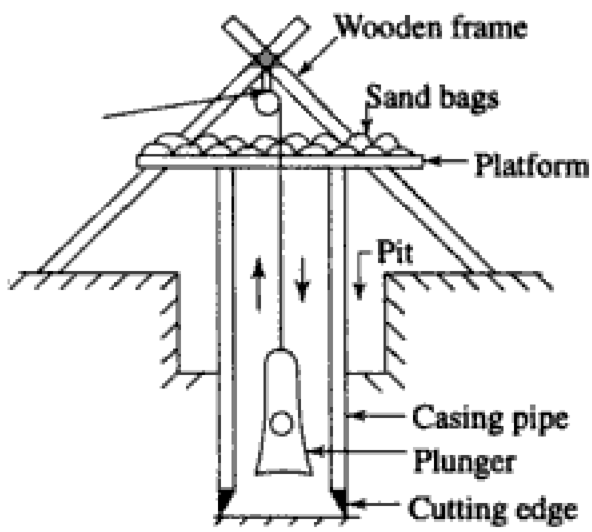


Fig. 4.28 Percussion method

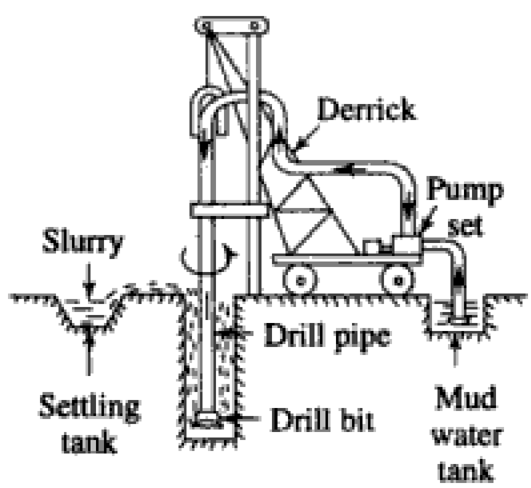


Fig. 4.29 Rotary method

4.10 YIELD OF TUBE WELL IN UNCONFINED AQUIFER

Let, H = Height of static water level from impervious layer, Q = Discharge of the well; S = Draw down; h = depth of water in well from the point of max drawdown to the top of impervious layer; R = Radius of circle of influence; r = Radius of well; O = A point considered as origin; T = A point on drawdown curve with co-ordinate (x, y).

Considering, that a cylindrical surface around the well passes vertically through the point $T(x, y)$ Fig. 4.30. Then, the area of cylindrical surface = $2\pi x, y$

If $\frac{dy}{dx}$ be the hydraulic gradient at point T. Then, by Darcy's law, Rate of flow = $K \frac{dy}{dx} \cdot (2\pi x y)$, K = coeff. of permeability

Again, Rate of flow = Discharge of well

$$Q = \frac{dy}{dx}(2\pi x y)$$

or $$Q \cdot \frac{dx}{x} = 2\pi ky \cdot dy \quad (1)$$

Integrating both sides of Eq. (1) between limits, $x = r$ and R, $Y = h$ and H

$$Q\int_r^R \frac{dx}{x} = 2\pi k\int_h^H y\,dy$$

or $$Q \log_e\left[\frac{R}{r}\right] = \pi K \quad (H^2 - h^2)$$

or $$Q = \frac{\pi K\left(H^2 - h^2\right)}{\log_e\left(\frac{R}{r}\right)}$$

or $$Q = \frac{\pi K\left(H^2 - h^2\right)}{2.303\log_{10}\left(\frac{R}{r}\right)}$$

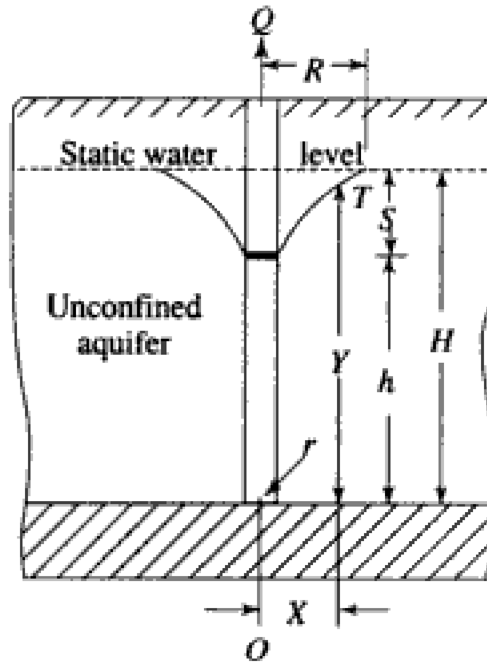


Fig. 4.30 Unconfined aquifer

or $$Q = \frac{1.36\,K\left(H^2 - h^2\right)}{\log_{10}\left(\frac{R}{r}\right)} \quad 5.8\text{ (a)}$$

This equation can be expressed in terms of drawdown S.

$$S = H - h$$

or $$H = S + h$$

and $$H + h = (s + 2h)$$

$$(H^2 - h^2) = (H + h)(H - h)$$

$$= (S + 2h) \cdot S$$

So, the expression in 5.8(a) may be written as,

$$Q = \frac{1.36\,Ks\,(S - 2h)}{\log_{10}\left(\frac{R}{r}\right)} \quad 5.8\text{ (b)}$$

4.11 YIELD OF TUBE WELL IN CONFINED AQUIFER

Let, H = Height of static water level from the bottom of confined aquifer; Q = Discharge of the well; S = Draw down; h = Height of the point of max. drawdown from the bottom of confined aquifer; R = Radius of the circle of influence; O = A point consider as origin; T = A point considered on drawdown curve with co-ordinate (x, y)

Considering that a cylindrical surface around the well passes vertically through the point $T(x, y)$ (Fig. 4.31). The area of cylindrical surface $= 2\pi xb$ (lying within confined aquifer)

If $\dfrac{dy}{dx}$ be the hydraulic gradient at T.

By Darcy's law,

Rate of flow through the cylindrical surface $= K \cdot \dfrac{dy}{dx} \cdot 2\pi xb$

But, Rate of flow = Discharge of well

Discharge of well, $Q = K \cdot \dfrac{dy}{dx}(2\pi xb)$

[K = coeff. of permeability].

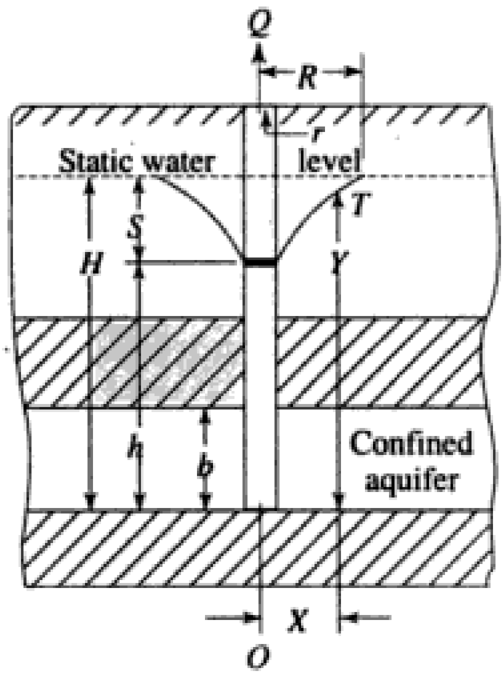


Fig. 4.31 Confined aquifer

or $$Q \cdot \frac{dx}{x} = 2\pi K b\, dy \quad (1)$$

Integrating both sides of the equation (1) between the limits,

$$x = r \text{ and } R,\ Y = h \text{ and } H$$

$$Q\int_r^R \frac{dx}{x} = 2\pi K b \int_h^H dy$$

or $$Q \log_e\left[\frac{R}{r}\right] = 2\pi K b\,(H - h)$$

or $$Q = \frac{2\pi Kb\,(H-h)}{\log_e\left(\frac{R}{r}\right)}$$

or $$Q = \frac{2\pi Kb\,(H-h)}{2.303\log_{10}\left(\frac{R}{r}\right)}$$

or $$Q = \frac{2.72\,Kb\,(H-h)}{\log_{10}\left(\frac{R}{r}\right)} \quad 5.9(a)$$

But, $$S = (H - h)$$

$\therefore$ $$Q = \frac{2.72\,K\,bs}{\log_{10}\left(\frac{R}{r}\right)} \quad 5.9(b)$$

For a confined aquifer, the coefficient of transmissibility is given by $T = Kb$. So, the equation 5.9(b) can be written as,

$$Q = \frac{2.72\,Ts}{\log_{10}\left(\frac{R}{r}\right)} \quad 5.9\,(c)$$

4.12 PROBLEMS ON YIELD OF TUBE WELL

Problem 1 A tube well fully penetrates on unconfined aquifer. Calculate the discharge of the tube well from the following data.

(a) Diameter of the well = 15 cm
(b) Draw down = 4 m
(c) Length of tube well strainer below drawdown = 10 m
(d) Coefficient of permeability of aquifer = 0.05 cm/sec
(c) Radius of circle of influence = 200 m

Solution Here,

$$K = 0.05 \text{ cm/sec} = 5 \times 10^{-4} \text{ m/sec}$$
$$S = 4 \text{ m}$$
$$h = 10 \text{ m (Effective length)}$$
$$r = 7.5 \text{ cm} = 0.075 \text{ m}$$
$$R = 200 \text{ m}$$

From the Eq. 5.8(b),

$$Q = \frac{1.36\, KS\,(S + 2h)}{\log_{10}\left(\frac{R}{r}\right)}$$

We get,

$$Q = \frac{1.36 \times 5 \times 10^{-4} \times 4\,(4 + 2 \times 10)}{\log_{10}\left(\frac{200}{0.075}\right)}$$

or

$$Q = \frac{0.06528}{3.4259} = 0.019 \text{ m}^3\text{/sec} = 19 \text{ lits/sec}$$

Problem 2 A tube well of 30 cm diameter penetrates an unconfined aquifer. During the pumping test following data was obtained

(a) Height of static water level from bottom of aquifer = 50 m
(b) Height of drawdown from bottom of aquifer = 45 m
(c) Radius of circle of influence = 300 m
(d) Coefficient of permeability = 50 m/day

Calculate the discharge of the well.

Solution Here

$$R = 300 \text{ m}$$
$$r = \frac{30}{2} = 15 \text{ cm} = 0.15 \text{ m}$$
$$H = 50 \text{ m}$$
$$h = 45 \text{ m}$$
$$K = 50 \text{ m/day} = \frac{50}{24 \times 60 \times 60} = 5.787 \times 10^{-4} \text{ m/sec}$$

From the Eq. 5.8(a)

$$Q = \frac{1.36\ K\left(H^2 - h^2\right)}{\log_{10} \frac{R}{r}}$$

We get,

$$Q = \frac{1.36 \times 5.787 \times 10^{-4}\ \left(50^2 - 45^2\right)}{\log_{10} \frac{300}{0.15}} = \frac{0.3738}{0.3010}$$

$$= 0.1132\ \text{m}^3/\text{sec} = 11.32\ \text{lits/sec}$$

Problem 3 A tube well penetrates fully into a confined aquifer. The following data was collected during observation. Calculate the discharge of the well

(a) Radius of tube well = 20 cm
(b) Thickness of confined aquifer = 25 m
(c) Drawdown = 4 m
(d) Radius of circle of influence = 300 m
(e) Coefficient of transmissibility = 125×10^{-4} m²/sec

Find also the coeff. of permeability.

Solution Here,

$R = 300$ m
$r = 20$ cm $= 0; 20$ m
$S = 4$ m
$b = 25$ m
$T = 125 \times 10^{-4}$ m²/sec

We know,
Coefficient of transmissibility, $T = K\,b$.

$$K = \frac{T}{b} = \frac{125 \times 10^{-4}}{25} = 5 \times 10^{-4}\ \text{m/sec}$$

So, coeff. of permeability is 5×10^{-4} m/sec.
Now, from equation, 5.9 (b)

$$Q = \frac{2.72\ K\ b\ s}{\log_{10} \frac{R}{r}}$$

$$= \frac{2.72 \times 5 \times 10^{-4} \times 25 \times 4}{\log_{10}\left(\frac{300}{20}\right)}$$

$$= \frac{1360 \times 10^{-4}}{3.1761} = 428.198 \times 10^{-4}\ \text{m}^3/\text{sec}$$

$$= 0.0428\ \text{m}^3/\text{sec} = 42.8\ \text{lits/sec}$$

Problem 4 A tube well penetrates completely a confined aquifer. Determined the diameter of the tube well from the following data.

(a) Required yield = 100 lits/sec

(b) Radius of circle of influence = 200 m
(c) Thickness of confined aquifer = 30 m
(d) Drawdown = 5 m
(e) Coefficient of permeability = 60 m/day

Solution Here,

$$Q = 100 \text{ lits/sec} = 0.10 \text{ cumec}$$
$$R = 200 \text{ m}$$
$$b = 30 \text{ m}$$
$$S = 5 \text{ m}$$
$$K = 60 \text{ m/day} = \frac{60}{24 \times 60 \times 60}$$
$$= 6.944 \times 10^{-4} \text{ m/sec}$$

Let, r be the radius of the tube well.
From, the Eq. 5.9(b)

$$Q = \frac{2.72\,K\,b\,s}{\log_{10}\left(\frac{R}{r}\right)}$$

We get,
$$0.10 = \frac{2.72 \times 6.944 \times 10^{-4} \times 30 \times 5}{\log_{10}\left(\frac{200}{r}\right)}$$

or
$$\log_{10}\left(\frac{200}{r}\right) = \frac{2.72 \times 6.944 \times 10^{-4} \times 30 \times 5}{0.10}$$
$$= \frac{2833.15 \times 10^{-4}}{0.10} = 2.8331$$

or
$$\frac{200}{r} = 680.92$$
$$r = \frac{200}{680.92} = 0.29 \text{ m} = 29 \text{ cm} = 30 \text{ cm (say)}$$

So, the diameter of the well is 60 cm.

4.13 TYPES OF STRAINERS

The strainers are perforated pipes around which the fine wire mesh is wrapped to arrest the sand particles. The clear water is allowed to enter the tube well. The strainers are provided in the tube well pipe corresponding to unconfined or confined aquifers. Generally, the strainers are of the following types.

(a) Cook Strainer This is a hollow brass tube manufactured by rolling. The slots are cut on the pipe in the form of a truncated hollow pyramid. The slots are narrow outside and wide towards inside. Fine wire mesh is wrapped around the strainer pipe to cover the slots. The size of the slots varies from 0.15 to 0.40 mm (Fig. 4.32).

(b) Tej Strainer This is a brass tube, but it is manufactured from brass sheet by bending it to the shape of a circular pipe. It consists of a joint which is welded. The tube is wrapped with fine wire mesh. These strainers are commonly used in India (Fig. 4.33).

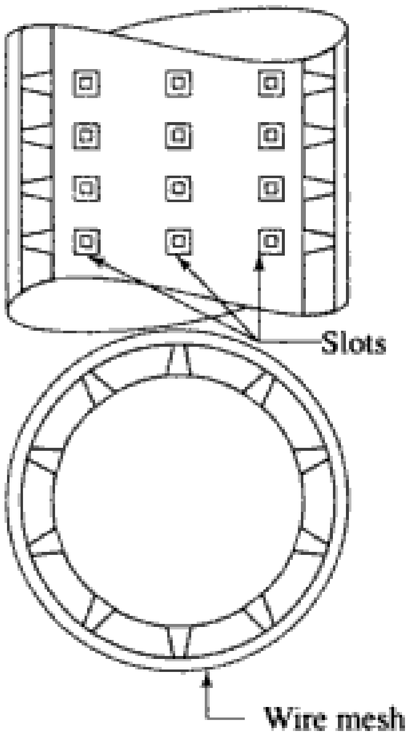


Fig. 4.32 Cook strainer

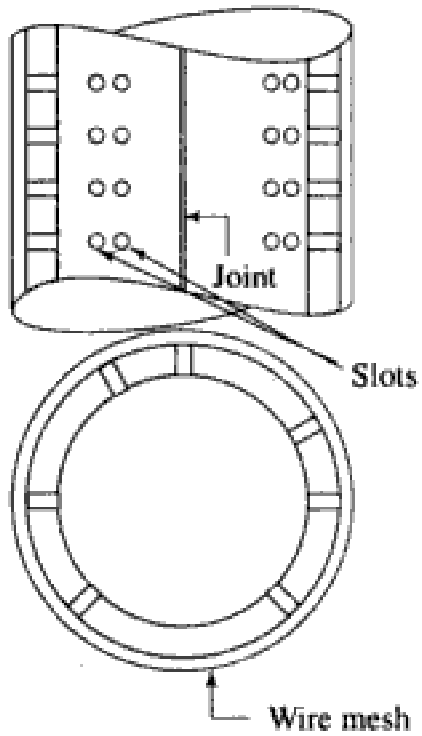


Fig. 4.33 Tej strainer

(c) Brownlie Strainer This strainer is manufactured from steel pipe having star shaped projections. The perforations are made on the circular body. The strainer pipe is first encased within a wire net. Then fine wire mesh is wrapped around the net. The net and the mesh are generally made of copper (Fig. 4.34).

(d) Ashford Strainer This is made of steel tube which is perforated. A thick copper wire is wound round the pipe on which fine wire mesh is wrapped. Thus a gap is maintained between the tube and the mesh (Fig. 4.35).

4.14 SHROUDING OF WELL

The method of filling the annular space between the casing pipe and the tube well pipe by an aggregate of gravel and coarse sand is termed as shrounding. It is required for slotted type tube well in sandy and unconsolidated soil. The shrounding is done for the following reasons,

(a) It prevents the finer particles of the soil coming in contact with the strainer.
(b) It prevents chocking of the strainer.
(c) It protects the well pipe from the corrosive effect of surrounding soil.
(d) It increases the effective diameter of well.

The shrounding materials should be well graded. Shrouding is essential when actual tube well pipe is to be sunk with the help of casing pipe, where an annular

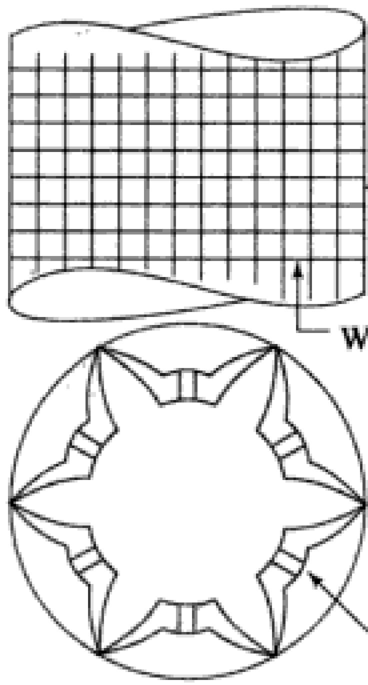


Fig. 4.34 Brownlie strainer

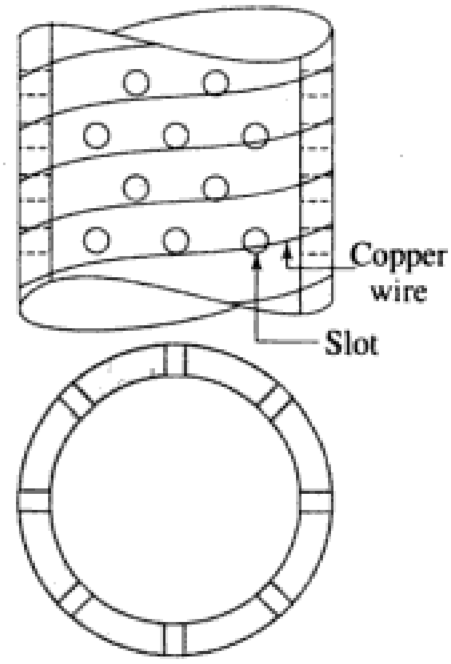


Fig. 4.35 Ashford strainer

space will appear when casing pipe is withdrawn. Here the casing pipe is sunk up to the required depth, then the tube well pipe fitted with strainers (slotted type) is inserted fully. The casing pipe is then withdrawn to a height of about 60 cm and the hollow space between the tube well pipe and well hole is immediately filled up with shrounding materials. The casing pipe is again lifted to a height of 30 cm and the space is filled up by the mixture of sand and gravel. In this way the casing pipe is withdrawn stage by stage and the spaces are filled up with the sand and gravel until the slotted strainer is completely covered (Fig. 4.36).

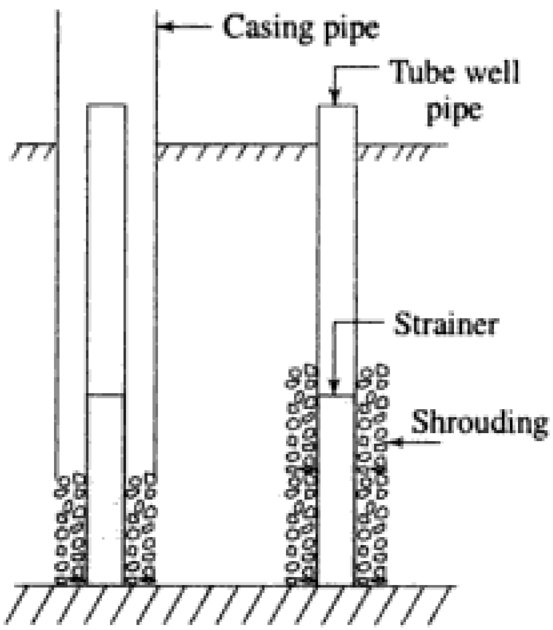


Fig. 4.36 Shrouding of well

4.15 DEVELOPMENT OF WELL

The method of extracting and removing the fine sand particles from the soil around the strainer is termed as development. The development is done for the following reasons,

(a) To prevent the sand particles from entering the well.
(b) To increase the specific capacity of the well.
(c) To increase the life of well.

The development is generally done by the following methods.

(i) By Pumping In this method, the pumping of the well is done repeatedly. At the beginning, the pumping operation is started and is continued till clear water comes out. Then the pumping is stopped. After a specified interval of time, the pumping is again started and continued until clear water comes out. In this way be successive operation if it is found that the sand particles are completely removed and the maximum discharge capacity has been attained, then the well is said to be completely developed.

(ii) By Compressed Air An air pipe is introduced into the well pipe and extends up to the zone of strainer. Then the compressed air having pressure (7 kg/cm^2) is sent through the air pipe by opening the air valve. The pressure of the air in the well develop a powerful agitation and the water flows outwards through the strainer and loosens the fine materials. The air valve is then closed. The inside pressure is decreased and consequently the water from outside enters the well with tremendous force bringing the loosened materials with it. The water of the well is then pumped out. Again, the air valve is opened and the same process is followed. This process is repeated several times till clear water comes out.

(iii) By Back Washing This method consists of an air compressor a discharge pipe and an air pipe. The discharge pipe and air pipe assembly is introduced into the well pipe.

The compressed air is then sent through the air pipe which forces the water in the well carrying the fine sand particles to come out through the discharge pipe. When the clear water comes out of the well, the air compressor is shut off. The water in the well is allowed to return to the static level. The process is repeated for several times until the clear water is discharged.

(iv) By Chemicals This method involves the use of some chemical compounds like hydrochloric acid and solid carbon dioxide (dry ice). At first hydrochloric acid is poured into the well and well top is capped. The compressed air is sent into the well through air pipe which forces the hydrochloric acid to enter the surrounding soil formation. The well cap is then removed and dry ice is introduced into the well. The gaseous carbon dioxide is released suddenly and high pressure is created in the well. This high pressure forces the muddy water to come out of the well in the form of jet. Thus the well is prepared to give clear water. The process is repeated for several times.

4.16 MAINTENANCE OF TUBE WELL

The maintenance work involves the method of cleaning or replacing the strainers or tube well pipes to make it workable for longer period. The following measures are generally taken,

(a) If the yield of the well is reduced due to the clogging of strainer, then the strainer may be cleaned by surging with the help of a plunger. The plunger is introduced into the well pipe below the water level. The water in the well is agitated by downward and upward stroke of the plunger. Thus, the sand particles sticking to the strainer are removed. The fine sand particles which come inside the well are pumped out.

(b) Sometimes compressed air is forced through the well pipe to remove the clogging.

(c) Dry ice may be dropped into the well pipe and the well top is tightly capped. The pressure of carbon dioxide vapour forces the soil particles away from the strainer.

(d) The internal corrosion of the well pipe or strainer may be removed by sulphuric acid.

(e) If muddy water is discharged through the well then it is an indication of perforation in the well pipe or damaged strainer. In that case, the resinking of the well should be done by removing the affected pipe and changing the wire net of the strainer.

4.17 FAILURE OF TUBE WELL

A tube well may fail due to following reasons.

(a) Corrosion The ground water contains acids, chlorides and sulphates, etc. which cause the corrosion of well pipes. Again, the corrosion damages the strainers. The following steps may be taken to reduce the corrosion

(i) Always thick pipes should be used.

(ii) Galvanished pipes or other anti-corrosion coated pipe should be used.

(iii) Periodical washing of the tube well should be done by sulphuric acid.

(b) Incrustation The ground water also contains calcium bicarbonate, magnesium, sulphate, etc. These compounds get deposited inside the tube well and the diameter of the well is reduced. This is known as incrustation. The incrustation can be arrested by the following steps,

(i) The water of the tube well is tested in the laboratory to determine the presence of alkali salts. The salts which are responsible for incrustation may be removed by titration. The titration is done by forcing the proper doses of acid to the well. The water of the well is then cleaned by pumping.

(ii) The tube well should not be left idle for a long period.

4.18 PUMPS

The mechanism by which the water is lifted from the under ground source to some height or to some place is known as pump. The pumps may be of the following types

(a) Centrifugal pump.

(b) Reciprocating pump.

(c) Rotary pump.

(d) Air lift pump.

Power for Pumps

The pumps can be operated by using various devices.

(a) ***Diesel Engine*** The diesel engines for operating the pumps are very common in village areas where electricity is not available. The diesel pumpset can be installed for pumping water from shallow tube well or deep tube well. The pumpset may be of low duty, medium duty and high duty depending on the quantity of discharge and the horse power of the engine.

(b) ***Electric Motor*** When electricity is available, the pumpset may be driven by electric motor. The motor is coupled with the pumping unit. Now-a-days, electric driven pumpset is very common. It is very effective for lifting water from deep tube well by the arrangement of submerged pumpset.

Lifting Head of a Pump

The total lift of a pump consists of the following heads.

(a) ***Suction Head (H_s)*** The distance between the static water level and the centreline of pump is known as suction head.

(b) ***Delivery Head (H_d)*** The distance between the centreline of pump and the point of delivery is known as delivery head.

(c) ***Head Loss (H_f)*** It is the total loss due to friction when the water flows through the pipe.

Hence, total head = $H_s + H_d + H_f$.

4.19 CENTRIFUGAL PUMP

the centrifugal pump involves the principle of centrifugal force. When the water in the chamber (casing) of a pump is rotated vigorously by the impellers about the central point, the centrifugal force develop which forces the water towards the periphery of the chamber. Thus the velocity head is converted to pressure head and this head forces the water through the delivery pipe. At the same time, the water from the ground water source is lifted up by suction through the suction pipe. Centrifugal pump may be of the following two types,

(a) ***Volute Type*** In this type, the chamber is spiral shaped (i.e. volute shaped) and consists of impellers which are rotated by motor.

The suction takes place through the centre of the impeller ring. When the impellers rotate, the water from the centre is forced towards periphery of the chamber. The velocity of flow in the chamber remains uniform. The velocity head is converted to pressure head which causes the water to flow through the delivery pipe (Fig. 4.37).

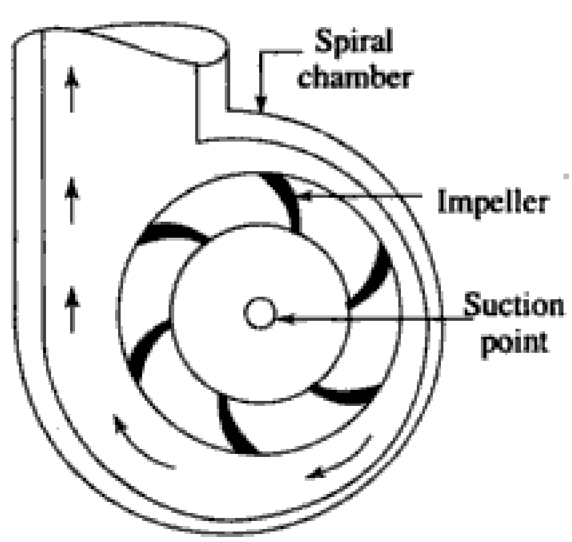


Fig. 4.37 Volute type

(b) Turbine Type In this type, a diffusion ring is provided between the impellers and the casing. The ring carries fixed diffusers or guide vanes. There are openings between the diffusers through which the water forces out towards the periphery. In this case also the velocity head is converted to pressure head which causes the water to flow through the delivery pipe (Fig. 4.38).

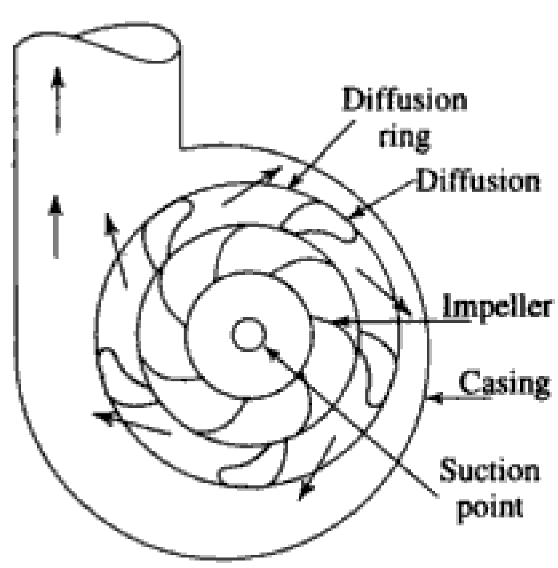


Fig. 4.38 Turbine type

Advantage of centrifugal pump

1. It requires minimum space for installation as it is compact in design.
2. It can be installed for high speed driving mechanism.
3. The working is simple and there is no valve in the pump, hence it is reliable and durable.

Disadvantages

1. The pump will not work, if the chamber is not full of water. So, the priming should always be done before starting the pump.
2. The pump will not work if there is any leakage in the suction side.

4.20 RECIPROCATING PUMP

This type of pump consists of a closed cylinder in which a piston moves to and fro by a connecting rod. The connecting rod is again hinged with a wheel which is roated by a motor. During the suction stroke, the suction valve is opened and delivery valve remains closed and water enters the cylinder. During the delivery stroke, the delivery valve is opened and the suction valve remains closed and water is forced through the delivery pipe. An inlet is provided for the priming which is necessary for starting the pump (Fig. 4.39).

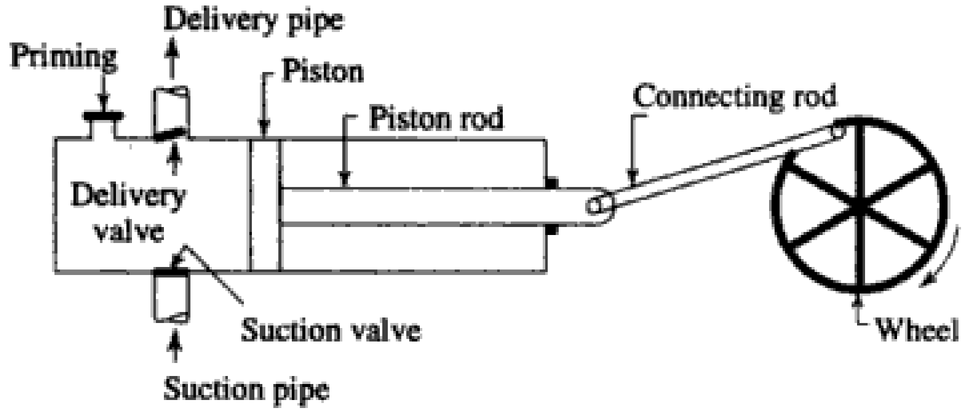


Fig. 4.39 Reciprocating pump

Advantages
1. It is suitable for large pumping units.
2. It gives constant discharge.

Disadvantages
1. It requires large space for installation.
2. It is unsuitable for pumping water containing high sediment.

4.21 ROTARY PUMP

It consists of two cams which are pivoted in a casing. These cams rotate in opposite directions and thereby the suction takes place through the suction pipe. The rotation of the cams pushes the water in upward direction through the delivery pipe (Fig. 4.40).

Advantages
1. The flow of water is uniform.
2. No priming is required.
3. It requires no valves and its operation is simple.

Disadvantages
1. It requires replacement of cams frequently and hence it involves more maintenance cost.
2. It cannot be used for pumping water containing high sediment.

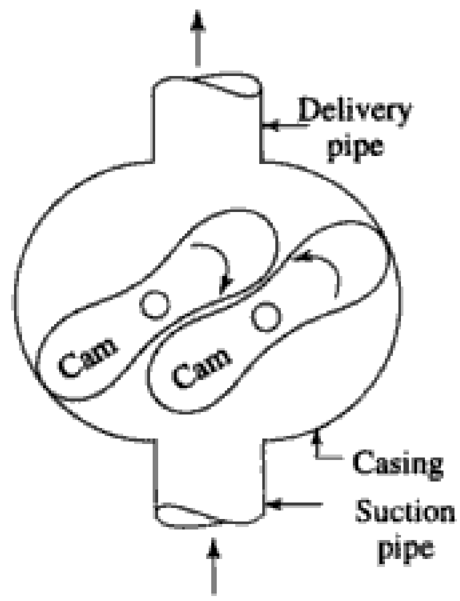


Fig. 4.40 Rotary pump

4.22 AIR LIFT PUMP

It consists of a casing pipe in which an educator pipe is introduced. An air pipe is also introduced into the casing pipe. The bottom end of the air pipe carries an air diffuser which is introduced into the educator pipe in upward direction. When compressed air is forced through the air pipe, a mixture of air and water is formed and rises up in the form of bubbles. This mixture has low specific gravity than the water in the casing pipe. Thus the pressure of the water in the educator pipe becomes less than the pressure of water in the casing pipe. This pressure difference forces the water to rise through the educator pipe and finally the water is discharged through the outlet. The efficient working of the pump depends on the air pipe's submergence depth. Generally, the depth of submergence should be about two-third of the length of air pipe (Fig.4.41).

4.23 HORSE POWER OF PUMP

The horse power of a pump is determined by the work done by the pump in raising a particular quantity of water to some height.

Let, W = Quantity of water in kg

H = Total head in m.

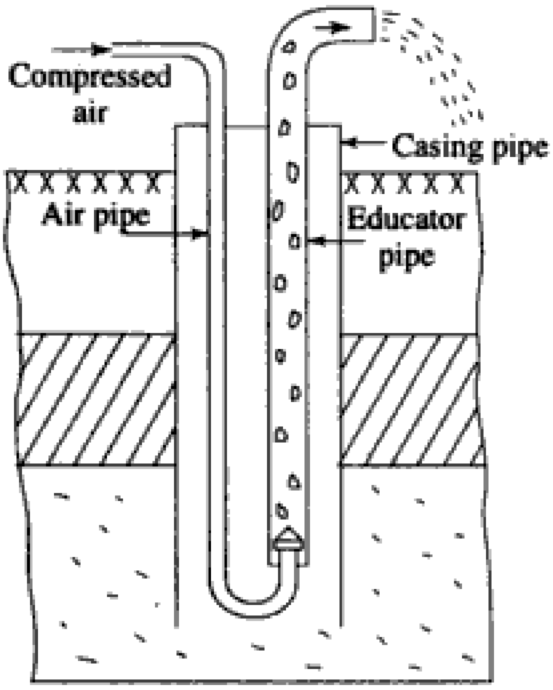


Fig. 4.41 Air lift pump

Then, work done by pump = $W \times H$

$= \omega \times Q \times H$ (i) $[W = \omega \times Q]$

where ω = Density of water 1000 kg/m^3; Q = Discharge in m^3/sec.

Again,

$$H = H_s + H_d + H_f$$

where,

H_s = Suction head,
H_d = Delivery head,
H_f = Head lost due to friction

The head lost by friction is given by, $H_f = \dfrac{f l Q^2}{3 d^5}$

where, f = Coefficient of friction; l = Total length of pipe (suction and delivery); d = Diameter of pipe in m.

Form Eq. (1)

Water horse power (W.H.P.) = $\dfrac{\omega \times Q \times H}{75}$

Considering the efficiency of the pump as η, Brake horse power

$$\text{(B.H.P.)} = \frac{(\omega \times Q \times H)}{75 \times \eta}$$

Example A centrifugal pump is required to lift water at the rate of 150 lits/sec.

Calculate the horse power of the engine from the following data.

(a) Suction head = 6 m.
(b) Coefficient of friction = 0.01
(c) Efficiency of pump = 75%

(d) Water is directly supplied to the field channel.
(d) Diameter of pipe = 15 cm.

Solution Here,

$$Q = 150 \text{ lits/sec} = 0.15 \text{ m}^3\text{/sec}$$

Suction head, $H_s = 6$ m.
Delivery head, $H_d = 0$ (As water is directly supplied to field).
Coeff. of friction $f = 0.01$.
Diameter of pipe = 15 cm = 0.15 m.

The length of the pipe where frictional effect may occur is taken equal to the suction head, so, $l = 6$ m.

$$H_f = \frac{f l Q^2}{3 d^5} = \frac{0.01 \times 6 \times (0.15)^2}{3 \times (0.15)^5} = 5.93 \text{ m}$$

So, total head, $H = H_s + H_d + H_f$

$$= 6 + 0 + 5.93 = 11.93 \text{ m}$$

Here, efficiency, $\eta = 75\% = 0.75$

$$\text{B.H.P.} = \frac{(\omega \times Q \times H)}{75 \times \eta} = \frac{1000 \times 0.15 \times 11.93}{75 \times 0.75} = 31.8 = 32 \text{ (say)}$$

4.24 METHODS OF LIFTING WATER FROM WELLS

1. Lifting of Water from Open Well The strainer is provided sufficiently below the static water level of the open well. The pump-set is installed almost at the ground level. The supply of water may not be available throughout the year as the well may be dried up in summer or the water table may goes below the suction head (Fig. 4.42).

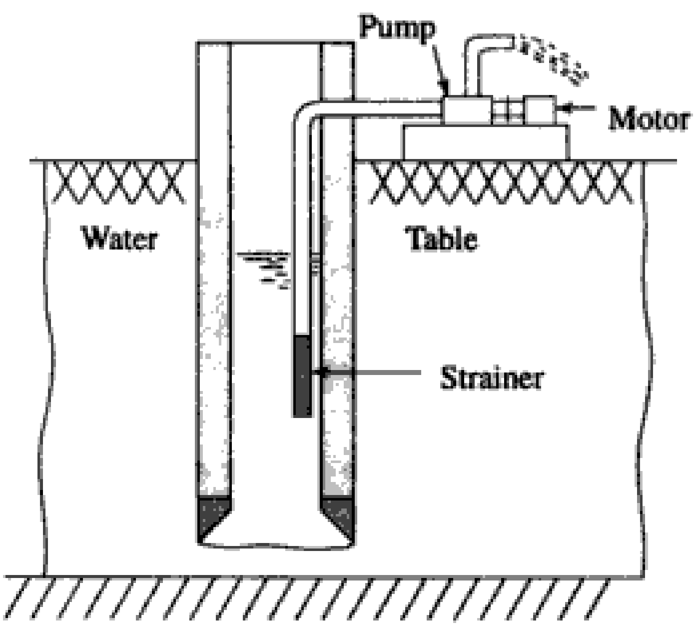


Fig. 4.42 Pumping from open well

2. Lifting of Water from Shallow Tube Well A cap is provided on the well top and a check valve is provided just below the pipe connecting the pump with the tube well. The supply of water may be stopped if the static water level goes below the suction head (i.e. 10.3 m) (Fig. 4.43).

3. Lifting of Water from Deep Tube Well by Bowl Assembly System In this case, the motor is installed at the head of the tube well. The bowl assembly of centrifugal turbine pump is introduced sufficiently below the lowest static water level (i.e. in summer). The motor is connected to the pump by driving shaft. The water is available throughout the year at constant flow rate (Fig. 4.44).

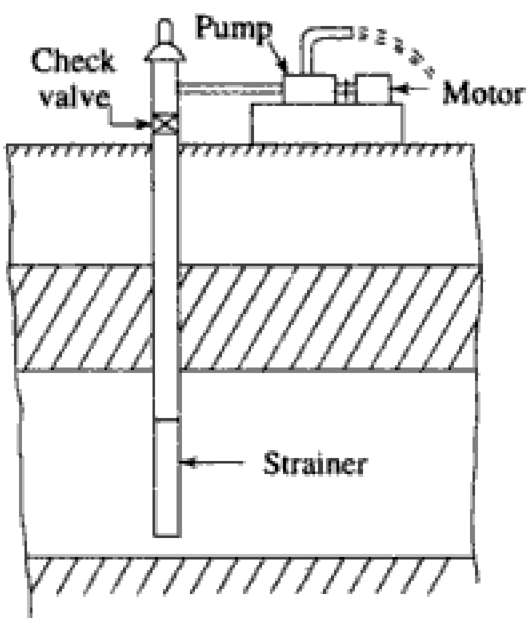


Fig. 4.43 Pumping from shallow tube well

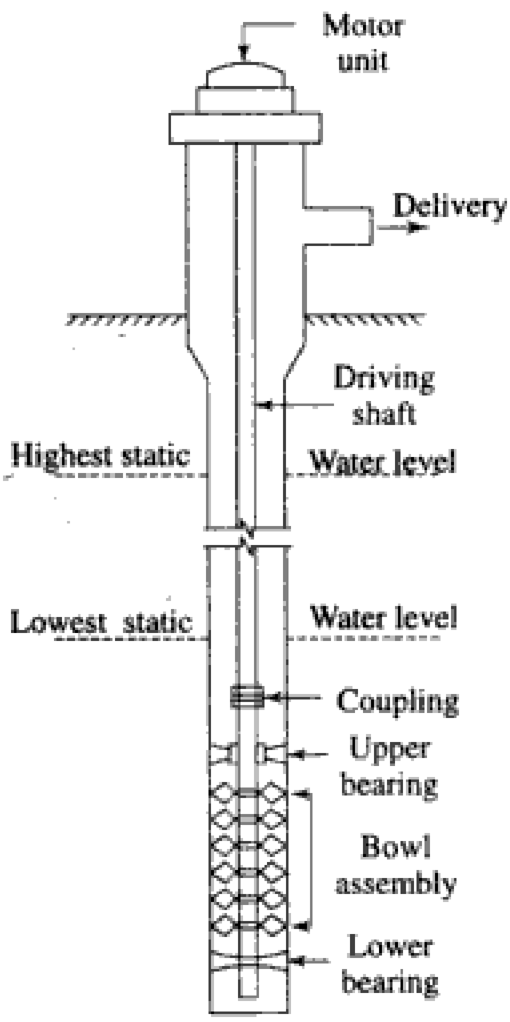


Fig. 4.44 Bowl assembly system

4. Lifting of Water From Deep Tube Well By Submersible Pump System In this system, a submersible pumpset (electric motor and centrifugal turbine pump) is lowered into the tube well by suspended cable. It is placed sufficiently below the lowest static water level. The water is available throughout the year at constant flow rate (Fig. 4.45).

4.25 SECTION OF DEEP TUBE WELL

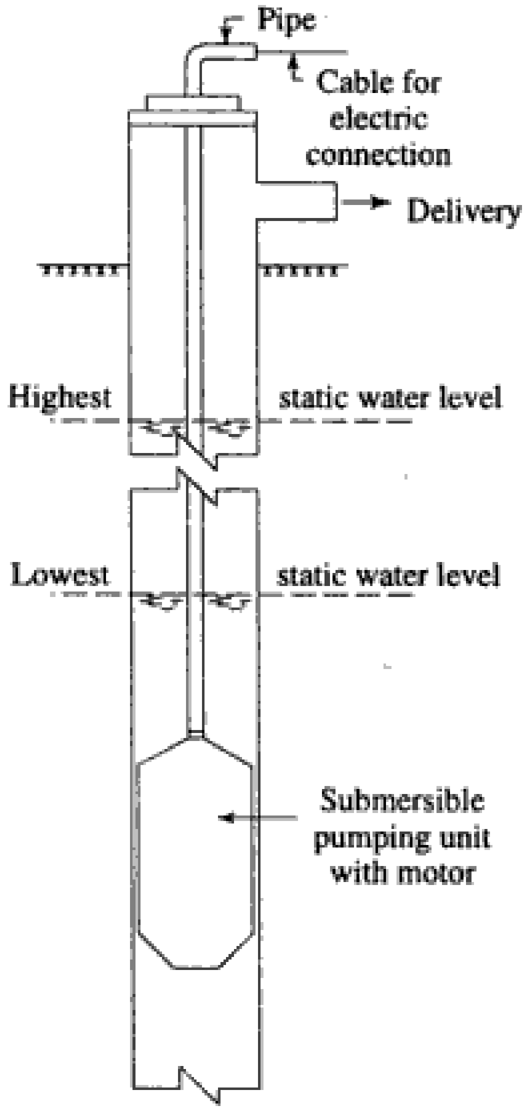


Fig. 4.45 Submersible pump system

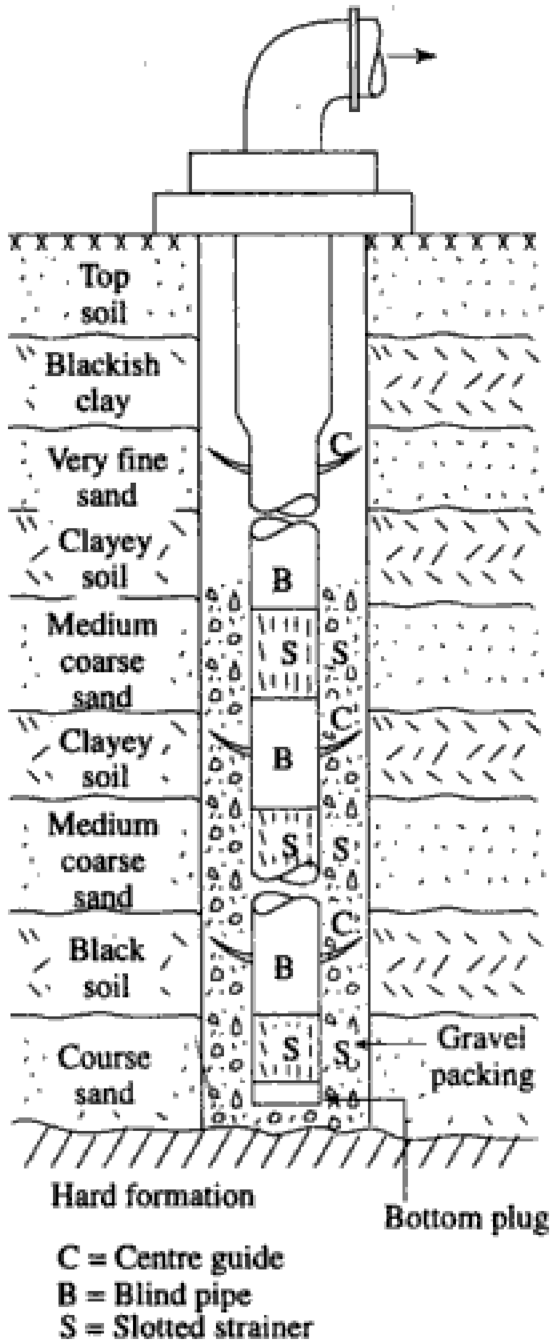


Fig. 4.46 Section of deep tube well

REVIEW QUESTIONS

1. Fill up the blanks with appropriate words
   - (i) Man or animal power is adopted for lifting water from _________ sources.
   - (ii) The permeable formation of the soil of the earth's crust is known as _________.
   - (iii) The impermeable formation which may contain water but is not capable of transmitting water is known as _________.
   - (iv) The discharge per unit draw down is known as _________.

(v) The base of the cone of depression is known as _________.

(vi) Due to interference among wells, the discharge of each well will be _________ but total discharge will be _________.

(vii) The well which draws water from _________ aquifer is known as shallow well.

(viii) The well which draws water from _________ aquifer is known as deep well.

(ix) Recuperation test is adopted to determine the _________ of open well.

(x) The deposition of Calcium Carbonate and Magnesium Carbonate inside the tube well is known as _________.

(xi) The solid Carbon dioxide is known as _________.

(xii) The shrouding is done for _________ type tube well.

(xiii) To increase the specific capacity of the well _________ of the well should be done.

(xiv) The distance between the static water level and the centreline of pump is known as _________.

(xv) The distance between the centreline of pump and the point of delivery is known as _________.

2. Distinguish between shallow well and deep well.
3. What are the different types of tube wells? Describe then with sketch.
4. Distinguish between specific yield and specific capacity.
5. What are the different methods of determining the yield of open well? Describe then with sketch.
6. What is shrouding? Why and how shrouding is done?
7. What is development of well? Why and how it is done?
8. How the failure of a well may occur? How failure can be prevented?
9. Describe a turbine pump with sketch.
10. Describe an air lift pump with sketch.

ANSWERS

1. (i) surface
(ii) aquifer
(iii) aquiclude
(iv) specific capacity
(v) circle of influence
(vi) decreased, increased
(vii) unconfined
(viii) confined
(ix) yield
(x) incrustation
(xi) dry ice
(xii) slotted
(xiii) shrouding
(xiv) suction head
(xv) delivery head

5

FLOW IRRIGATION

5.1 INTRODUCTION

The irrigation system in which the water flows under gravity from the source to the agricultural land is known as flow irrigation. The flow irrigation involves,

(a) The construction of weir or barrage across a river (known as diversion head works).

(b) The construction of dam across a river valley (to form a storage reservoir).

(c) The excavation of canal system (Network of canals to cover the command area).

This type of irrigation is popular now-a-days because a vast area can be irrigated under this system. Some important projects (such as Bhakra Nangal Project, Ukai Project, Damodar Valley Project, etc) have been implemented in India to develop agriculture and to make the country self sufficient in food. The flow irrigation may be of two types, Inundation irrigation and Perennial irrigation.

In inundation irrigation, the canals are excavated from the banks of the inundation river. The bed level of the canal is such that the water can flow in rainy season only when the water level in the river rises above the canal bed. The construction of hydraulic structures is not necessary in this system. There is no head regulator to control the flow of water through the canal. In this system water is not available throughout the year.

In perennial irrigation either a weir or a barrage is constructed across the perennial river to raise the water level or a dam is constructed to form a storage reservoir. Then the network of canals (i.e. main, branch, distributory) is constructed from the source to the agricultural lands. Here, head regulator is constructed to control the flow of water through the canal. In this system, water is available throughout the year.

5.2 TYPES OF CANALS

1. Based on Purpose Based on the purpose of service, the canals are designated as (a) Irrigation canal (b) Navigation canal (c) Power canal (d) Feeder canal.

(a) Irrigation Canal The canal which is constructed to carry water from the source to the agricultural land for the purpose of irrigation is known as irrigation canal such as Bhakra Canal, Rajasthan Canal, etc.

(b) Navigation Canal The canal which is constructed for the purpose of inland navigation is known as navigation canal. This type of canal is also utilised for irrigation such as Ganga-Bramhaputra nagivation cum irrigation canal.

(c) Power Canal The canal which is constructed to supply water with very high force to the hydroelectric power station for the purpose of moving turbine to generate electric power is known as power canal or hydel canal such as Nangal Hydel Canal.

(d) Feeder Canal The canal which is constructed to feed another canal or river for the purpose of irrigation or navigation is known as feeder canal such as Farakka barrage feeder canal.

2. Based on Nature of Supply Based on the nature of supply, the canals are designated as (a) Inundation canal (b) Perennial canal.

(a) Inundation Canal The canal which is excavated from the banks of the inundation river to carry water to the agricultural land in rainy season only when the river flows to its full capacity is known as inundation canal. No regulator is provided at the head of such canal. The flow of water through the canal depends on the fluctuation of water level in the river. When the water level rises above the bed level of the canal the water starts flowing through the canal. When the water level falls below the bed level of the canal, the flow of water through the canal stops.

(b) Perennial Canal The canal which can supply water to the agricultural land throughout the year is known as perennial canal. This type of canal is taken from the up stream side of the diversion head works (weir or barrage) or from the storage reservoir with regulator at the head of the canal.

3. Based on Discharge According to the discharge capacity, the canals are designated as (a) Main canal (b) Branch canal (c) Distributory channel (d) Field channel.

(a) Main Canal The large canal which is taken directly from the diversion head work or from storage reservoir to supply water to the network of other small canals is known as main canal. The irrigation water is not directly supplied to the field from the main canal. The water is taken to the field through the branch canal, distributory channel and field channel. So the main canal is the backbone of the canal system (Fig. 5.1).

(b) Branch Canals The branch canals are taken from either side of the main canal at suitable points so that the whole command area can be covered by the network. The discharge capacity of the branch canal is smaller than that of the main canal. The discharge varies from 5 to 10 cumec (Fig. 5.1).

(c) Distributory Channels The distributory channels are taken from the branch canals to supply water to different sectors. The discharge capacity of these channels varies from 0.25 to 3 cumec. Again, these are designated as major distributory and minor distributory according to their function in the total network (Fig. 5.1).

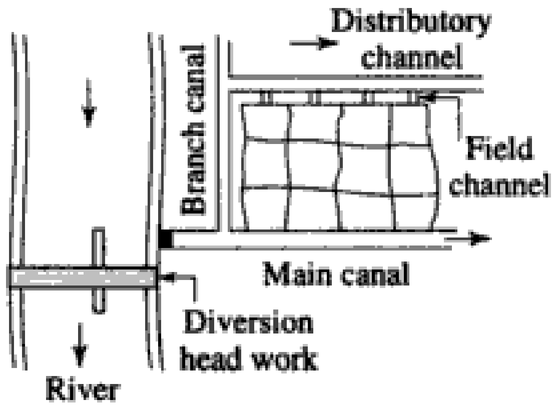


Fig. 5.1 Canal system

(d) Field Channels These channels are taken from the outlets of the distributory channels by the cultivators to supply water to their own lands. These channels are maintained by the cultivators (Fig. 5.1).

4. Based on Alignment Depending upon the alignment, the canals are designated as (a) Ridge or watershed canal (b) Contour canal, (c) Side slope canal.

(a) Ridge or Watershed Canal The canal which is aligned along the ridge line (watershed line) is known as ridge canal or watershed canal. The advantage of this type of canal is that it can irrigate the areas on both sides. Again there is no possibility of crossing any natural drainage and hence no cross-drainage work is necessary (Fig. 5.2).

(b) Contour Canal The canal which is aligned approximately parallel to the contour lines is known as contour canal. This canal can irrigate the areas on one side only. This canal may cross natural drainage and hence cross-drainage works are necessary (Fig. 5.3).

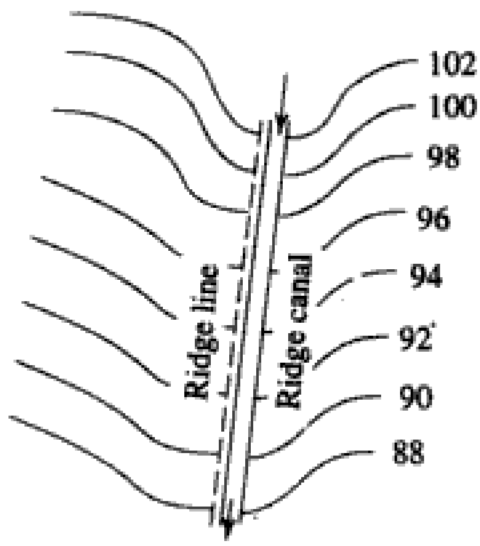


Fig. 5.2 Ridge canal

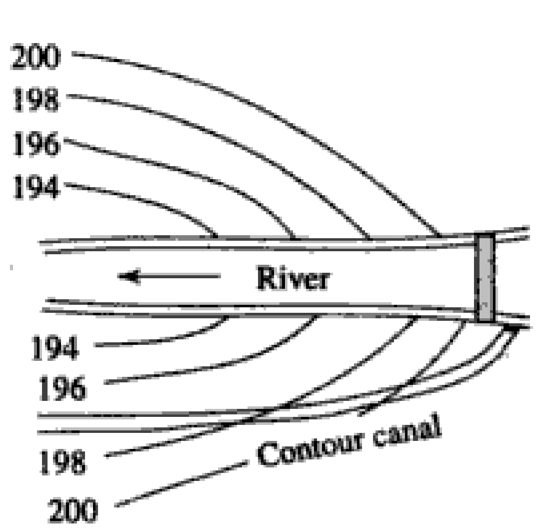


Fig. 5.3 Contour canal

(c) Side Slope Canal The canal which is aligned approximately at right angles to the contour lines is known as side slope canal. It can irrigate the areas on one side only. Again, it does not cross any natural drainage and hence the cross-drainage works are not necessary (Fig. 5.4).

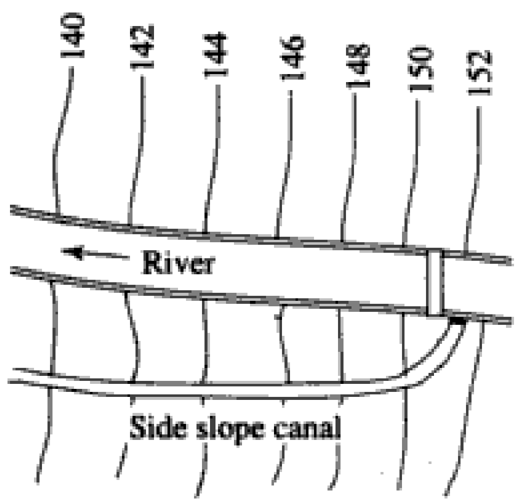


Fig. 5.4 Side slope canal

5.3 SELECTION OF BARRAGE OR DAM SITE

1. Selection of Barrage Site While fixing the site for a barrage, the following points should be considered,

(a) The site should not be on the curvature of the river.

(b) The river should be straight at least for a distance of one kilometre both on the up stream and down stream sides (Fig. 5.5).

(c) The river bank should be well defined.

(d) The width of the river should be minimum.

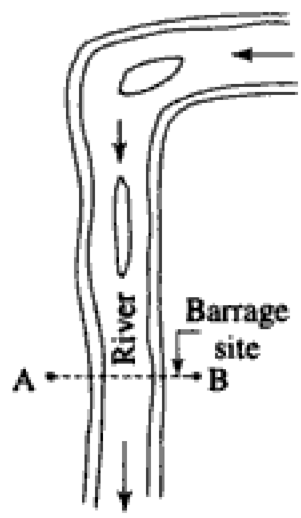


Fig. 5.5 Barrage site

(e) The storage reservoir should not submerge much valuable lands, villages, etc.

(f) The gross command area of the irrigation project should be nearer to the barrage site so that the length of main canal may be minimum to avoid transmission loss.

(g) The elevation of the barrage site should be higher than the command area so that the flow of water by gravity may be achieved.

(h) Construction materials and labours should be available near the site.

2. Selection of Dam Site While fixing the dam site, the certain points should be taken into account.

(a) The site should be on the valley so that deep reservoir may be formed with minimum surface area (Fig. 5.6).

(b) The site should be such that the length of dam may be minimum.

(c) Stable foundation should be available at the site.

(d) At the site, the rocks should not contain cracks, fissures, etc. which may cause loss due to leakage.

(e) Construction materials and labours should be available near the site.

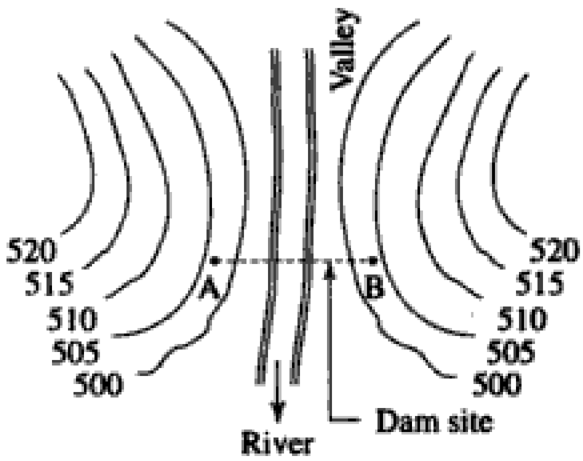


Fig. 5.6 Dam site

5.4 SELECTION OF ALIGNMENT OF PERENNIAL CANAL

The following points should be kept in mind while marking the tentative alignment of a canal,

1. The alignment should not pass through the valuable lands, religious places, villages, etc. to avoid unnecessary compensation and unwanted conflict.
2. The alignment should be short as far as possible, but to make it short the alignment should not be taken through the area where irrigation is not at all possible.
3. The alignment should be straight as far as possible.
4. If the curve is unavoidable in the alignment, then it should be provided according to IS: 5968–1970.
   Some references are given in the following table

Discharge (cumec)	–	*Radius (m)*
80–100	–	1200–1500
30–80	–	800–1000
15–30	–	400–600
5–15	–	100–150

5. The alignment should cross the natural stream, drainage, etc. approximately at right angles. At the crossing point, the width of the drainage should be minimum and the banks should be well defined.
6. The alignment should not involve heavy cutting or banking. It is preferable if balancing depth of cutting and banking may be achieved.
7. The alignment along the ridge line or watershed line is very good as the watershed canal can irrigate the areas on both the sides. Moreover, the cross-drainage works may be avoided.
8. The alignment should be such that the maximum area may be irrigated with minimum length of canal.
9. The alignment should not pass through the marshy land or water logged area, because the canal may collapse due to heavy moisture in the area.
10. The alignment should not pass through sandy soil as the percolation loss will be more and the duty of canal will be less.

5.5 SELECTION OF SITE FOR INUNDATION CANAL

The inundation canal is simply an open cut in the bank of the inundation river. Sometimes, such a canal is taken off from the perennial river. However, the main idea of an inundation canal is that the irrigation facilities may be available without constructing any hydraulic structure or regulation work. While selecting the site for take-off point of the inundation canal, following points should be considered

1. The inundation canal should be taken off from the concave side of the curve in the river. This is to prevent the deposition of silt at the head of the canal (Fig. 5.7).
2. The take-off point should be on well defined banks of the river.
3. The take-off point should be close to the area to be irrigated. This is to avoid unnecessary excavation.
4. If a river consists of shoals in its coarse, then the canal should be taken off from the tail region of the shoal. Because, the shoal will create a pocket in front of the canal head making possible to control the entry of heavy silt in the canal (Fig. 5.8).

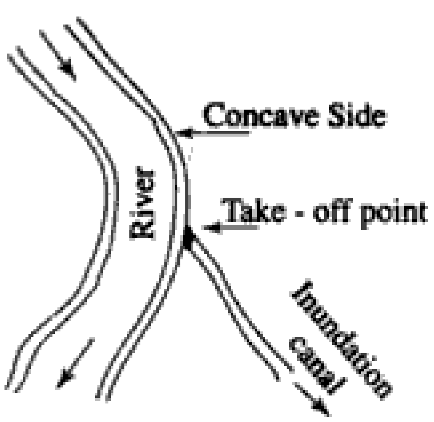


Fig. 5.7 Take-off point

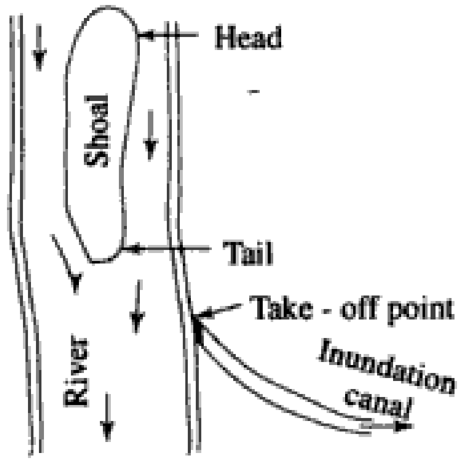


Fig. 5.8 Shoal

5.6 MAINTENANCE OF INUNDATION CANAL

In inundation canal system no silt excluder is constructed at the head of the canal to prevent the deposition of silt. So, every year the head reach and also the whole canal system goes on getting silted. Thus the capacity of the canals go on reducing year by year and ultimately the irrigation system may fail. Hence, provisions and maintenance systems should be provided to keep the canal system workable for a longer duration. The following are measures to be taken.

(1) Subsidiary Canals The subsidiary canals are taken off from the same river to connect the main canal. But the head of these subsidiary canals are kept closed. When the head reach of the main canal gets silted and the discharge capacity gets reduced, the heads of the subsidiary canals are opened to increase the discharge of the main canal. The length of these canals should be kept minimum and these are taken through the uncultivable lands as these are not meant for irrigation but for feeding the main canal when necessary (Fig. 5.9).

(2) Link Canal Sedimentation in the inundation canal system is a problem to be dealt with. To overcome this problem sometimes a link canal is excavated to connect the inundation canals which are taken in series from the same bank of the river. Normally, the head of the link canal is kept closed. When the discharge of the main canal decreases, the link canal is opened to increase the discharge of the main canal. It is sometimes also called the feader canal (Fig. 5.10).

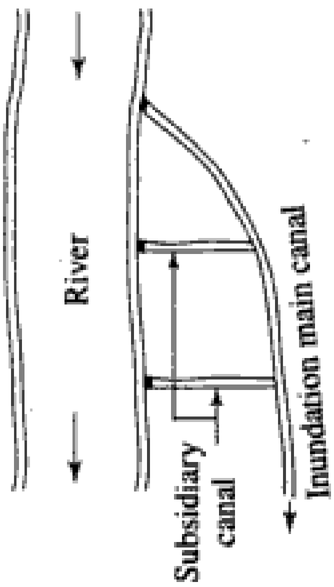


Fig. 5.9 Subsidiary canal

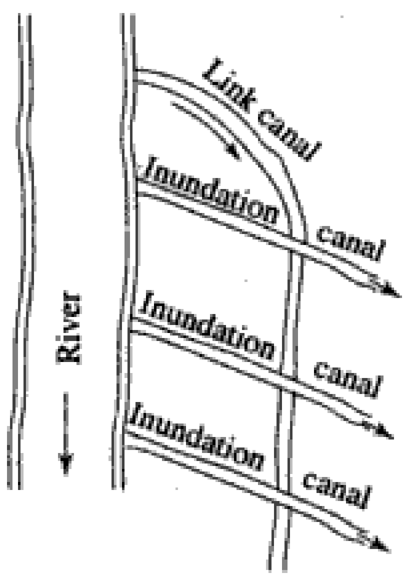


Fig. 5.10 Link canal

(3) Flood Regulator and Escape Channel As there is no controlling system at the head of the inundation canal, the excess water during flood may flow through the canal and submerge the vast area causing damage to the crops and suffering to people. For this reason, the flood regulator and escape channel are provided at suitable place. During heavy rainfall, when the water level of the river rises above the highest flood level (H.F.L), the flood regulator is kept closed and the flood water is sent back to the same river through the escape channel (Fig. 5.11).

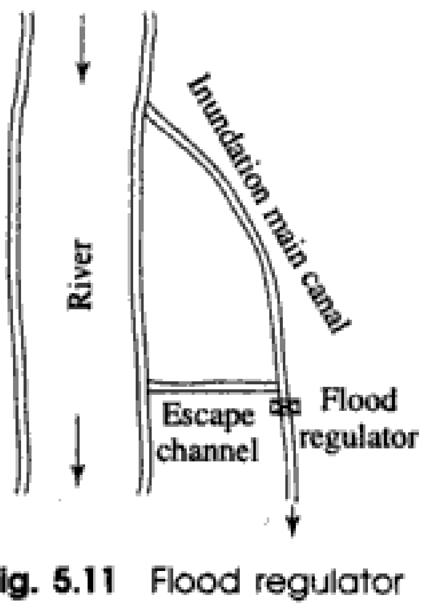


Fig. 5.11 Flood regulator

(4) Re-excavation In spite of all these maintenance works, sometimes it may be found necessary to re-excavate the inundation canals. So, it is better to execute minor excavation stage by stage every year to avoid major excavation after a long period.

5. ADVANTAGES AND DISADVANTAGES OF INUNDATION IRRIGATION

Advantages The following are the advantages of inundation irrigation.

1. In this system, no hydraulic structure is constructed.
2. In this system, the maintenance cost is low.
3. The water carried by the canal contains fine silt which improves the fertility of the land.
4. The canals run in rainy season only, and in other seasons the canals remain practically dry. So, there is no possibility of water-logging.

Disadvantages The following are the disadvantages of this system.

1. As there is no control over the flow of water over irrigation may spoil the crops.
2. The supply of water is uncertain.
3. It is not applicable in rabi season.
4. The head of the canal may be damaged during flood.
5. In case the river changes its course, the whole system is to be abandoned.
6. Silting is the main problem which involves recuring cleaning expenditure.

5.8 COMPARISON BETWEEN INUNDATION AND PERENNIAL IRRIGATION

Inundation irrigation	*Perennial irrigation*
1. The irrigation water is available in rainy season only.	1. The irrigation water is available throughout the year.
2. No hydraulic structure is necessary.	2. Hydraulic structures are necessary, (such as diversion headhead works, cross-drainage works, etc.).
3. The canal water contains plenty of silt which makes the land fertile.	3. The canal water contain practically no silt and hence chemical manure is essential.
4. Large area cannot be included under this system.	4. Large area can be included under this system.
5. The silting of the canal bed is a major problem.	5. Negligible silting takes place in the canal bed.
6. Water tax cannot be imposed.	6. Water tax can be imposed.
7. Initial cost is low.	7. Initial cost is high.
8. No technical persons are required for the operation of the irrigation system.	8. Technical persons are always required for the operation of the irrigation system.
9. The main canal is not provided with regulator and hence there is a possibility of over irrigation.	9. The main canal is provided with head regulator and hence there is no possibility of over irrigation.

5.9 SYSTEM OF BANDHARA IRRIGATION

This is a minor irrigation system suitable for irrigating isolated areas, up to 500 hectares. The bandhara is similar to weir which is constructed across a small stream to raise the water level on the up stream side to divert the water through the canal [Fig. 5.12(a)].

The height of the bandhara depends on the water level to be raised on the up stream side [Fig. 5.12 (b)].

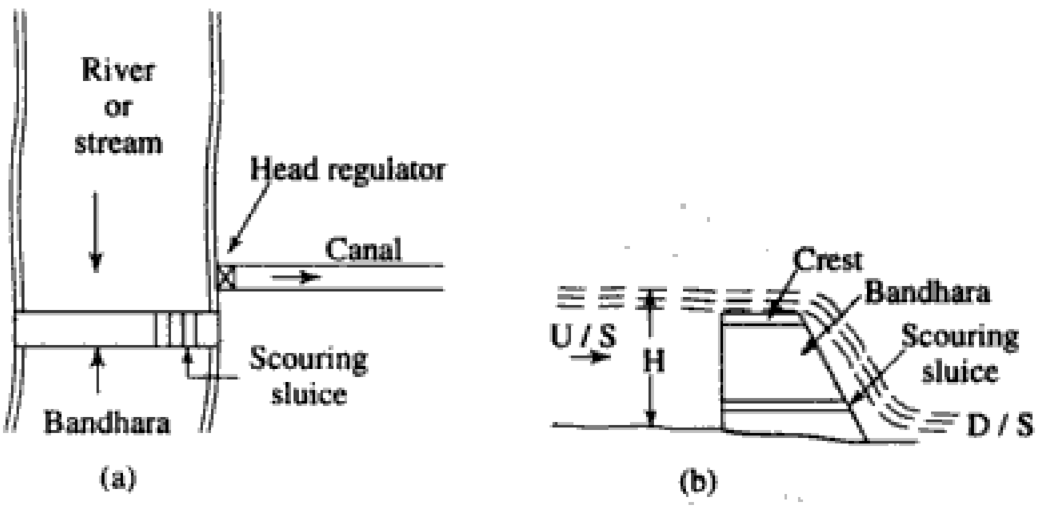


Fig. 5.12 Bandhara Irrigation

It is constructed with brick masonry or stone masonry with R.C.C. crest. The crest width varies from 1 m to 2 m. The scouring sluices are provided at the bottom of the bandhara near the head reach of the canal. The function of scouring sluices is to remove the silt which may get deposited in front of the canal head. Normally, the sluices are kept closed and these are opened when the deposited silt is to be removed. The surplus water is allowed to pass over the crest of the bandhara.

In this system, the water is directly taken from the main canal and supplied to the agricultural land. The total area under a bandhara is known as *Thal*. Again, the *Thal* is divided into several zones which are known an *Phad*. That's why, sometimes this system is known as '*Phad* irrigation system.' (Fig. 5.13).

This system is suitable for small streams. Sometimes, more than one bandhara may be constructed on the same stream at a reasonable interval to

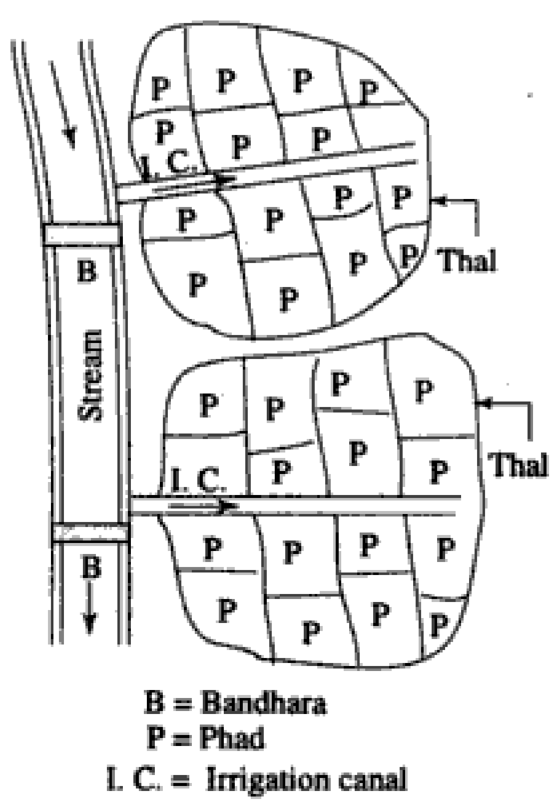


Fig. 5.13 *Thal and phad*

irrigate the areas under different zones. While selecting the site for bandhara, the following points should be kept in mind.

(a) The banks of the stream should be high and well defined.
(b) The source of water should be reliable for constant supply.
(c) Stable foundation should be available for the construction of bandhara.

5.10 ADVANTAGES AND DISADVANTAGES OF BANDHARA IRRIGATION SYSTEM

Advantages The following are the advantages of this system.

(a) The water of small streams can be utilised for irrigation purpose by constructing a simple structure.
(b) The culturable area is generally close to the source. Hence there is less possibility of transmission loss.
(c) As there is no loss due to transmission, evaporation and percolation, the duty of water is high.

Disadvantages The following are the disadvantages of this system (a) Normally, the discharge capacity of small streams is low. Moreover, if bandhara irrigation system is implemented in such streams, the people residing on the down stream side will not get water for their use.

(b) The supply of water mainly depends on rainfall. So, in the period of drought this system is practically useless.

REVIEW QUESTIONS

1. Fill up the blanks with appropriate word/words.
   (i) The river in which the water flows to its full capacity in rainy season only is known as _________ river.
   (ii) The river in which the water flows more or less to its full capacity throughout the year is known as _________ river.
   (iii) The canal which is constructed to feed another canal or river is known as _________ canal.
   (iv) The canal which is constructed to generate hydroelectric power is known as _________ canal.
   (v) The canal in which water flows in rainy season only is known as _________ canal.
   (vi) The canal in which water flows throughout the year is known as _________ canal.
   (vii) The canal which is aligned along the watershed line is known as _________ canal.
   (viii) The canal which is aligned at right angles to the contour lines is known as _________ canal.
   (ix) The area included under bandhara irrigation system is known as

(x) In bandhare irrigation, the '*thal*' is divided into several zones which are known as _________.

2. Distinguish between the following,
   (a) Power canal and feeder canal.
   (b) Inundation canal and perennial canal.
   (c) Contour canal and watershed canal.
   (d) Main canal and branch canal.
3. How are canals classified based on the purpose of service?
4. How are canals classified based on the discharge capacity?
5. How are canals classified based on the alignment?
6. Explain the measures to be taken for the maintenance of inundation canals.

ANSWERS

1. (i) Inundation (ii) Perennial
   (iii) Feeder (iv) Power
   (v) Inundation (vi) Perennial
   (vii) Water shed (viii) Side slope
   (ix) Thal (x) Phad

6

CANAL SECTION

6.1 TERMS RELATING TO CANAL SECTION

The canal section may be in fully cutting or fully banking or partial cutting and partial banking according to the natural ground surface and the permissible bed slope of the canal. But there are several terms in the canal section with which a civil engineer should be acquainted to design the section and to execute the work. The following are the different terms related to the canal section (Fig. 6.1).

1. Canal bank
2. Berm
3. Hydraulic gradient
4. Counter berm
5. Free board
6. Side slope
7. Service road or inspection road
8. Dowel or Dowla
9. Borrowpit
10. Spoil bank
11. Land width

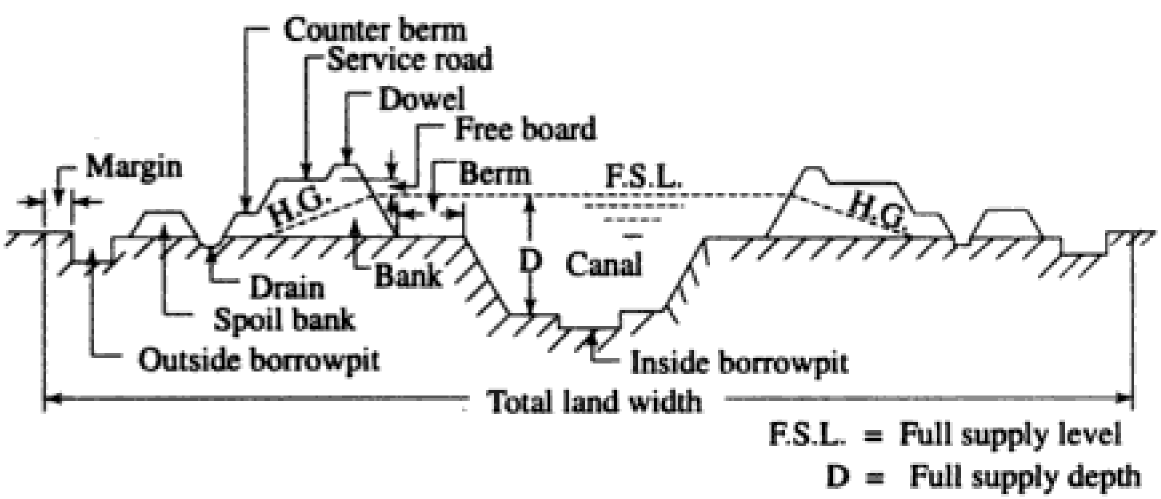


Fig. 6.1 Canal section

6.2 CANAL BANK

The canal bank is necessary to retain water in the canal to the full supply level. But the section of the canal bank is different for different site conditions. The following are the different forms for different site conditions.

(a) When the Canal Fully in Cutting In this case, the banks are constructed on both sides of the canal to provide only a inspection road. Here, the hydraulic gradient has no function. So, the height of the bank will be low and the top width will be minimum just to provide the road way. The side slope will be $1\frac{1}{2}:1$ or $2:1$ according to the nature of the soil (Fig. 6.2).

(b) When the Canal in Partial Cutting and Banking In this case, the banks are constructed on sides of the canal to retain water. The height of the banks depend on the fully supply level of the canal. Again, the section of the canal depends on the hydraulic gradient. The top width and the side slope of the bank should be such that the hydraulic gradient should have a minimum cover of 0.5 m (Fig. 6.3).

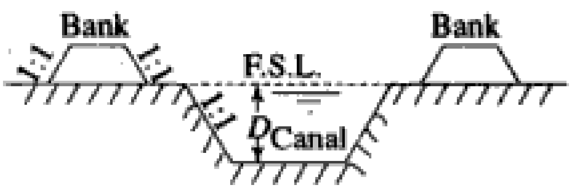


Fig. 6.2 Canal in full cutting

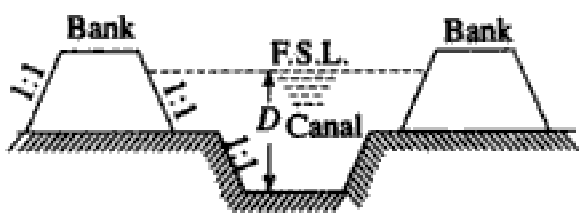


Fig. 6.3 Canal in partial cutting and partial banking

(c) When the Canal in Full Banking In this case, the canal and both the canal banks are constructed above the ground level. The height of the bank will be high and its section will be large due to the hydraulic gradient. But to minimise the cross section of the bank a core wall of puddle clay is provided which deflects the hydraulic gradient downwards (Fig. 6.4).

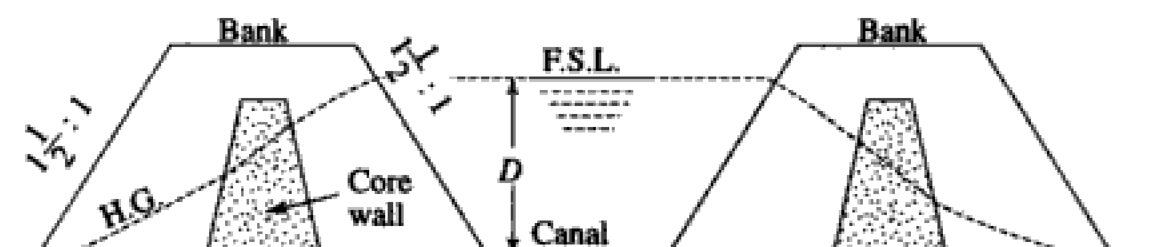


Fig. 6.4 Canal in full banking

6.3 BERM

The distance between the toe of the bank and the top edge of cutting is termed as berm (Fig. 6.5). The berm is provided for the following reasons,

(a) To protect the bank from erosion.
(b) To provide a space for widening the canal section in future if necessary.
(c) To protect the bank from sliding down towards the canal section.

(d) The silt deposition on the berm makes an impervious lining.
(e) If necessary borrowpit can be excavated on the berm.

The width of the berm depends on various factors such as capacity of the canal, the nature of the soil, the site condition, etc. However, the width of the berm varies from *D* to 2 *D*, where, *D* is the full supply depth of the canal.

6.4 HYDRAULIC GRADIENT

When the water is retained by the canal bank, the seepage occurs through the body of the bank. Due to the resistance of the soil, the saturation line forms a sloping line which may pass through countryside of the bank. This sloping line is known as the hydraulic gradient or saturation gradient. The soil below this line is saturated, but the soil above this line is dry. The hydraulic gradient depends on the permeability of the soil. So, while constructing the bank, the soil should be tested in soil testing laboratory and the nature of the hydraulic gradient should be ascertained. This will help in fixing the height, top width and side slope of the bank. The following are the approximate values of hydraulic gradient for different soil (Fig. 6.6).

Soil		H.G.
Clayey soil	–	1:4
Alluvial soil	–	1:5
Sandy soil	–	1:6

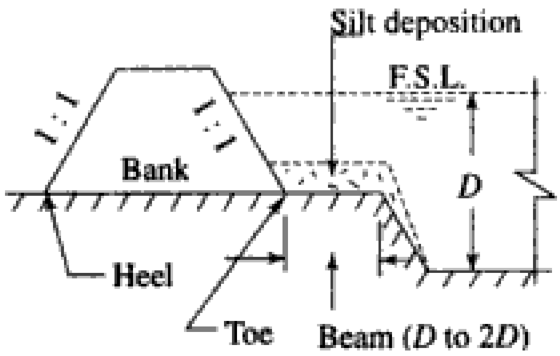


Fig. 6.5 Berm

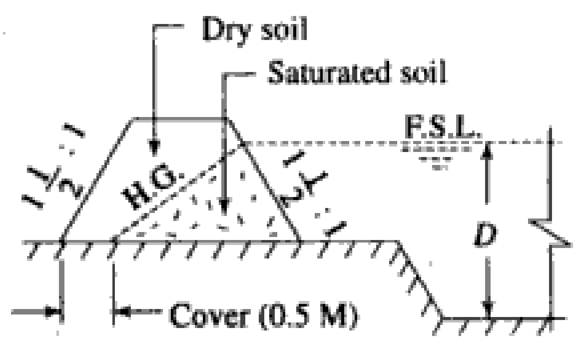


Fig. 6.6 Hydrualic Gradient

6.5 COUNTER BERM

When the water is retained by a canal bank the hydraulic gradient line passes through the body of the bank. For stability of the bank, this gradient should not intersect the outer side of the bank. It should pass through the base and a minimum cover of 0.5 m should always be maintained. Sometimes, it may occur that the hydraulic gradient line intersects the outer side of the bank. In that case, a projection is provided on the bank to obtain minimum cover. This projection is known as counterberm. The width of this berm depends on the site condition (Fig. 6.7).

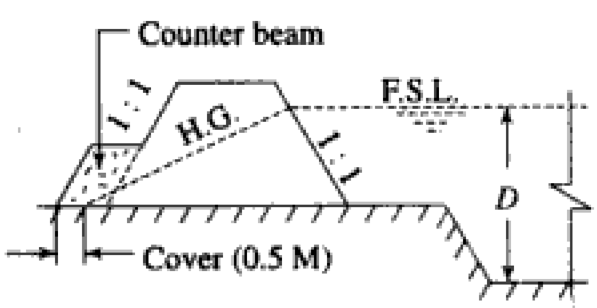


Fig. 6.7 Counter berm

6.6 FREE BOARD

It is the distance between the full supply level and top of the bank. The amount of free board varies from 0.6 m to 0.75 m.

It is provided for the following reasons (Fig. 6.8).

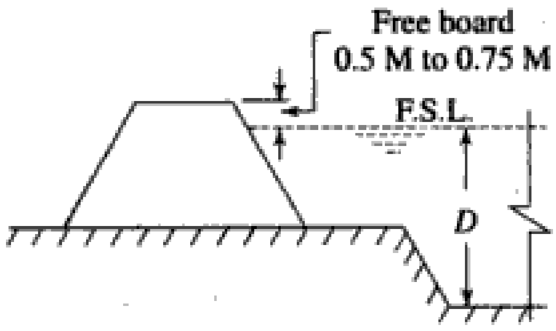


Fig. 6.8 Free board

(a) To keep a sufficient margin so that the canal water does not overtop the bank in case of heavy rainfall or fluctuation in water supply.
(b) To keep the saturation gradient much below the top of the bank.

6.7 SIDE SLOPE

The side slopes of the canal bank and canal section depend on the angle of repose of the soil existing on the site. So, to determine the side slopes of different sections, the soil samples should be collected from the site and should be tested in the soil testing laboratory. The necessity of such test is that if the permissible slope (to maintain angle of repose) is not provided in an embankment or cutting, then the soil in that place will go on sliding gradually until the angle of repose for that particular soil is attained.

For instance, suppose an embankment was constructed with side slope 1:1 but according to the nature of the soil, the side slope should be $1\frac{1}{2}$: 1. Then the initial shape A B C D will automatically take the final shape A_1, B_1, C_1, D_1 after slide in the due course (Fig. 6.9).

Again, an opposite incident may occur, suppose, an embankment was constructed with side slope 2:1, but latter it was found that the side slope of 1:1 was sufficient to maintain the angle of repose for that soil. In this case, an unnecessary earthwork was done (Fig. 6.10).

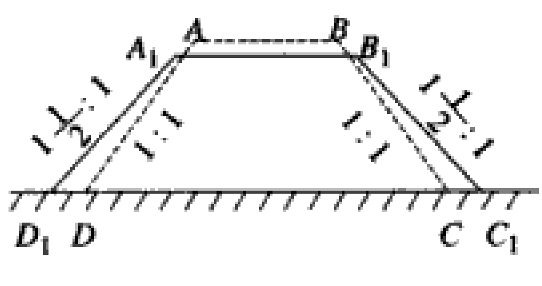


Fig. 6.9 Sliding of bank

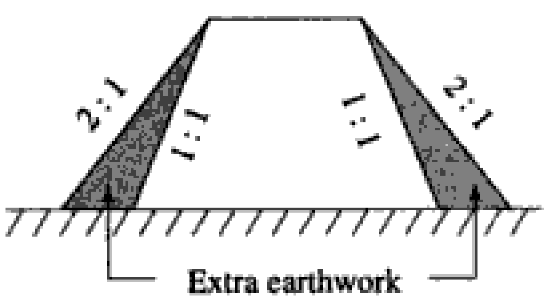


Fig. 6.10 Extra earth filling

The permissible side slopes for some soil are given in the following table:

Type of soil	*Side slope in cutting*	*Side slope in banking*
Clayey soil	1:1	$1\frac{1}{2}$:1
Alluvial soil	1:1	2:1
Sandy loam	$1\frac{1}{2}$:1	2:1
Sandy soil	2:1	3:1

6.8 SERVICE ROAD

The roadway which is provided on the top of the canal bank for inspection and maintenance works is known as service road or inspection road. For main canal, the service roads are provided on both the banks. But for branch canals, the road is provided on one bank only. The width of the service roads for main canal varies from 4 to 6 m. The width of the road for the branch canal varies from 3 to 4 m.

The initial purpose of the service road is to conduct inspection and maintenance works. But finally these roads serve the purpose of communication between the different villages and for transporting agricultural goods. Therefore it becomes necessary to construct metalled road to serve these purposes (Fig. 6.11).

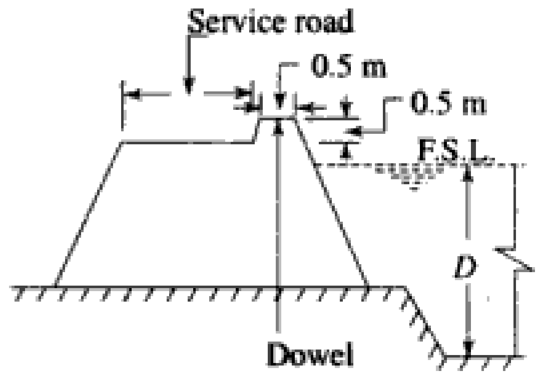


Fig. 6.11 Service road and dowel

6.9 DOWEL OR DOWLA

The protective small embankment which is provided on the canal side of the service road for the safety of the vehicles plying on it is known as dowel or dowla. Practically it acts as a curb on the canal side of the road. It is provided above the F.S.L. with a provision of freeboard. The top width is generally 0.5 m and the height above the road level is about 0.5 m. The side slope is similar to the side slope of the bank (Fig. 6.11).

6.10 SPOIL BANK

When the canal is constructed in full cutting, the excavated earth may not be completely required for forming the bank. In such a case, the extra earth is deposited in the form of small banks which are known as spoil banks. The spoil banks are provided on one side or both sides of the canal bank depending on the quantity of excess earth and the available space. The spoil banks run parallel to the main bank. But are not continuous, sufficient spaces are left between the adjacent spoil banks for proper drainage (Fig. 6.12).

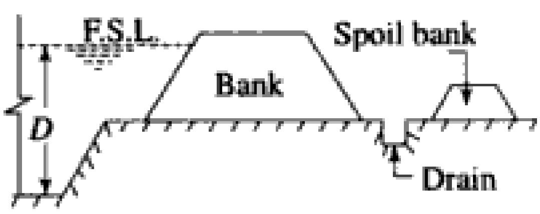


Fig. 6.12 Spoil bank

6.11 BORROWPIT

When the canal is constructed in partial cutting and partial banking, the excavated earth may not be sufficient for forming the required bank. In such a case, the extra earth required for the construction of banks is taken from some pits which are known as borrowpits. The borrowpits may be inside or outside, the canal.

The inside borrowpit may be located at the centre of the canal. The width of the borrowpit should be half of the base width of canal. The maximum depth should be 1 m. The excavation is done in a number of borrowpits leaving a gap between them. The gap is generally half of the length of each borrowpit. The idea behind this is that the borrowpits will act as water pockets where the silt will be deposited and ultimately the canal bed will get levelled up.

The outer borrowpit may be adjacent to the heal of the bank with a clearance of 1 m between the heel and edge of borrowpit. But the outer borrowpit may create some inconvenience. So, it is better to borrow earth from the barren lands far away from the canal (Fig. 6.13).

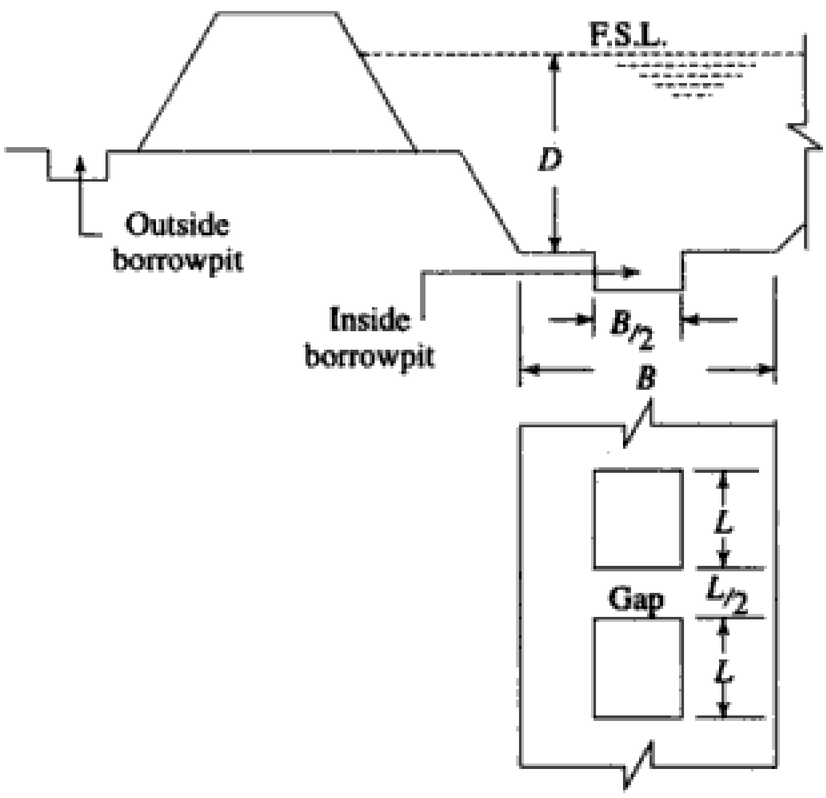


Fig. 6.13 Borrowpits

6.12 LAND WIDTH

The total land width required for the construction of a canal depends on the nature of the site condition, such as fully in cutting or fully in banking or partly in cutting and partly in banking. These conditions arise according to the designed bed level of the canal and the natural ground surface. So, total land width differs with the site condition. However, to determine the total land width the following dimensions should be added

1. Top width of the canal.
2. Twice the berm width.
3. Twice the bottom width of banks.
4. A margin of one metre from the heel of the bank on both sides.

5. Width of external borrowpit if any.
6. A margin of 0.5 m from the outer edge of borrowpit on both sides, if external borrowpit becomes necessary.

6.13 BALANCING DEPTH

In constructing a canal section, if the quantity of excavated earth can be fully utilised for making the banks on both sides, then that canal section is known as economical section. The depth of cutting for that ideal condition is known as balancing depth. In this case, no borrowpit on spoil bank needs to be constructed. This condition may not occur in all the cases. It happens only when the canal section is partly in cutting and partly in banking. The cost of earth work will also be balanced.

The method of finding the balancing depth is described here.

Example Find the balancing depth for a canal section having the following data.

1. Base width of canal = 10 m.
2. Side slope in cutting = 1 : 1.
3. Side slope in banking = 2 : 1.
4. Top width of bank = 3 m.
5. Height of bank above G.L. = 3 m.

Solution Refer Fig. 6.14.

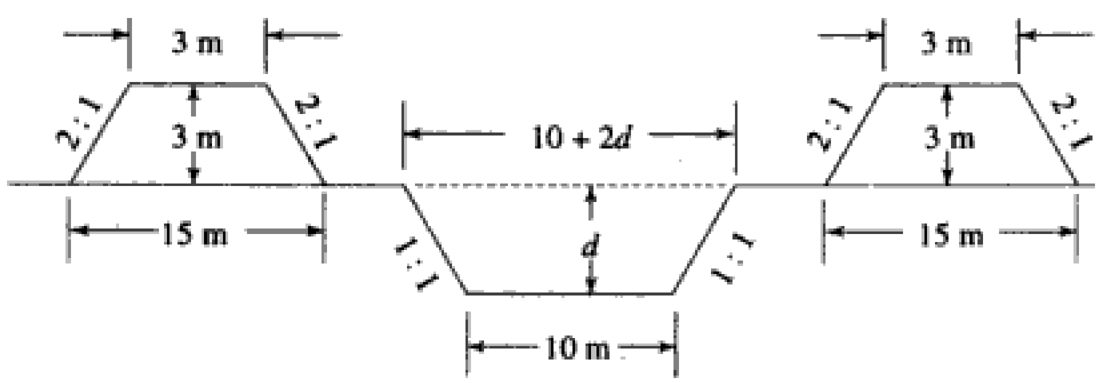


Fig. 6.14

$$\text{Area of banking} = 2 \times \frac{15+3}{2} \times 3 = 54 \text{ sq. m.} \qquad (1)$$

Let d be the balancing depth of cutting.

$$\text{Area of cutting} = \frac{10+10+2d}{2} \times d = (10+d)\,d \qquad (2)$$

Equating the area of banking and cutting,

$$(10 + d)\,d = 54$$

$$d^2 + 10d - 54 = 0$$

$$d = \frac{-10 \pm \sqrt{100+216}}{2} = \frac{-10 \pm 17.8}{2}$$

$$d = \frac{-10+17.8}{2} = 3.89 \text{ m} \qquad \text{(Neglecting the negative sign)}$$

So, the balancing depth is 3.89 m.

6.14 CANAL IN FULL CUTTING

Draw the section of a canal in full cutting with the following data

(a) R.L. of G.L. = 152.50 m.
(b) R.L. of canal bed = 150.00 m.
(c) Bed width of canal = 15.00 m.
(d) Top width of bank = 3 m.
(e) height of bank = 1.5 m.
(f) Berm = 1.0 m.
(g) Side slope in cutting = 1:1
(h) Side slope in banking = 2:1
(i) Full supply depth = 2.5 m.

Find also the total land width required.

Solution Refer Fig. 6.15.

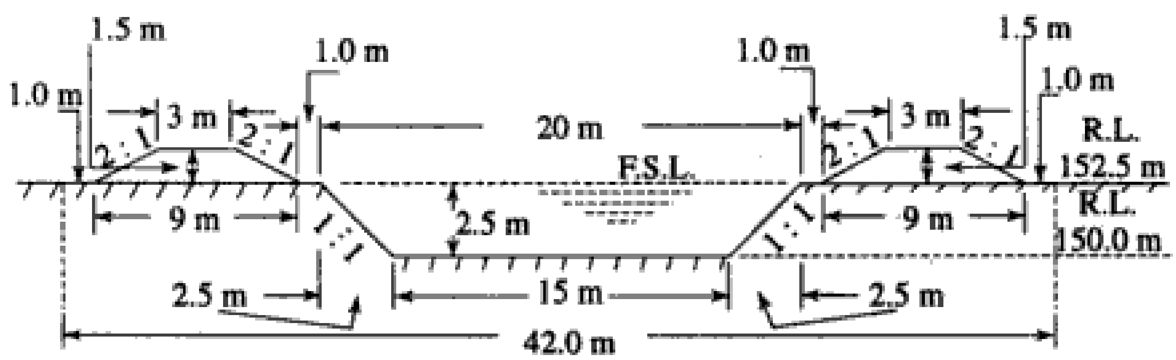


Fig. 6.15

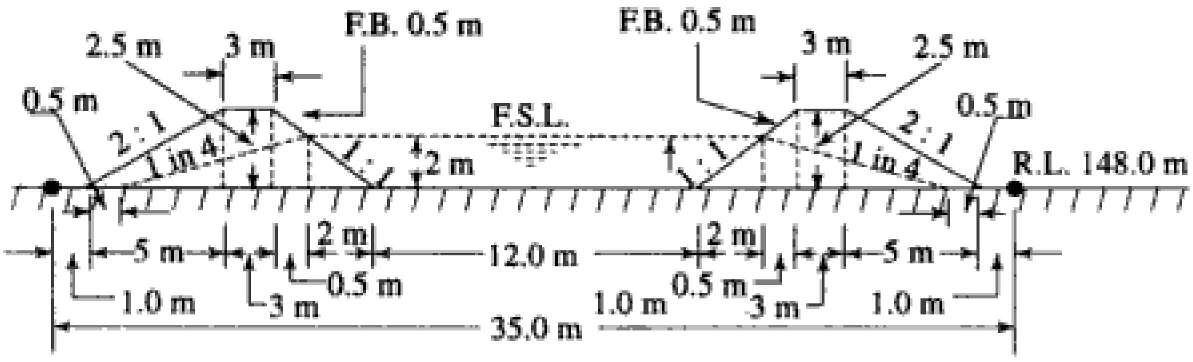
6.15 CANAL IN FULL BANKING

Draw the section of a canal in full banking with the following data

(a) R.L. of G.L. = 148.00.
(b) The canal bed is just at G.L.
(c) Bed width of canal = 12.0 m.
(d) Fully supply depth = 2.0 m.
(c) Free board = 0.5 m.
(f) Hydraulic gradient = 1 in 4.
(g) Top width of bank = 3 m.
(h) Side slope of bank – {1:1 (canal side), 2:1 (country side)}

Final also the land width.

Solution Refer Fig. 6.16.



R.F. = 1 : 300

Fig. 6.16

6.16 CANAL IN PARTIAL CUTTING AND PARTIAL BANKING

Draw the section of a canal in partial cutting and partial banking with the following data

(a) R.L. of G.L. = 150.00 m.
(b) R.L. of canal bed = 149.00 m.
(c) Full supply depth = 2.0 m.
(d) Free board = 0.5 m.
(e) Berm = 1.0 m.
(f) Canal bed width = 10.0 m.
(g) Top width of bank = 3.0 m.
(h) H.G. line = 1 in 6.
(i) Side slope in banking = 2 : 1.
(j) Side slope in cutting = 1 : 1.

Mark the total land width.

Solution Refer Fig. 6.17.

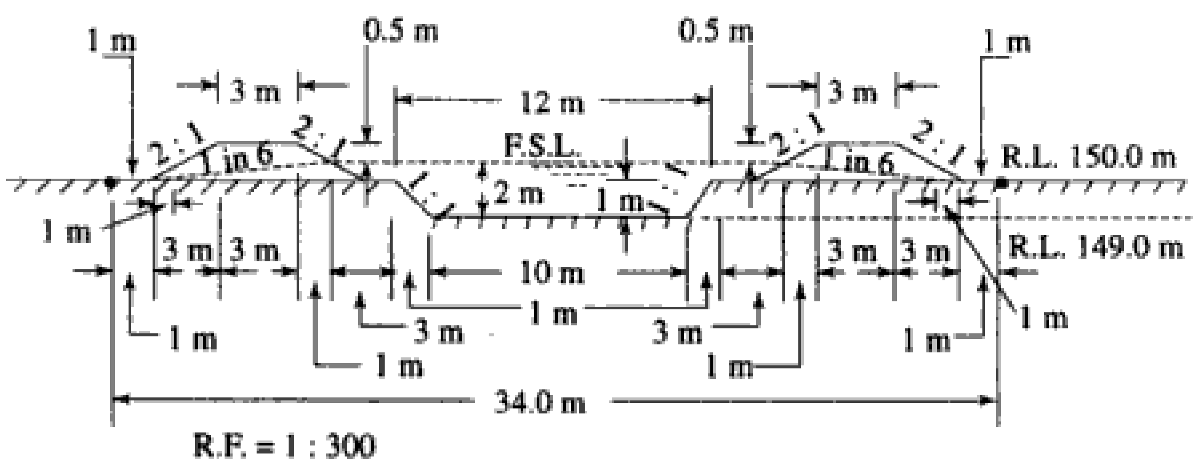


Fig. 6.17

REVIEW QUESTIONS

1. Fill up the blanks with appropriate word/words.
   (i) The distance between the top of the bank and top edge of cutting is known as _________

(ii) In case of canal bank a _________ may be required to provide a minimum cover over the hydraulic gradient.
(iii) The distance between the F.S.L. and the top of embankment is known as _________ .
(iv) For the safety of the vehicle running, over the service road _________ is provided.
(v) In case of canal in full cutting a _________ bank may be required to be constructed.
(vi) In case of canal bank sometimes extra earth is taken from _________ .

2. Distinguish between the following terms.
   (a) Berm and counter berm.
   (b) Spoil bank and canal bank.
   (c) Hydraulic gradient and longitudinal gradient.
3. What are the different types of canal section? Explain with sketch.
4. (a) What is meant by balancing depth of cutting and banking?
   (b) Why it is required?
   (c) How it is determined?
5. Distinguish, with neat sketch, between the canal in full cutting and the canal in full banking.

ANSWERS

1. (i) berm
   (ii) counter berm
   (iii) free board
   (iv) dowel
   (v) spoil
   (vi) borrowpit

7

CANAL LINING

7.1 OBJECT OF CANAL LINING

The following are the main objects of canal lining:

(a) To Control Seepage The seepage loss is the maximum loss in unlined canals. Due to seepage the duty of canal water is much reduced which involves enhancement of storage capacity of a reservoir by constructing high dam. Thus, the expenditure of the project is increased. So, to control the seepage loss through the bed and sides of the canal, the lining of the canal is necessary.

(b) To prevent Water-Logging Along the course of the canal, there may be low lying areas on one side or both sides of the canal. Due to the seepage of water through the sides of the canal, these areas may get converted into marshy lands. This water-logging makes the land alkaline which is unsuitable for agriculture. This water-logged area may become the breeding place of mosquitos which are responsible for many infectious diseases.

(c) To Increase the Capacity of Canal In unlined canal, the velocity of flow should be fixed such that the silting and scouring is avoided. In practice, the velocity should always be kept below 1 m/sec. Due to the low velocity, the discharge capacity of the canal becomes low. In unlined canal, if the capacity of the canal is to be increased the cross-sectional area has to be increased which involves more land width. So, the lining of the canal should be such that the velocity and the discharge of the canal are more with minimum cross-sectional area.

(d) To Increase the Command Area If the lining is provided in the canals the various losses can be controlled and ultimately the command area of the project may be enhanced.

(e) To Protect the Canal from the Damage by Flood The unlined canals may be severely damaged by scouring and erosion caused due to the high velocity of flood water at the time of heavy rainfall. So, to protect the canals from the damage, the lining should be provided.

(f) To Control the Growth of Weeds The growth of various types of weeds along the sides of the canals is a common problem. Again, some types of weeds are found to grow along the bed of the canals. These weeds reduce the velocity of flow and the capacity of the canals. So, the unlined canals require excessive maintenance works for clearing the weeds. If lining is provided in the canal, the growth of weeds can be stopped and velocity and the capacity of the canal may be increased.

7.2 TYPES OF LINING

The following are the different types of linings which are generally recommended according to the various site conditions.

1. Cement concrete lining
2. Pre-cast concrete lining
3. Cement mortar lining
4. Lime concrete lining
5. Brick lining
6. Boulder lining
7. Shot crete lining
8. Asphalt lining
9. Bentonite and clay lining
10. Soil-cement lining

7.3 CEMENT CONCRETE LINING

This lining is recommended for the canal in full banking. The cement concrete lining (cast in-situ) is widely accepted as the best impervious lining. It can resist the effect of scouring and erosion very efficiently. The velocity of flow may be kept above 2.5 m/sec. It can eliminate completely growth of weeds. The lining is done by the following steps,

(a) Preparation of sub-grade The sub-grade is prepared by ramming the surface properly with a layer of sand (about 15 cm). Then, a slurry of cement and sand (1:3) is spread uniformly over the prepared bed.

(b) Laying of concrete The cement concrete of grade M_{15} is spread uniformly according to the desired thickness (generally, the thickness varies from 100 mm to 150 mm). After laying, the concrete is tapped gently until the slurry comes on the top. The curing is done for two weeks. As the concrete is liable to get damaged by the change of temperature, the expansion joints are provided at appropriate places. Normally no re-inforcement is required for this cement concrete. But in special cases, a network of 6 mm diameter rods may be provided with spacing 10 cm centre to centre (Fig. 7.1).

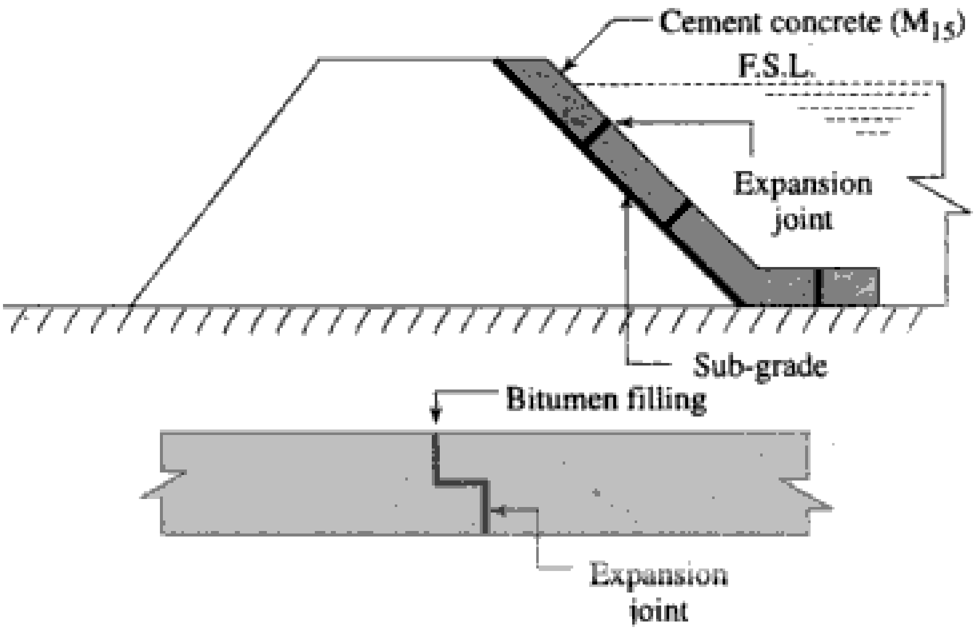


Fig. 7.1 Cement concrete lining

7.4 PRE-CAST CONCRETE LINING

This lining is recommended for the canal in full banking. It consists of pre-cast concrete slabs of size 60 cm × 60 cm × 5 cm which are set along the canal bank and bed with cement mortar (1 :6). A network of 6 mm diameter rod is provided in the slab with spacing 10 cm centre of centre. The proportion of the concrete is recommended as 1 : 2 : 4. Rebates are provided on all the four sides of the slab so that proper joints may be obtained when they are placed side by side. The joints are finished with cement mortar (1:3). Expansion joints are provided at a suitable interval. The slabs are set in the following sequence,

(a) The sub-grade is prepared by properly ramming the soil with a layer of sand. The bed is levelled so that the slabs can be placed easily.

(b) The slabs are stacked as per estimate along the course of the canal. The slabs are placed with cement mortar (1 : 6) by setting the rebates properly. The joints are finished with cement mortar (1:3).

(c) The curing is done for a week (Fig. 7.2).

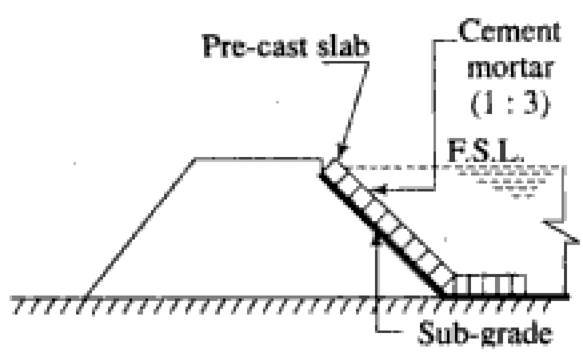


Fig. 7.2 Pre-cast concrete lining

7.5 CEMENT MORTAR LINING

This type of lining is recommended for the canal fully in cutting where hard soil or clayey soil is available. The thickness of the cement mortar (1:4) is generally 2.5 cm. The sub-grade is prepared by ramming the soil after cutting. Then, over the conpacted sub-grade, the cement mortar is laid uniformly and the surface is finished with neat cement polish. This lining is impervious, but is not durable. The curing should be done properly (Fig. 7.3).

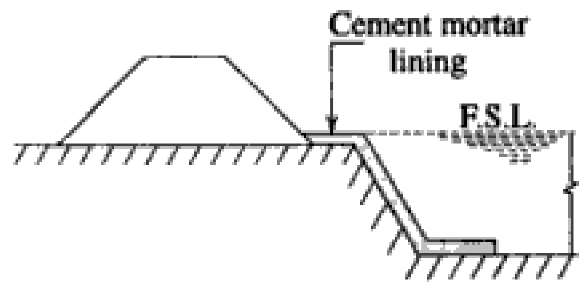


Fig. 7.3 Cement mortar lining

7.6 LIME CONCRETE LINING

When hydraulic lime, surki and brick ballast are available in plenty along the course of the canal or in the vicinity of the irrigation project, then the lining of the canal may be made by the lime concrete of proportion 1:1:6. The procedure of laying this concrete is same as that of the cement concrete lining. Here, the thickness of concrete varies from 150 mm to 225 mm and the curing should be done for longer period. This lining is less durable than the cement concrete lining. However, it is recommended because of the availability of the materials and also because of the economics.

7.7 BRICK LINING

This lining is prepared by the double layer brick flat soling laid with cement mortar (1:6) over the compacted sub-grade. The first class bricks should be recommended for the work. The surface of the lining is finished with cement plaster (1:3) (Fig. 7.4). The curing should be done perfectly.

This lining is always preferred for the following reasons,

(a) This lining is economical.
(b) Work can be done very quickly,
(c) Expansion joints are not required.
(d) Repair works can be done easily.
(e) Bricks can be manufactured from the excavated earth near the site.

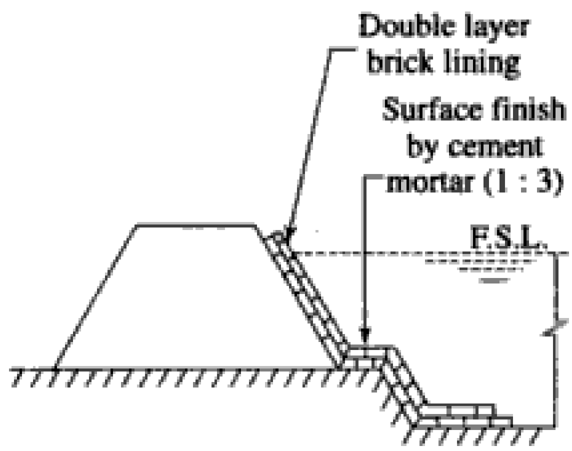


Fig. 7.4 Brick lining

However this lining has certain disadvantages,

(a) It is not completely impervious.
(b) It has low resistance against erosion.
(c) It is not so much durable.

7.8 BOULDER LINING

In hilly areas where the boulders are available in plenty, this type of lining is generally recommended. The boulders are laid in single or double layer maintaining the slope of the banks and the bed level of the canal. The joints of the boulders are grouted with cement mortar (1:6). The surface is finished with cement mortar (1:3). Curing is necessary in this lining too. This lining is very durable and impervious. But the transporting cost of the material is very high. So, it cannot be recommended for all cases (Fig. 7.5).

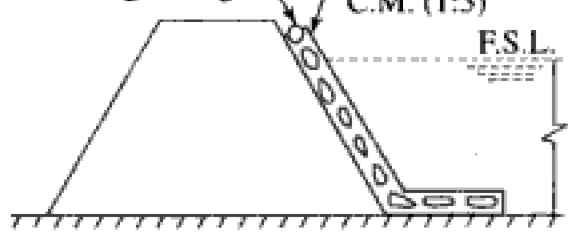


Fig. 7.5 Boulder lining

7.9 SHOT CRETE LINING

In this system, the cement mortar (1:4) is directly applied on the sub-grade by an equipment known as cement gun. The mortar is termed as shot crete and the lining is known as shot crete lining. The process is also known as guniting, as a gun is used for laying the mortar. Sometimes, this lining is known as gunited lining. The lining is done in two ways,

(a) By Dry Mix In this method, a mixture of cement and moist sand is prepared and loaded in the cement gun. Then it is forced through the nozzle of the gun with the help of compressed air. The mortar spreads over the sub-grade to a thickness which varies from 2.5 cm to 5 cm.

(b) By Wet Mix In this process, the mixture of cement, sand and water is prepared according to the approved consistency. The mixture is loaded in the gun and forced on the sub-grade.

This type of lining is very costly and it is not durable. It is suitable for resurfacing the old cement concrete lining.

7.10 ASPHALT LINING

This lining is prepared by spraying asphalt (i.e. bitumen) at a very high temperature (about 150°C) on the subgrade to a thickness varies from 3 mm to 6 mm. The hot asphalt when becomes cold forms a water proof membrane over the subgrade. This membrane is covered with a layer of earth and gravel. The lining is very cheap and can control the seepage of water very effectively but it cannot control the growth of weeds.

7.11 BENTONITE AND CLAY LINING

In this lining a mixture of bentonite and clay are mixed thoroughly to form a sticky mass. This mass is spread over the sub-grade to form an impervious membrance which is effective in controlling the seepage of water, but it cannot control the growth of weeds. This lining is generally recommended for small channels.

7.12 SOIL-CEMENT LINING

This lining is prepared with a mixture of soil and cement. The usual quantity of cement is 10 per cent of the weight of dry soil. The soil and cement are thoroughly mixed to get an uniform texture. The mixture is laid on the sub-grade and it is made thoroughly compact. The lining is efficient to control the seepage of water, but it cannot control the growth of weeds. So, this is recommended for small channels only.

7.13 ADVANTAGES AND DISADVANTAGES OF CANAL LINING

Advantages

1. It reduces the loss of water due to seepage and hence the duty is enhanced.
2. It controls the water logging and hence the bad effects of water-logging are eliminated.
3. It provides smooth surface and hence the velocity of flow can be increased.
4. Due to the increased velocity the discharge capacity of a canal is also increased.
5. Due to the increased velocity, the evaporation loss also be reduced.
6. It eliminates the effect of scouring in the canal bed.
7. The increased velocity eliminates the possibility of silting in the canal bed.
8. It controls the growth of weeds along the canal sides and bed.
9. It provides the stable section of the canal.
10. It reduces the requirement of land width for the canal, because smaller section of the canal can produce greater discharge.
11. It prevents the sub-soil salt to come in contact with the canal water.
12. It reduces the maintenance cost for the canals.

Disadvantages

1. The initial cost of the canal lining is very high. So, it makes the project very expensive with respect to the output.
2. It involves much difficulties for repairing the demaged section of lining.
3. It takes too much time to complete the project work.
4. It becomes difficult, if the outlets are required to be shifted or new outlets are required to be provided, because the dismantling of the lined section is difficult.

7.14 SELECTION OF TYPE OF LINING

The selection of particular type of lining depends on the following factors,

(1) Imperviousness When the canal passes through the sandy soil, the seepage loss is maximum and the canal is unstable. So, to make the canal perfectly impervious and reasonably stable, the most impervious types of linings should be recommended such as cement concrete lining, pre-cast concrete lining, boulder lining, etc.

(2) Smoothness The smoothness of the canal bed and sides, increases the velocity of flow which further increases the discharge of the canal. Due to the increased discharge, the duty of water will be more. So, to increase the duty, the canal surface should be made smooth. The lining like cement concrete, pre-cast cement concrete, etc. gives smooth surface to the canal.

(3) Durability The ultimate benefit of any project depends on the durability of the hydraulic structures, canals, etc. So, to make the canal section durable against all adverse effects like scouring, erosion, weather action, etc. the most strong and impervious types of lining should be recommended.

(4) Economy The lining should be economically viable with the benefits that may be accrued from the expected revenue, yield of crop, etc. So, by studying the overall benefits the type of lining should be recommended.

(5) Site Condition The canal may pass through the marshy land, loose sandy soil, alluvial soil, black clayey soil, hard soil, etc. So, according to the soil and site condition the type of lining should be recommended.

(6) Life of Project Every project should be designed to serve the future three or four decades successfully. The type of lining should be recommended keeping in mind the life of the project.

(7) Availability of Construction Materials The expenditure of lining depends on the availability of construction materials, carriage charges, etc. To reduce the expenditure of lining, the materials which are available in the vicinity of the project should be utilised.

7.15 JUSTIFICATION OF LINING

To justify the lining of canal, the following points should be kept in mind.

(a) Capacity of Reservoir If it is found that a reservoir can be formed for some maximum capacity which is just sufficient to fulfil the water requirement of the project, but increase in the reservoir capacity is not at all possible. In such a case, it is justified to eliminate the losses due to seepage, evaporation, etc from the lining of canal.

(b) Reduction of Dam Height In unlined canal system of irrigation, the maximum water is lost due to seepage, evaporation, etc. To overcome these losses the capacity of the reservoir is increased by increasing the dam height. If canal lining is provided, these unnecessary losses may be controlled. In such a case, it is not required to increase the reservoir capacity by increasing the dam height.

(c) Location of Command Area If the command area of the irrigation project is far away from the reservoir site, then the length of the canal will be more and consequently the transmission losses will be more. These losses will effect the project in various ways. So having the lining of the canal is justified to control the unnecessary losses.

(d) Nature of Land If the command area of the project consists of valuable fertile land, then all measures should be taken so that minimum land width may be required for the construction of canals. In this case, the lining should be compulsory, as it requires smaller cross-sectional area for greater discharge. Thus, the land width can be reduced by providing canal lining.

(e) Cost-benefit Study The cost and benefit of the project should be studied in details.

(1) Cost The annual cost of the project should be worked out by considering the following items,

(a) The annual cost of lining.
(b) The annual interest of the money invested for lining.
(c) Other annual expenditure allied with lining work.

Let the total annual cost be denoted by C.

(2) Benefits The annual benefits of the project should also be worked out by considering the following items,

(a) The annual saving of money by eliminating the seepage with respect to the unlined canal.
(b) The annual saving of money for maintenance work with respect to the unlined canal.

Suppose, the total annual benefit is denoted by B. Then, the lining of canal is justified, if $B > C$.

7.16 ECONOMICS OF CANAL LINING

To recommend the lining in canal, it is necessary to ascertain that the total annual cost incurred for lining can be recovered within some specified period or during the life time of the project. So, the annual cost of lining and the annual benefits are to be worked out separately in details. If the value of annual benefits exceed the annual cost incurred, then the lining should be considered economical. The following procedure is adopted to determine the annual cost and benefits.

1. Determination of Annual Cost

(a) Annual depreciation charge $= \dfrac{\text{Initial cost}}{\text{Useful life}} = X$ (say)

(b) Average annual interest $= \dfrac{\text{Initial cost}}{2} \times$ Rate of interest $= Y$ (say)

Average annual cost of lining $= (X + Y)$ (1)

2. Determination of Annual Benefits

(a) Saving by eliminating seepage loss $= S_1$ (say)
(b) Saving in maintenance works $= S_2$ (Say)
(c) Other benefits (if any) $= S_3$ (Say)

Total benefit $= (S_1 + S_2 + S_3)$ (2)

If the total benefit $(S_1 + S_2 + S_3)$ is found greater than the total cost $(X + Y)$, then the implementation of lining in canal may be considered as economical.

Example A canal of length 5 km and of discharge capacity 3.5 cumec is proposed to be lined with boulder lining. The total cost of lining is estimated as 4 lakhs. The life of the lining is considered as 60 years. Justify the lining in the canal from the following data,

(a) Rate of interest = 8%
(b) Seepage loss = 2%
(c) Revenue for irrigation water = Rs 75.00 per hect-m.
(d) Maintenance cost per km for lined final = Rs 1000.00
(e) Maintenance cost per km for unlined canal = Rs 2500.00
(f) Base period of crop = 120 days
(g) Additional benefit/km = Rs 1000.00

Solution The annual cost and benefit are ascertained per km.

1. Determination of Annual Cost Per km

(a) Annual depreciation charge/km $= \dfrac{4,00,000}{60 \times 5} =$ Rs 1333.00

(b) Average annual interest/km $= \dfrac{4,00,000}{2 \times 5} \times 0.08 =$ Rs 3200.00

Total annual cost/km = 1333.33 + 3200.00 = Rs 4533.33

2. Determination Annual Benefit Per km

1 cumec-day = 8.84 ha–m.

3.5 cumec for 120 days $= 8.84 \times 3.5 \times 120$

= 3628.8 ha–m.

∴ Total flow of water = 3628.8 ha–m.

So, saving in seepage loss by providing

lining $= 3628.8 \times .02 = 72.576$ ha – m.

(a) Amount of saving $= 72.576 \times 75 =$ Rs 5443.20
(b) Saving in maintenance works = 2500 – 1000 = Rs 1500.00
(c) Additional benefit = Rs 1000.00

Total benefit = 5443.20 + 1500.00 + 1000.00 = 7943.20

Therefore, annual saving = 7943.20 – 4533.33 = Rs 3409.87

It is seen that the saving is quite considerable. So, the lining of the canal may be considered as economical.

REVIEW QUESTIONS

1. Fill up the blanks with appropriate word/words:
   (i) Lining of canal is done to control the _________ of water.

(ii) Lining of canal is done to prevent the _________.
(iii) Lining of canal is done to increase the _________ of canal.
(iv) The growth of _________ are controlled by providing lining in canal.
(v) The lining is known as _________ lining when the cement mortar is applied on sub-grade by cement gun.
(vi) For small channels _________ cement lining is recommended.

2. Why should lining be provided in canals?
3. Describe the following with neat sketch,
   (a) Cement concrete lining
   (b) Pre-cast lining
   (c) Brick lining
   (d) Boulder lining.
4. What are the merits and demerits of canal lining?
5. Explain the economics of canal lining.

Answers

1. (i) seepage
   (ii) waterlogging
   (iii) capacity
   (iv) weeds
   (v) shot-crete
   (vi) soil

8

DRAINAGE CHANNEL DESIGN

8.1 INTRODUCTION

In rainy season, the water logging is the main problem in rural and urban areas. If proper drainage system is not provided, then it will pose problems to the people of those areas. To implement a proper drainage system, the following steps should be taken,

(a) The watershed area is to be demarcated and the total area should be ascertained from the contour map.

(b) The water logged area is to be demarcated and the area should be calculated from the contour map.

(c) The volume of water collected in the water logged area is calculated by studying the contour map.

(d) If rainfall records are available, the run-off may be calculated directly.

(e) If rainfall records are not available, the intensity of rainfall is calculated from storm duration and hence the run-off may be calculated.

After collecting the above informations the drainage channel may be designed according to the desired conditions. The method of collection of data and the procedure for design are enumerated step-by-step in the following sections.

8.2 VALUE OF RUN-OFF COEFFICIENT (K)

Nature of surface	*Value of K*
1. Urban Area:	
(a) 30% are impervious	0.40 – 0.50
(b) 50% area impervious	0.55 – 0.65
(c) 70% area impervious	0.65 – 0.80
(d) Residential area	0.30 – 0.40
2. Rural Area:	
(a) Cultivated area	0.30 – 0.60
(b) Rough pasture	0.10 – 0.40
(c) Forest area	0.10 – 0.40
(d) Area covered by huts	0.20 – 0.30

8.3 VALUE OF IMPERMEABILITY FACTOR (p)

Surface	*Value of p*
(1) Residential area with many buildings	0.70 – 0.90
(2) Residential area with detached houses	0.25 – 0.50
(3) Areas with few buildings	0.20 – 0.25
(4) Cultivated areas	0.15 – 0.20
(5) Forest areas	0.10 – 0.20
(6) Rough pasture and barren lands	0.10 – 0.20
(7) Gardens, lawns, parks etc	0.05 – 0.10

8.4 DETERMINATION OF RUN-OFF COEFFICIENT (K)

When the run-off coefficient is not given, but the nature of the surface of different areas and the impermeability factors of the areas are known, then the value of K may be determined by the following expression,

$$K = \frac{a_1 p_1 + a_2 p_2 + \dots}{A}$$

where, a_1, a_2 = Area of different surfaces, p_1, p_2 = Impermeability factors of those surfaces, A = Total surface area.

Example A catchment area consists of the following surfaces,

1. Cultivated area = 50 hectares ($p = 0.20$)
2. Forest area = 30 hectares ($p = 0.10$)
3. Garden = 5 hectares ($p = 0.05$)
4. Residential area = 15 hectares ($p = 0.50$)

Find the run-off coefficient of that catchment area.

Solution Here, total area = 50 + 30 + 5 + 15 = 100 hectares

From, the expression, $K = \frac{a_1 p_1 + a_2 p_2 \dots}{A}$

We get run off coefficient, $K = \frac{50 \times 0.20 + 30 \times 0.10 + 5 \times 0.05 + 15 \times 0.50}{100}$

$$= \frac{20.75}{100} = 0.2075$$

8.5 DETERMINATION OF RAINFALL INTENSITY (*i*)

When rainfall records are not available, the intensity of rainfall is ascertained by the following empirical formulae,

(a) $i = \frac{672}{t + 10}$ (For storm duration 5 to 20 mins)

(b) $i = \frac{1020}{t + 10}$ (For storm duration 20 to 100 mins)

(c) $i = \dfrac{3430}{t + 18}$ (where rainfall is frequent).

Here, i = intensity of rainfall in mm/hr, t = duration of storm in mins.

Example 1 Find the intensity of rainfall when the storm continues for a period of 90 mins.

Solution Using the formula,

$$i = \frac{1020}{t + 10}$$

Intensity of rainfall, $i = \dfrac{1020}{90 + 10} = \dfrac{1020}{100} = 10.20$ mm/hr.

Example 2 Find the intensity of rainfall when the rainfall occurs frequently and the total duration of rainfall is 110 mins.

Solution Using the formula,

$$i = \frac{3430}{t + 18}$$

Intensity of rainfall, $i = \dfrac{3430}{110 + 18} + \dfrac{3430}{128} = 26.79$ mm/hr.

8.6 ESTIMATION OF RUN-OFF BY RATIONAL METHOD

In this method, the run off is given by the following expression,

(a) $Q = \dfrac{K\,i\,A}{36}$

where, Q = run-off in cumec, K = coefficient of run-off or impermeability factor, i = intensity of rainfall in cm/hr, A = catchment area in hectares, or

(b) $Q = \dfrac{K\,i\,A}{360}$ where, Q, K & A = As before, i = Intensity of rainfall in mm/hr.

Example A catchment area of 20 sq km consists of two-third rural area and one-third urban area. The rainfall in the whole catchment area is recorded as 25 mm/hr. Find the total run-off from the catchment area.

Solution Total area = 20 sq km = 20 × 1000 × 1000 sq m = 2000 hectares (1 hectare = 10,000 m^2)

Now, Rural area = 2000 × 2/3 = 1333.33 hectares.

Urban area = 2000 × 1/3 = 606.67 hectares.

Rainfall intensity, i = 25 mm/hr = 2.5 cm/hr.
Assuming, run off coefficient (K)
for rural area = 0.30, and for urban area = 0.50.

the run-off by rational method is given by, $Q = \dfrac{K\,i\,A}{36}$

$$\text{Run-off } Q = \frac{0.30 \times 2.5 \times 1333.33 + 0.50 \times 2.5 \times 666.67}{36}$$

$$= \frac{999.99 + 833.33}{36} = 50.925 \text{ cumec}$$

8.7 DETERMINATION OF RUN-OFF BY EMPIRICAL FORMULAE

The following empirical formulae are generally adopted for estimating the run-off from a catchment area.

(a) Dicken's formula

$$Q = C \times A^{3/4}$$

where, Q = Run-off in cumec, C = a coefficient varies from 11.5 to 25, A = Catchment area in sq.km.

(b) Ryve's formula

$Q = C \times A^{2/3}$ where, Q = Run-off in cumec, C = a coefficient varies from 6.75 to 10.00, A = Catchment area in sq.km.

(c) Inglish formula

$$Q = \frac{123\,A}{\sqrt{A + 10.4}}$$

where, Q = Run-off in cumec, A = catchment area in sq.km.

8.8 COMPUTATION OF VOLUME OF WATER COLLECTED IN A WATER LOGGED AREA

When rainfall records are not available, the volume of water collector in a water logged area is found out by the following methods.

(a) From contour map If a contour map is not available for the affected area then it needs to be prepared by Contour Survey. The area enclosed by each contour is determined by planimeter. Then, by applying Prismoidal or Trapezoidal rule the volume of water is calculated.

(b) From cross-sections The affected area is enclosed by a number of cross-sections at regular intervals. The cross-sectional levelling is done and the cross-sections are plotted according to any suitable scale. The highest water level that attained in the last ten years is marked on the cross-sections. The areas of the probable water sections are calculated. Then by applying Prismoidal rule or Trapezoidal rule the volume of water is ascertained.

8.9 CHANNEL DESIGN FORMULAE

The mean velocity of flow is determined by the following formulae.

1. Mean Velocity by Chezy's Formula

Mean velocity, $V = C\sqrt{R \cdot S}$

where, V = mean velocity in m/sec, R = hydraulic mean depth in m, S = longitudinal slope or hydraulic gradient, C = Chezy's constant.

The Chezy's constant (C) may be calculated by Bazin's formula or Kutter's formula.

(a) By Bazin's formula

$$C = \frac{87}{1 + \frac{K}{\sqrt{R}}}$$

where,

K = Bazin's constant, R = as stated earlier

Nature of surface	*Value of K*
Smooth surface	0.10
Brick lining surface	0.29
Rubble masonry surface	0.80
Earthen channel	1.30 to 1.50

(b) By Kutter's formula

$$C = \frac{23 + \frac{1}{N} + \frac{0.00155}{S}}{1 + \left(23 + \frac{0.00155}{S}\right) \times \frac{N}{\sqrt{R}}}$$

where, N = Kutter's constant or roughness coefficient or rugosity factor, R and S = as stated earlier.

Nature of surface	*Value of N*
Smooth surface	0.01
Brick masonry or stone masonry lining	0.017
Earthen channel	0.02 to 0.025

2. Mean Velocity by Manning's Formula

$$V = \frac{1}{N} \times R^{2/3} \times S^{1/2}$$

where, V = mean velocity in m/sec, R, N, and S = as stated earlier.

8.10 PROBLEMS ON DRAINAGE CHANNEL DESIGN

Problem 1 A drainage channel which outfalls into a tidal river is to be designed with the following field data,

(i) Catchment area = 15 sq.km out of which 10 sq.km is rural and 5 sq.km is semiurban.
(ii) Storm duration = 60 mins.
(iii) Tide lockage period = 8 hrs.
(iv) Coefficient of rugosity (N) = 0.025
(v) Available longitudinal slope = 15 cm per km.

Design the channel, assuming reasonable data, if required.

Solution

Intensity of rainfall is found out from expression 9.5 (b).

$$i = \frac{1020}{t+10} \text{ here, } t = 60 \text{ mins.}$$

$$\text{Intensity of rainfall, } i = \frac{1020}{60+10} = 14.5 \text{ mm/hr,}$$

$$= 15 \text{ mm/hr (say)}, = 1.5 \text{ cm/hr.}$$

$$\text{Rural area } (A_1) = 10 \text{ sq. km} = \frac{10 \times 1000 \times 1000}{10{,}000}$$

$$= 1000 \text{ hectares } (1 \text{ ha} = 10{,}000 \text{ m}^2)$$

$$\text{Semi-urban area, } (A_2) = 5 \text{ sq. km} = \frac{5 \times 1000 \times 1000}{10{,}000}$$

$$= 500 \text{ hectares.}$$

Let us assume, run-off coefficient (K_1) for rural area = 0.30 and run-off coefficient (K_2) for semi-urban area = 0.50.

By rational formula, the run-off is given by,

$$Q = \frac{K\,i\,A}{36} \text{ cumec.}$$

So,

$$Q = \frac{K_1\,i\,A_1}{36} + \frac{K_2\,i\,A_2}{36}$$

or

$$Q = \frac{0.30 \times 1.5 \times 1000}{36} + \frac{0.50 \times 1.5 \times 500}{36}$$

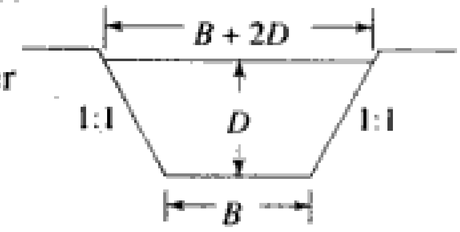
$$= 12.50 + 10.42 = 22.92 \text{ cumec}$$

So, the volume of water accumulated in 24 hrs = 22.92 × 24 × 60 × 60 = 1980288 m^3.

As the tide lockage period is 8 hrs. This volume of water should be drained-off in 16 hrs.

Therefore, the actual discharge of the drainage channel is equal to

$$Q = \frac{1980288}{16 \times 60 \times 60} = 34.38 \text{ cumec}$$



Let, B = Bed width of channel, D = Depth of water and side slope = 1:1 (assumed)

Cross-sectional area, $A = \frac{B + B + 2D}{2} \times D$

$$= (B + D) \times D \quad \text{(i)}$$

Wetted perimeter, $P_w = B + 2 \times \sqrt{2}\ D$ (ii)

Hydraulic mean depth, $R = \frac{A}{P_w} = \frac{(B + D)\ D}{B + 2\sqrt{2}\ D}$ (iii)

We know, for economical section,

$$R = D/2 \quad \text{(iv)}$$

From Eqs (iii) and (iv) $D/2 = \frac{(B + D)\ D}{B + 2\sqrt{2}\ D}$

or $2\ B\ D + 2\ D^2 = B\ D + 2\ \sqrt{2}\ D^2$

or $BD = D^2\left(2\sqrt{2} - 2\right)$

or $B = 0.828\ D$ (v)

We know, $Q = A \times V$

where, $V = \frac{1}{N} \times R^{2/3} \times S^{1/2}$

Here, $N = 0.025,\ S = \frac{15}{1000 \times 100} = \frac{1}{6666.67}$

So, $$34.38 = (B + D)\ D \times \frac{1}{0.025} \times \left\{\frac{(B + D)\ D}{B + 2\sqrt{2}D}\right\}^{2/3} \times \left(\frac{1}{6666.67}\right)^{1/2}$$

$$= \frac{1.828\ D^2}{0.025} \times \left\{\frac{1.828\ D^2}{3.656\ D}\right\}^{2/3} \times 0.0122$$

$$= \frac{1.828\ D^2 \times (1.828D)^{2/3} \times 0.0122}{0.025 \times (3.656)^{2/3}}$$

or $$34.38 = \frac{D^{8/3} \times 2.733 \times 0.0122}{0.025 \times 2.373} = 0.562\ D^{8/3}$$

$$D^{8/3} = 61.17$$

$$D = 4.68 \text{ m}$$

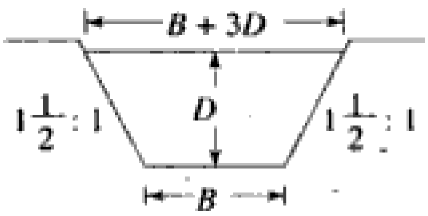
$\therefore$ $B = 0.828 \times 4.68 = 3.875$ m

So, the bed width of channel is 3.875 m and depth of water is 4.68 m.

Problem 2 An area is waterlogged between August and October when rainfall occurs 150 mm/day. The total water logged area is 600 hectares. Design a drainage channel to remove the drainage congestion with the following data

(a) Run-off coefficient = 0.60
(b) Velocity of flow not to exceed 0.75 m/sec
(c) Side slope = $1^{1/2} : 1$

Assume reasonable data, if required.



Solution

Drainage area, $A = 600$ hectares

Intensity of rainfall, $i = 150$ mm/day $= 0.625$ cm/hr

Run-off coefficient $= 0.60$

$$Q = \frac{K\,i\,A}{36} = \frac{0.60 \times 0.625 \times 600}{36} = 6.25 \text{ cumec}$$

We know, $Q = A \times V$

$$A = \frac{Q}{V} = \frac{6.25}{0.75} = 8.33 \text{ m}^2 \quad (1)$$

Let, D = Depth of water, B = Bed width of channel

Cross-sectional A area,
$$A = \frac{B + B + 3D}{2} \times D = \frac{(2B + 3D)\,D}{2} \quad (2)$$

Wetted perimeter,
$$P_w = B + 2\sqrt{D^2 + (1.5D)^2} = B + 2\,D\,\sqrt{3.25}$$
$$= B + 3.6\,D \quad (3)$$

Hydraulic mean depth,
$$R = \frac{A}{P_w}$$
$$R = \frac{(2B + 3D)\,D}{2\,(B + 3.6\,D)} \quad (4)$$

Again for economical section
$$R = D/2 \quad (5)$$

From (4) and (5)
$$D/2 = \frac{(2B + 3D)\,D}{2\,(B + 3.6D)}$$

or $4\,B + 6\,D = 2\,B + 7.2\,D$ or $2\,B = 1.2\,D$

or
$$B = 0.6\,D \quad (6)$$

Equating (1) and (2), we get
$$8.33 = \frac{(2B + 3D)\,D}{2} \quad (7)$$

Putting the value of B in Eq. (7)

$$8.33 = \frac{(2 \times 0.6D + 3D)\,D}{2}$$

or $$16.66 = 4.2\,D^2$$

$$D = 1.99 \text{ m} = 2.00 \text{ m (say)}$$

$$B = 0.6\,D = 0.6 \times 2 = 1.2 \text{ m}$$

So, the bed width of the channel is 1.2 m and depth of water is 2.0 m.

Problem 3 From the contour map of a water logged area, the following data was recorded.

Contour (m)	198.0	198.5	199.0	199.5	200.0
Area (m^2)	1750	2200	15900	19000	21000

Design a rectangular channel to drain out the whole water collected in the above area in 2 hrs. Assume longitudinal slope as 1 in 4000, roughness coefficient (N) as 0.01 and the velocity of flow not to exceed 1m/sec.

Assume necessary data.

Solution Volume of water collected in the area,
By Prismoidal formula,

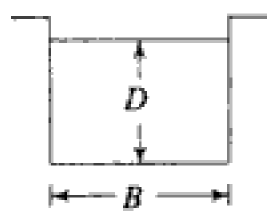
$$\text{Volume} = \frac{0.5}{3}\,\{1750 + 21{,}000 + 2\,(15{,}900) + 4\,(2200 + 19000)\}$$

$$= \frac{0.5}{3}\,\{22750 + 31800 + 84800\} = 23225 \text{ m}^3$$

This volume of water is to be drained off in 2 hr.

∴ Discharge of the channel, $$Q = \frac{23225}{2 \times 60 \times 60} = 3.225 \text{ cumec}$$

Let, B = bed width of channel,
D = depth of water in channel.



Here, $$A = \frac{Q}{V} = \frac{3.225}{1} = 3.225 \text{ m}^2$$

Cross-sectional Area $= B \times D$, ∴ $B \times D = 3.225$ (1)

Wetted perimeter, $P_w = B + 2\,D$ (2)

Hydraulic mean depth, $$R = \frac{BD}{B + 2D}$$

or $$R = \frac{3.225}{B + 2D} \qquad (3)$$

From manning's formula,

$$V = \frac{1}{N} \times R^{2/3} \times S^{1/2}$$

$$I = \frac{1}{0.01} \times R^{2/3} \times \left(\frac{1}{4000}\right)^{1/2}$$

Here, $V = 1\text{m/sec}, S = \frac{1}{4000}$

or $1 = 100 \times R^{2/3} \times 0.0158$

or $1 = 1.58\ R^{2/3}$

$R^{2/3} = 0.6329$

or $R = 0.50$ (4)

From Eqs (3) and (4)

$$0.50 = \frac{3.225}{B + 2D}$$

or $0.50\ B + D = 3.225$

or $D = 3.225 - 0.50\ B$ (5)

Putting the value of D in (1)

$$B\ (3.225 - 0.50\ B) = 3.225$$

or $0.50\ B^2 - 3.225\ B + 3.225 = 0$

$$B = \frac{3.225 \pm (3.225)^2 - 4 \times 0.5 \times 3.225}{2 \times 0.5}$$

$$= \frac{3.225 \pm 1.987}{1} = 5.212 \text{ m} \quad \text{or} \quad 1.238 \text{ m}$$

1st case

When, $B = 5.212 \text{ m}, D = \frac{3.225}{1.212} = 0.618 \text{ m}$ (Abnormally low)

2nd case

When, $B = 1.238 \text{ m}, D = \frac{3.225}{1.238} = 2.61 \text{ m}$ (may be accepted)

In 1st case, the depth is abnormally low and the land width will be more, so it may be avoided.

In 2nd case, the land width and the depth are reasonable, so it may be accepted. That is, Bed width = 1.238 m and depth = 2.61 m.

Review Questions

1. What is the difference between drainage channel and irrigation canal?
2. How can run-off coefficient (K) be determined?
3. How will you ascertain rainfall, when records are not available?
4. Explain the methods of determining the volume of water collected in a waterlogged area.

5. Design a drainage channel to clear the rain water within 12 hrs. from a basin having the following data
   (a) Basin area = 10 sq. km (rural area)
   (b) Intensity of rainfall = 15 mm/hr.
   (c) Coefficient of rugosity (N) = 0.025.
   (d) Longitudinal slope = 1 in 5000.

   Assume reasonable data, if required.
6. An agricultural land of area 500 hectares is found to be waterlogged during rainy season and the crops get spoiled every year. Design a suitable channel to clear off the area having the following informations,
   (a) Maximum rainfall during rainy season was recorded as 250 mm/hr.
   (b) Allowable side slope of channel – $1^{1/2}$: 1.
   (c) Permissible velocity = 1.0 m/sec.

   Assume reasonable data, if required.

9

DESIGN OF IRRIGATION CANAL

9.1 INTRODUCTION

The success of the flow irrigation depends on the perfect design of the network of canals. The canals may be excavated through the different types of soils such as alluvial soil, non-alluvial soil, etc. The design considerations naturally vary according to the type of soil. Again, the velocity of flow in the canal should be critical. That means, the velocity should be non-silting and non-scouring. If the velocity becomes less than the critical velocity, then silting will take place and the capacity of the canal will be reduced. If the velocity becomes more than the critical velocity then scouring will take place and the canal will be damaged. So, the determination of critical velocity is very important in canal design.

After long research in different canals and different conditions R.G. Kennedy, Executive Engineer, Punjab and Gerald Lacey, Chief Engineer, U.P. have established some theories for the design of canals which are known as 'Kennedy's theory' and 'Lacey's theory'. These two theories are based on the characteristics of sediment load (i.e. silt) in canal water. The behaviour of the silt is explained by the theory which is known as 'silt theory'.

Again, the design of unlined and lined canals involves different practical and economical considerations which will be studied in this chapter.

9.2 DEFINITION OF DIFFERENT TERMS RELATED TO CANAL DESIGN

(i) Alluvial Soil The soil which is formed by the continuous deposition of silt is known as alluvial soil. The river carries heavy charge of silt in rainy season. When the river overflows its banks during the flood, the silt particles get deposited on the adjoining areas. This deposition of silt continues year after year. This type of soil is found in deltaic region of a river. This soil is permeable and soft and very fertile. The river passing through this type of soil has a tendency to change its course.

(2) Non-Alluvial Soil The soil which is formed by the disintegration of rock formations is known as non-alluvial soil. It is found in the mountainous region of a river. The soil is hard and impermeable in nature. This is not fertile. The river passing through this type of soil has no tendency to change its course.

(3) Silt Factor (f) If designing of a canal in alluvial soil, the suspended silt and the deposited silt in the canal bed should be taken into consideration with great importance. During the investigation works in various canals in alluvial soil, Gerald Lacey established the effect of silt on the determination of discharge and the canal section. So, he introduced a factor which is known as 'silt factor'. It depends on the mean particle size of silt. It is denoted by 'f'. The silt factor is determined by the expression,

$$f = 1.76\sqrt{m_r}$$

where, m_r = mean particle size of silt in mm.

Particle	*Particle size (in mm)*	*Silt factor (f)*
Very fine silt	0.05	0.40
Fine silt	0.12	0.60
Medium silt	0.23	0.85
Coarse silt	0.32	1.00

(4) Coefficient of Rugosity (N) The roughness of the canal bed affects the velocity of flow. The roughness is caused due to the ripples formed on the bed of the canal. So, a coefficients was introduced by R.G. Kennedy for calculating the mean velocity of flow. This coefficient is known as coefficient of rugosity and it is denoted by 'N'. The value of 'N' depends on the type of bed materials of the canal.

Materials	*Value of N*
Earth	0.0225
Masonry	0.02
Concrete	0.013 to 0.018

(5) Mean Velocity (V) The velocity of flow measured by surface flow is known as surface velocity. This velocity is not constant at all depths of water in the canal or river. It is found by observation that the velocity at a depth 0.6 D represents the mean velocity, where 'D' is the depth of water in the canal or river. After along investigations in various canals, Chezy and Manning have established the following expressions for finding the mean velocity of flow.

(a) Mean Velocity by Chezy's expression

$$V = C \times \sqrt{RS}$$

where, V = mean velocity in m/sec, C = chezy's constant, R = hydraulic mean depth, S = longitudinal slope of bed (as 1 in n).

(b) Mean Velocity by Manning's expression

$$V = \frac{1}{N} \times R^{2/3} \times S^{1/2}$$

where, N = coefficient of rugosity, V, R and S = as stated earlier.

(6) Critical Velocity (V_0) When the velocity of flow is such that there is no silting or scouring action in the canal bed, then that velocity is known as critical velocity. It is denoted by 'V_0'. The value of V_0 was given by Kennedy according to the following expression, $V_0 = 0.546 \times D^{0.64}$ where, D = Depth of water

(7) Critical Velocity Ratio (C.V.R.) The ratio of the mean velocity 'V' to the critical velocity 'V_0' is known as critical velocity ratio. It is denoted by m.

i.e. $$\text{C.V.R.} = \frac{V}{V_0} \quad \text{or} \quad m = \frac{V}{V_0}$$

When $m = 1$ there will be no silting or scouring, When $m > 1$, scouring will occur. When $m < 1$, silting will occur.

So, by finding the value of m, the condition of the canal can be predicted whether it will have silting or scouring.

(8) Regime Channel When the character of the bed and bank materials of the channel are same as that of the transported materials and when the silt charge and silt grade are constant, then the channel is said to be in its regime and the channel is called regime channel. This ideal condition is not practically possible.

(9) Hydraulic mean depth or radius The ratio of the cross-sectional area of flow to the wetted perimeter of the channel is known as hydraulic mean depth, or radius. It is generally denoted by R.

So, $$R = \frac{A}{P_w}$$

where, A = cross-sectional area of flow, P_w = wetted perimeter.

(10) Full Supply Discharge (F.S.D.) The maximum discharge capacity of the canal for which it is designed, is known as full supply discharge. The water level of the canal corresponding to the full supply discharge is known as full supply level (F.S.L).

(11) Economical Section If a canal section is such that the earth obtained from cutting (i.e. excavation) can be fully utilised in forming the banks, then that section is known as economical section. Again, the discharge will be maximum with minimum cross-section area. Here, no extra earth is required from borrowpit and no earth is in excess to form the spoil bank. This condition can only arise in case of partial cutting and partial banking. Sometimes, this condition is designated as balancing of cutting and banking. Here, the depth of cutting is called balancing depth.

9.3 UNLINED CANAL DESIGN ON NON-ALLUVIAL SOIL

The non-alluvial soils are stable and nearly impervious. For the design of canal in this type of soil, the coefficient of rugosity plays an important role, but the other factor like silt factor has no role. Here, the velocity of flow is considered very close to critical velocity. So, the mean velocity given by Chezy's expression or Manning's expression is considered for the design of canal in this soil. The following formulae are adopted for the design.

(1) Mean Velocity by Chezy's Formula

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

where, V = mean velocity in m/sec, C = chezy's constant, R = hydraulic mean depth in m, S = bed slope of canal as 1 in n.

Again, the Chezy's constant C can be calculated by.

(a) Bazin's Formula

$$C = \frac{87}{1 + \frac{K}{\sqrt{R}}}$$

where, K = Bazin's constant, R = as stated earlier.

(b) Kutter's Formula

$$C = \frac{23 + \frac{0.00155}{S} + \frac{1}{N}}{1 + \left(23 + \frac{0.00155}{S}\right) \times \frac{N}{\sqrt{R}}}$$

where, N = Coefficient of rugosity, S and R = as stated earlier.

(2) Mean Velocity by Manning's Formula

$$V = \frac{1}{N} \times R^{2/3} \times S^{1/2}$$

where, V, N, R and S = as stated earlier.

(3) Discharge by Following Relation

$$Q = A \times V,$$

where, Q = discharge in cumec, A = cross-sectional area of water section in m^2, V = mean velocity in m/sec.

Note

(a) If value of K is not given, then it may be assumed as follows, for unlined channel, K = 1.30 to 1.75,
For lined channel, K = 0.45 to 0.85.

(b) If the value of N is not given, then it may be assumed as follows,
For unlined channel, N = 0.0225, For lined channel, N = 0.333.

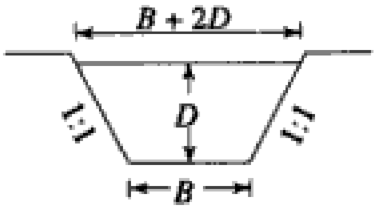
Problem 1 Design an irrigation canal with the following data

(a) Discharge of the canal = 24 cumec.
(b) Permissible mean velocity = 0.80 m/sec.
(c) Bed slope = 1 in 5000.
(d) Side slope = 1:1.
(e) Chezy's constant, $C = 44$.

Solution Given data, $Q = 24$ cumec, $V = 0.80$ m/sec, $S = \dfrac{1}{5000} = 0.0002$, Side slope = 1:1.

Let, B = bed width, D = depth of water.

Cross-sectional area, $A = (B + D)D$.



Again, $$A = \frac{Q}{V} = \frac{24}{0.8} = 30 \text{ m}^2$$

$$30 = (B + D)\, D \quad (1)$$

Wetted perimeter, $P_w = B + 2\sqrt{2}\, D = B + 2.828\, D$

Hydraulic mean depth, $$R = \frac{A}{P_w} = \frac{30}{B + 2.828\, D} \quad (2)$$

From Chezy's formula,

$$V = C \times \sqrt{RS}$$

$$0.80 = 44\, \sqrt{R \times 0.0002}$$

or $$0.64 = 1936 \times R \times 0.0002$$

$$R = 1.65 \quad (3)$$

From (2) and (3), $$1.65 = \frac{30}{B + 2.828\, D}$$

or $1.65\, B + 4.67\, D = 30$

or $$B = 18.18 - 2.83\, D \quad (4)$$

putting the value of B in Eq. (1)

$$30 = (18.18 - 2.83\, D + D)\, D$$
$$= (18.18 - 1.83\, D)D$$
$$= 18.18\, D - 1.83\, D^2$$

or $$1.83\, D^2 - 18.18\, D + 30 = 0$$

or $$D = \frac{18.18 \pm \sqrt{(18.18)^2 - 4 \times 1.83 \times 30}}{2 \times 1.83}$$

$$= \frac{18.18 \pm 10.53}{3.66} = 7.84 \text{ or } 2.09 \text{ m}$$

When $D = 7.84$ m

$$B = 18.18 - 2.83 \times 7.84$$
$$= -4.00 \text{ (It is absurb)}$$

When, $D = 2.09$ m

$B = 18.18 - 2.83 \times 2.09$

$= 12.27$ m (it is acceptable)

Check

$A = (B + D)\,D$

$= (12.27 + 2.09) \times 2.09$

$= 30.01$ (Checked and found correct)

So, finally, bed width = 12.27 m, depth of water = 2.09 m.

Problem 2 Design a most economical trapezoidal section of a canal having the following data,

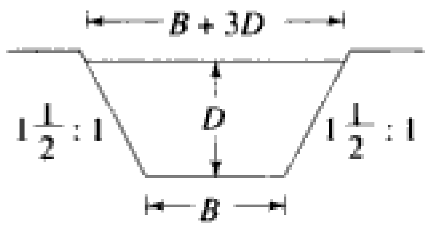
(a) Discharge of the canal = 20 cumec.

(b) Permissible mean velocity = 0.85 m/sec.

(c) Bazim's constant, $K = 1.30$.

(d) Side slope of canal = $1\frac{1}{2}$:1.

Find also the allowable bed slope of the canal.



Solution

Let, B = bed width, D = depth of water

Cross-sectional area, $$A = \frac{B + B + 3D}{2} \times D$$

$$= (B + 1.5D)D \qquad (1)$$

Wetted perimeter, $$P_w = B + 2\sqrt{D^2 + (1.5\,D)^2} = B + 3.6\,0D \qquad (2)$$

Hydraulic mean depth, $$R = \frac{A}{P_w} = \frac{(B + 1.5\,D)\,D}{B + 3.60\,D} \qquad (3)$$

Again, we know that for economical section,

$$R = D/2 \qquad (4)$$

So, from (3) and (4)

$$D/2 = \frac{(B + 1.5\,D)\,D}{B + 3.60\,D}$$

or $$2\,BD + 3D^2 = BD + 3.60\,D^2$$

$$B = 0.60\,D \qquad (5)$$

Again, from, $Q = A \times V$

$$A = \frac{Q}{V} = \frac{20}{0.85} = 23.53 \text{ m}^2.$$

So, from (1), $$23.53 = (B - 1.5\,D)\,D \qquad (6)$$

or $$23.53 = (0.60\,D + 1.5\,D)\,D$$

[Putting the value of B in Eq. (6)]

or $D = 3.35$ m

From Eq. (5),

$B = 0.60 \times 3.35 = 2.01$ m

Now, $A = 23.53$ m^2 (obtained previously)

$P_w = B + 3.60\,D = 2.01 + 3.60 \times 3.35 = 14.07$ m

$$R = \frac{23.53}{14.07} = 1.67 \text{ m}$$

By Bazin's formula,
$$C = \frac{87}{1 + \frac{K}{\sqrt{R}}} = \frac{87}{1 + \frac{1.30}{\sqrt{1.67}}} = 43.5$$

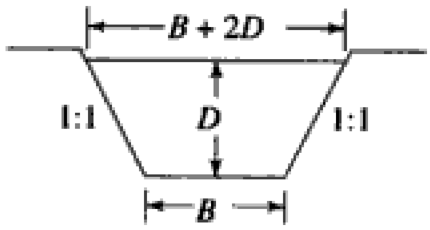
From Chezy's formula,
$$V = C \times \sqrt{RS}$$
$$0.85 = 43.5\sqrt{1.67 \times S}$$

or
$$S = \frac{(0.85)^2}{(43.5)^2 \times 1.67} = \frac{1}{4373.78} = \frac{1}{4374} \text{ (say)}$$

So, bed width = 2.01 m, depth of water = 3.35 m, longitudinal slope = $\frac{1}{4374}$.

Problem 3 Find the bed width and bed slope of a canal with the following data,

(a) Discharge of canal = 40 cumec
(b) Permissible mean velocity = 0.95 m/sec
(c) Coefficient of rugosity (N) = 0.0225
(d) Side slope = 1:1
(e) B/D ratio = 6.5



Solution

Let, B = bed width, D = depth of water

Cross-sectional Area,
$$A = (B + D) \times D \quad (1)$$

Wetted perimeter,
$$P_w = B + 2\sqrt{2}\,D \quad (2)$$

Now,
$$A = \frac{Q}{V} = \frac{40}{0.95} = 42.11 \text{ m}^2$$

$$B/D = 6.5 \text{ (given)}$$
$$B = 6.5\,D \quad (3)$$

From Eq. (1) $42.11 = (6.5D + D)\,D$ [Putting the value of B in Eq. (1)]

or $D = 2.37$ m

$B = 6.5 \times 2.37 = 15.40$ m

From Eq. (2) $P_w = 15.40 + 2\sqrt{2} \times 2.37 = 22.10$ m.

Hydraulic mean depth,
$$R = \frac{A}{P_w} = \frac{42.11}{22.20} = 1.90 \text{ m}$$

From Manning's formula,
$$V = \frac{1}{N} \times R^{2/3} \times S^{1/2}$$
$$0.95 = \frac{1}{0.0225} \times (1.90)^{2/3} \times S^{1/2}$$

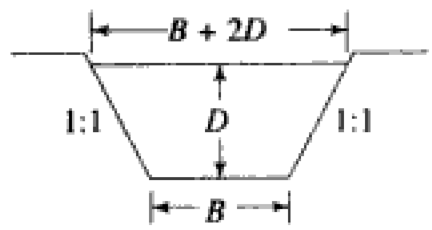
or $$0.95 = 44.44 \times 1.534 \times S^{1/2}$$
$$S^{1/2} = 0.01394$$
or $$S = 0.000194$$
or $$S = \frac{1}{5154.6} = \frac{1}{5155} \text{ (say)}$$

So, Bed width of canal = 15.40 m.

$$\text{Bed slope} = \frac{1}{5155}$$

Problem 4 Find the efficient cross-section of a canal having the discharge 10 cumec. Assume, bed slope 1 in 5000, value of $N = 0.0025$ C.V.R. (m) = 1, full supply depth not to exceed 1.60 m and side slope = 1:1.

Solution



Let, B = bed width, D = depth of water

Cross-sectional area, $A = (B + D)D$ (1)

Wetted perimeter, $P_w = B + 2\sqrt{2}\,D$ (2)

Taking $D = 1.60$ m

We know, Critical velocity, $V_0 = 0.546 \times m \times D^{0.64}$

or $V_0 = 0.546 \times 1 \times 1.60^{0.64} = 0.737$ m/sec.

Since, C.V.R. = 1.

Mean velocity is taken as 0.737 m/sec (i.e. $V = V_0$)

$$\text{c/s area } A = \frac{Q}{V} = \frac{10}{0.737} = 13.56 \text{ m}^2$$

From Eq. (1)

$$13.56 = (B + 1.60) \times 1.60 = 1.60\,B + 2.56$$
$$B = 6.88 \text{ m}$$

From Eq. (2) $$P_w = 6.88 + 2\sqrt{2} \times 1.60 = 11.41 \text{ m}$$

$$\text{Hydraulic mean depth, } R = \frac{13.56}{11.41} = 1.19 \text{ m.}$$

To find the mean velocity by Chezy's formula, the value of C is calculated by Kutter's formula.

$$C = \frac{23 + \dfrac{0.00155}{S} + \dfrac{1}{N}}{1 + \left(23 + \dfrac{0.00155}{S}\right) \times \dfrac{N}{\sqrt{R}}}$$

$$= \frac{23 + \dfrac{0.00155}{0.0002} + \dfrac{1}{0.0225}}{1 + \left(23 + \dfrac{0.00155}{0.0002}\right) \times \dfrac{0.0225}{\sqrt{1.19}}} \qquad \left(S = \frac{1}{5000} = 0.0002\right)$$

$$= \frac{23 + 7.75 + 44.44}{1 + 30.75 \times 0.02} = \frac{75.19}{1.615} = 46.57$$

Mean velocity, $V = C \times \sqrt{RS}$

$$= 46.57 \sqrt{1.19 \times 0.0002}$$

$$= 0.718 \text{ m/sec}$$

$$\text{So, C.V.R.} = \frac{V}{V_0} = \frac{0.718}{0.737} = 0.974$$

As, C.V.R. is less than 1.

The silting may occur in the canal bed. So, by adjusting the full supply depth by trial, the critical velocity ratio should be made exactly or nearly equal to 1.

9.4 UNLINED CANAL DESIGN ON ALLUVIAL SOIL BY KENNEDY'S THEORY

After long investigations on the channels of upper Bari Doab canal system in Punjab (Pakistan). R.G. Kennedy arrived at a theory which states that, the silt carried by flowing water in a channel is kept in suspension by the vertical component of eddy current which is formed over the entire bed width of the channel and the suspended silt rises up gently towards the surface.

The following assumptions are made in support of his theory,

(a) The eddy current is developed due to the roughness of the bed.
(b) The quality of the suspended silt is proportional to bed width.
(c) It is applicable to those channels which are flowing through the bed consisting of sandy silt or same grade of silt as in upper Bari Doab canal system.

He established the idea of critical velocity 'V_0' which will make a channel free from silting or scouring. From, long observations, he established a relation between the critical velocity and the full supply depth as follows,

$$V_0 = C \times D^n \tag{1}$$

The values of C and n where found out as 0.546 and 0.64 respectively.

Thus, $$V_0 = 0.546 \times D^{0.64} \tag{2}$$

Again, the realised that the critical velocity was affected by the grade of silt. So, he introduced another factor (m) which is known as critical velocity ratio (C.V.R).

Thus, $$V_0 = 0.546 \times m \times D^{0.64} \tag{3}$$

Drawbacks in Kennedy's Theory

1. The theory is limited to average regime channel only.
2. The design of channel is based on the trial and error method.
3. The value of 'm' was fixed arbitrarily.
4. Silt charge and silt grade are not considered.
5. There is no equation for determining the bed slope and it depends on Kutter's equation only.
6. The ratio of 'B' to 'D' has no significance in his theory.

Expressions Adopted for the Design by Kennedy

(a) Critical velocity, $V_0 = 0.546 \times m \times D^{0.64}$

(b) Mean velocity, $V = C \times \sqrt{RS}$

where, m = critical velocity ratio, D = full supply depth in m, R = hydraulic mean depth of radius in m, S = bed slope as 1 in 'n'.

The value of 'C' is calculated by Kutter's formula,

$$C = \frac{23 + \dfrac{0.00155}{S} + \dfrac{1}{N}}{1 + \left(23 + \dfrac{0.00155}{S}\right) \times \dfrac{N}{\sqrt{R}}}$$

where, N = rugosity coefficient which is taken as unlined earthen channel.

(c) B/D ratio assumed between 3.5 to 12.

(d) Discharge, $Q = A \times V$

where, A = c/s area in m^2, V = mean velocity in m/sec.

(e) The full supply depth is fixed by trial to satisfy the value of 'm'. Generally, the trial depth is assumed between 1 m to 2 m. If the condition is not satisfied within this limit, then it may be assumed accordingly.

Problem 1 Design an irrigation channel with the following data,

(a) Full supply discharge = 6 cumec

(b) Rugosity coefficient (N) = 0.0225

(c) C.V.R. (m) = 1

(d) Bed slope = 1 in 5000

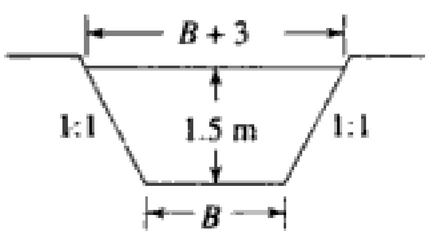
Assume other reasonable data for the design.

Solution Given data, $Q = 6$ cumec, $N = 0.0225$, $m = 1$, $S = \dfrac{1}{5000} = 0.0002$

Let us assume, Full supply depth, $D = 1.5$ m

Side slope = 1:1

Bed width = B



Cross-sectional area, $A = \dfrac{B + B + 3}{2} \times 1.5$

or $A = 1.5\,B + 2.25$ (1)

Wetted perimeter, $P_w = B + 2\sqrt{2} \times 1.5$

or $P_w = B + 4.24$ (2)

Now, critical velocity, $V_0 = 0.546 \times 1 \times 1.5^{0.64} = 0.707$ m/sec.

$$A = \frac{Q}{V} = \frac{6}{0.707} = 8.49 \text{ m}^2 \text{ [As } m = 1, V = V_0]$$

From, (1)

$$8.49 = 1.5\,B + 2.25$$

$$B = 4.16 \text{ m}$$

So, $P_w = 4.16 + 4.24 = 8.40$ m

$$R = \frac{A}{P_w} = \frac{8.49}{8.40} = 1.0 \text{ m}$$

The value of *C* is found out by Kutter's formula

$$C = \frac{23 + \dfrac{0.00155}{0.0002} + \dfrac{1}{0.0225}}{1 + \left(23 + \dfrac{0.00155}{0.0002} \times \dfrac{0.0225}{\sqrt{1.0}}\right)}$$

$$= 44.49$$

Mean velocity by Chezy's formula.

$$V = 44.49\ \sqrt{1 \times 0.0002}$$

$$= 0.629 \text{ m/sec}$$

$$\text{C.V.R.} = -\frac{0.629}{0.707} = 0.889$$

As the critical velocity ratio is much less than 1, the channel will be in silting. So, the design is not satisfactory. Here, the full supply depth is to be assumed by trials to get the satisfactory result.

First Trial Let us assume, the full supply depth as 1.25 m.

so, $V_0 = 0.546 \times 1 \times 1.25^{0.64} = 0.629$ m/sec

$$A = \frac{6}{0.629} = 9.53 \text{ m}^2 \qquad \text{[Considering } V = V_0\text{]}$$

From (1) $9.53 = 1.25\,B + 1.56$

$$B = 6.38 \text{ m}$$

$$P_w = 6.38 + 2\sqrt{2} \times 1.25 = 9.92 \text{ m}$$

$$R = \frac{9.53}{9.92} = 0.96 \text{ m}$$

Therefore, the value of *C* is modified by,

$$C = \frac{23 + \dfrac{0.00155}{0.0002} + \dfrac{1}{0.0225}}{1 + \left(23 + \dfrac{0.00155}{0.0002}\right) \times \dfrac{0.0225}{\sqrt{0.96}}}$$

$$= \frac{75.19}{1.70} = 44.23$$

$$V = 44.23\ \sqrt{0.96 \times 0.0002} = 0.613 \text{ m/sec}$$

$$\text{C.V.R.} = \frac{0.613}{0.629} = 0.97$$

In this case, the C.V.R. is very close to 1. So, the design may be accepted.

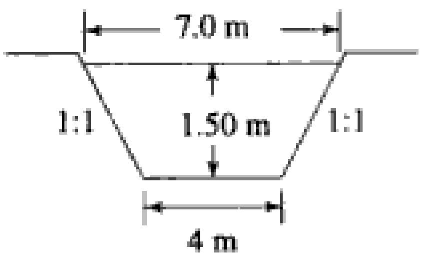
So, finally, $D = 1.25$ m, and $B = 6.38$ m.

Problem 2 Find the maximum discharge through an irrigation channel having the bed width 4 m and fully supply depth is 1.50 m. Given that $N = 0.02$, $S = 0.0002$, side slope = 1 : 1.

Assume reasonable data, if necessary. Comment whether the channel will be in scouring or silting.

Solution

Cross-sectional area, $A = \dfrac{4+7}{2} \times 1.5 = 8.25 \text{ m}^2$



Wetted perimeter, $P_w = 4 + 2\sqrt{(1.5)^2 + (1.5)^2}$

$= 4 + 4.24 = 8.24$ m

Hydraulic mean depth, $R = \dfrac{A}{P_w} = \dfrac{8.25}{8.24} = 1.0$ m

According to Manning's formula

$$V = \frac{1}{N} \times R^{2/3} \times S^{1/2}$$

$$= \frac{1}{0.02} \times 1^{2/3} \times (0.0002)^{1/2}$$

$$= 50 \times 1 \times 0.014 = 0.707 \text{ m/sec}$$

Discharge, $Q = A \times V$

$= 8.25 \times 0.707 = 5.83$ cumec

According to Kennedy's formula,

Critical velocity, $V_0 = 0.546 \times 1 \times (1.50)^{0.64}$, (Taking $m = 1$)

$= 0.707$ m/sec

For finding the C.V.R. the mean velocity by Chezy's formula is required.

$$\text{Value of } C = \frac{23 + \dfrac{0.00155}{0.0002} + \dfrac{1}{0.02}}{1 + \left(23 + \dfrac{0.00155}{0.0002}\right) \times \dfrac{0.002}{\sqrt{1.0}}}$$

$= 50$

Mean velocity, $V = C \times \sqrt{RS}$

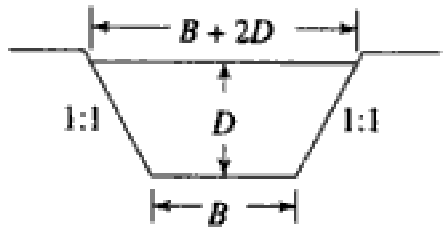
$= 50 \times \sqrt{1 \times 0.0002} = 0.707$ m/sec

$$\text{So, C.V.R.} = \frac{V}{V_0} = \frac{0.707}{0.707} = 1$$

Since, C.V.R. is exactly 1. The channel is perfectly non-scouring and non-silting.

Problem 3 Design an irrigation channel with the following data

(a) Full supply discharge = 10 cumec
(b) Bazin's constant $K = 1.3$
(c) C.V.R. (m) = 1
(d) B/D ratio = 4
(e) Side slope = 1 : 1



Solution

Let, B = bed width, D = full supply depth.

$$A = (B + D)D, \left[\frac{B}{D} = 4 \text{ or } B = 4D\right]$$

$$= (4D + D)D = 5D^2 \quad (1)$$

Again, $V_0 = 0.546 \times 1 \times D^{0.64}$

$$= 0.546 \times D^{0.64} \quad (2)$$

As, $m = 1$, So, $V_0 = V$,

From $Q = A \times V$

or $10 = 5D^2 \times 0.546 \times D^{0.64}$

$$= 2.73 \times D^{2.64}$$

$$D = \left(\frac{10}{2.73}\right)^{\frac{1}{2.64}}$$

$$= 1.64 \text{ m}$$

$$B = 4 \times D = 4 \times 1.64 = 6.56 \text{ m}$$

$$A = 5 \times (1.64)^2 = 13.45 \text{ m}^2$$

$$P_w = B + 2\sqrt{2}\,D = 6.56 + 2\sqrt{2} \times 1.64 = 11.20 \text{ m}$$

$$R = \frac{A}{P_w} = \frac{13.45}{11.20} = 1.20 \text{ m}$$

$$V_0 = 0.546 \times (1.64)^{0.64}$$

$$= 0.749 \text{ m/sec}$$

From Bazin's formula, $C = \dfrac{87}{1 + \dfrac{K}{\sqrt{R}}} = \dfrac{87}{1 + \dfrac{1.3}{\sqrt{1.20}}} = 39.7$

By Chezy's formula,

mean velocity $V = C \times \sqrt{RS}$

$$0.749 = 39.7 \times \sqrt{1.2 \times S}$$

$$S = \frac{1}{3377}$$

So, bed width of channel = 6.56 m, full supply depth = 1.64 m, bed slope $= \dfrac{1}{3377}$.

9.5 UNLINED CANAL DESIGN ON ALLUVIAL SOIL BY LACEY'S THEORY

Lacey's theory is based on the, concept of regime condition of the channel. The regime condition will be satisfied if,

(a) The channel flows uniformly in unlimited incoherent alluvium of the same character which is transported by the channel.
(b) The silt grade and silt charge remains constant.
(c) The discharge remains constant.

In his theory, he states that the silt carried by the flowing water is kept in suspension by the vertical component of eddies. The eddies are generated at all the points on the wetted perimeter of the channel section. Again, he assumed the hydraulic mean radius R, as the variable factor and he recognised the importance of silt grade for which in introduced a factor which is known as silt factor 'f'.

Thus, he deduced the velocity as, $V = \sqrt{2/5\ f\,R}$

where, V = mean velocity in m/sec, f = silt factor, R = hydraulic mean radius in m.

Then he deduced the relationship between A, V, Q, P, S and 'f' as follows.

1. $f = 1.76\sqrt{m_r}$
2. $Af^2 = 140\ V^5$
3. $V = \left(\dfrac{Qf^2}{140}\right)^{\frac{1}{6}}$
4. $P = 4.75\sqrt{Q}$
5. Regime flow equation

$$V = 10.8\ R^{2/3}\ S^{1/3}$$

6. Regime slope equation

(a) $S = \dfrac{f^{3/2}}{4980\,R^{1/3}}$

(b) $S = \dfrac{f^{5/3}}{3340\,Q^{1/6}}$

7. Regime scour depth

$$R = 0.47 \times \left(\frac{Q}{f}\right)^{1/3}$$

where, f = silt factor, m_r = mean particle size of silt in mm, A = cross-sectional area in m^2, V = mean velocity in m/sec, Q = discharge in cumec, P = wetted perimeter in m, R = hydraulic mean radius or scour depth, S = bed slope.

From 6 (b)

8. $Q = \left[\dfrac{f^{5/3}}{3340 \times S}\right]^6$

Problem 1 Design a channel section with the following data.

(a) Full supply discharge = 10 cumec
(b) Mean diameter of silt particles = 0.33 mm
(c) Side slope = 1/2 : 1

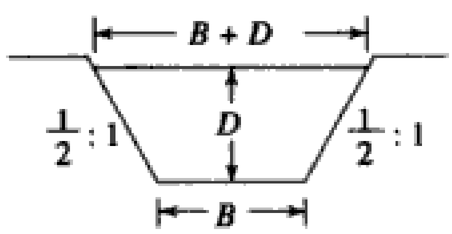
Find also the bed slope of the channel.

Solution

1. Silt factor, $f = 1.76\sqrt{0.33} = 1.0$
2. Mean velocity $V = \left(\dfrac{Qf^2}{140}\right)^{1/6} = \left(\dfrac{10 \times 1^2}{140}\right)^{1/6}$
   $= 0.64$ m/sec
3. c/s area $= \dfrac{Q}{V} = \dfrac{10}{0.64} = 15.62$ m^2
4. Wetted perimeter, $P = 4.75\sqrt{Q} = 4.75\sqrt{10} = 15.02$ m
5. Hydraulic mean radius, $R = 5/2 \cdot \dfrac{V^2}{f} = -\dfrac{5 \times (0.64)^2}{2 \times 1} = 1.02$ m

   Check $R = \dfrac{A}{P} = \dfrac{15.62}{15.02} = 1.03$ m may be taken as correct.
6. Bed slope, $S = \dfrac{f^{5/3}}{3340(Q)^{1/6}} = \dfrac{(1)^{5/3}}{3340 \times (10)^{1/6}} = \dfrac{1}{4902}$

Cross-sectional area, $A = BD + \dfrac{D^2}{2}$



or $15.62 = BD + 0.5\, D^2$ (1)

wetted perimeter, $P = B + \sqrt{5} \times D$

or $15.02 = B + 2.24\, D$

or $B = 15.02 - 2.24\, D$ (2)

Putting the value of B in (1)

$$15.62 = (15.02 - 2.24\, D) \times D + 0.5\, D^2$$
$$= 15.02\, D - 2.24\, D^2 + 0.5\, D^2$$
$$= 15.02\, D - 1.74\, D^2$$

or $1.74\, D^2 - 15.02\, D + 15.62 = 0$

$$D = \frac{15.02 \pm \sqrt{(15.02)^2 - 4 \times 1.74 \times 15.62}}{2 \times 1.74}$$
$$= \frac{15.02 \pm 10.80}{3.48} = 7.42 \text{ or } 1.21 \text{ m}$$

1. When, $D = 7.42$ m

$$B = 15.02 - 2.24 \times 7.42$$
$$= -1.60 \text{ m (which is absurd)}$$

2. When $D = 1.21$ m,

$$B = 15.02 - 2.24 \times 1.21 = 12.30 \text{ m (which is acceptable)}$$

So, F.S.D. = 1.21 m

Bed width = 12.30 m, bed slope = 1 in 4902.

Problem 2 Find the section and maximum discharge of a channel with the following data,

(a) Bed slope = 1 in 5000
(b) Lacey's silt factor = 0.95
(c) Side slope = 1 : 1

Solution

1. Discharge, $Q = \left[\dfrac{f^{5/3}}{3340 \times S}\right]^6 = \left[\dfrac{(0.95)^{5/3}}{3340 \times 0.0002}\right]^6 = 6.74$ cumec

$$\left(S = \frac{1}{5000} = 0.0002\right)$$

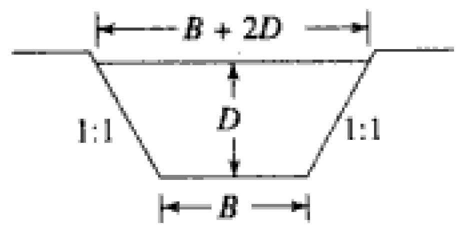
2. Mean velocity $V = \left[\dfrac{Qf^2}{140}\right]^{1/6} = \left[\dfrac{6.74 \times (0.95)^2}{140}\right]^{1/6} = 0.59$ m/sec

3. Cross-sectional area, $A = \dfrac{6.74}{0.59} = 11.42 \text{ m}^2$

4. Wetted perimeter, $P = 4.75\sqrt{Q} = 4.75\sqrt{6.74} = 12.33$ m

5. Hydraulic mean radius,

$$R = \frac{11.42}{12.33} = 0.92 \text{ m}$$



Check

Again, $R = 5/2.\dfrac{V^2}{f} = \dfrac{5}{2}\cdot\dfrac{(0.59)^2}{0.95} = 0.92$ m

Let, B = bed width, D = full supply depth

$$\text{c/s Area} = (B + D)\,D$$

or $$11.42 = (B + D)\cdot D \quad (1)$$

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

or $$12.33 = B + 2.82\,D$$

or $$B = 12.33 - 2.82\,D \quad (2)$$

From (1) and (2)

$$11.42 = (12.33 - 2.82\,D + D)D$$
$$= 12.33\,D - 1.82\,D^2$$

or $$1.82\,D^2 - 12.33\,D + 11.42 = 0$$

$$D = \frac{12.33 \pm \sqrt{(12.33)^2 - 4 \times 1.82 \times 11.42}}{2 \times 1.82}$$

$$= \frac{12.33 \pm 8.30}{2.64} = 7.80 \text{ or } 1.52 \text{ m}$$

When, $D = 7.80$ m, $B = 12.33 - 2.82 \times 7.80 = -9.66$ m (which is absurd)
When, $D = 1.52$ m, $B = 12.33 - 2.82 \times 1.52 = 8.0$ m (which is acceptable)
So, Bed width = 8.0 m, Full supply depth = 1.52 m, Discharge, $Q = 6.74$ cumec.

Drawbacks in Lacey's Theory The followings are the drawbacks of Lacey's theory,

1. The concept of true regime is theoretical and cannot be achieved practically.
2. The various equations are derived by considering the silt factor f which is not at all constant.
3. The concentration of silt is not taken into account.
4. Silt grade and silt charge are not clearly defined.
5. The equations are empirical and based on the available data from a particular type of channel. So, it may not be true for a different type of channel.
6. The characteristics of regime channel may not be same for all cases.

9.6 COMPARISON BETWEEN KENNEDY'S THEORY AND LACEY'S THEORY

Kennedy's theory	*Lacey's theory*
1. It states that the silt carried by the flowing water is kept in suspension pension by the vertical component of eddies which are generated from the bed of the channel.	1. It states that the silt carried by the flowing water is kept in suspension by the vertical component of eddies which are generated from the entire wetted perimeter of the channel.
2. It gives relation between 'V' and 'D'.	2. It gives relation between 'V' and 'R'.
3. In this theory, a factor known as critical velocity ratio 'm' is introduced to make the equation applicable to different channels with different silt grades.	3. In this theory, a factor known as silt factor 'f' is introduced to make the equation applicable to different channels with different silt grades.
4. In this theory, Kutter's equation is used for finding the mean velocity.	4. This theory gives an equation for finding the mean velocity.
5. This theory gives no equation for bed slope.	5. This theory gives an equation for bed slope.
6. In this theory, the design is based on trial-and-error method.	6. This theory does not involve trial-and-error method.

Table 2 Design Parameters for Frapezoidal Section

Design parameter	*Side slope*		
	1:1	*1.5 : 1*	*1.25 : 1*
Sectional area (A)	$BD + 1.785\ D^2$	$BD + 2.088\ D^2$	$BD + 1.925\ D^2$
Wetted perimeter (P)	$B + 3.570\ D$	$B + 4.176\ D$	$B + 3.85\ D$
Hydraulic mean depth or radius (R)	A/P	A/P	A/P
Velocity (V)	$\frac{1}{N} \times R^{2/3} \times S^{1/2}$	—	—
Discharge	$A \times V$	—	—

Problem 1 Design a lined canal to carry a discharge of 40 cumec. Assume bed slope as 1 in 5000, $N = 0.0225$ and side slope—1:1.

Solution Since the discharge is less than 50, the circular section is suitable. From Table 1. $A = 1.785\ D^2$, D = Full supply depth, $P = 3.57\ D$.

Hydraulic mean depth $R = \dfrac{A}{P} = \dfrac{1.785\ D^2}{0.57\ D} = 0.5\ D$

So, $$V = \frac{1}{0.0225} \times (0.5\ D)^{2/3} \times (0.0002)^{1/2} \left[S = \frac{1}{5000} = 0.0002\right]$$

$$= 0.38\ D^{2/3}$$

Again, $$Q = A \times V$$

or $$40 = 1.785\ D^2 \times 0.38\ D^{2/3} = 0.678\ D^{8/3}$$

or $$D^{8/3} = 58.99$$

$$D = 4.61 \text{ m}$$

$$V = 0.38 \times (4.61)^{2/3} = 1.05 \text{ m/sec}$$

$$A = 1.785 \times (4.61)^2 = 37.93 \text{ m}^2$$

$$P = 3.57 \times 4.61 = 16.45 \text{ m}$$

Problem 2 Design a lined canal having the following data

(a) Full supply discharge = 200 cumec.
(b) Side slope = 1.25 : 1
(c) Bed slope = 1 in 5000
(d) Rugosity coefficient = 0.018
(e) Permissible velocity = 1.75 m/sec

Solution Since the discharge is more than 50 cumec, the trapezoidal section will be acceptable.

From Table 2,

Sectional area, $A = B\ D + 1.925\ D^2$ (1)

Wetted perimeter, $P = B + 3.85\ D$ (2)

Now, $$A = \frac{200}{1.75} = 114.28 \text{ m}^2$$

8. Explain clearly the difference between Kennedy's theory and Lacey's theory.
9. Design an irrigation channel by Kennedy's theory having the following data
   (a) F.S.D. = 5 cumec
   (b) Coefficient of rugosity = 0.0225
   (c) C.V.R. = 1
   (d) Bed slope = 0.0002
   (e) Side slope = 1/2 : 1
   (f) Allowable depth = 1 m

   Check the design by Chezy's formula

   (*Ans*. B = 8.59 m)
10. Design an irrigation channel by Lacey's theory having the following data
   (a) F.S.D. = 5 cumec
   (b) Silt factor = 1
   (c) Side slope = 1/2 : 1

   (*Ans*. B = 8.45 m, D = 1m, S = 1/4400)

10

CANAL HEAD WORKS

10.1 INTRODUCTION

The water flows through the irrigation canal under the force of gravity. So, the elevation of the head of the canal must be higher than the command area of the irrigation project. Now, to form a storage reservoir or to raise the water level at the head of the canal, some structures are constructed which are known as canal head works. The canal head works may be of two forms.

1. Storage Head Works When a dam is constructed across a river valley to form a storage reservoir, it is known as storage head work. The water is supplied to the canal from this reservoir through the canal head regulator. Again, this reservoir serves the multipurpose functions such as hydro-electric power generation, flood control, fishery, etc.

2. Diversion Head Works When a weir or barrage is constructed across a perennial river to raise the water level and to divert the water to the canal, then it is known as diversion head work. The flow of water in the canal is controlled by canal head regulator.

10.2 OBJECT OF DIVERSION HEAD WORKS

The following are the objects of diversion of head works

(a) To raise the water level at the head of the canal.
(b) To form a storage by constructing dykes on both the banks of the river so that water is available throughout the year.
(c) To control the entry of silt into the canal and to control the deposition of silt at the head of the canal.
(d) To control the fluctuation of water level in the river during different seasons.

10.3 SELECTION OF SITE FOR STORAGE HEAD WORKS

The following points should be remembered while selecting the site for storage head works,

(a) The site should be preferably in hilly area where a river valley is available to form a deep reservoir with minimum surface area.
(b) Good foundation should be available at the dam site.
(c) There should be no fissures, cracks or premeable formations in the reservoir area.
(d) The catchment area should be large enough so that the required water capacity of the reservoir may be fulfilled.
(e) The tributories should not carry much sediments which may cause reservoir sedimentation.
(f) The catchment area should not consist of the zone of mica particles or the zone of slips. If unavoidable, then necessary steps should be taken for the stabilisation of those zones.
(g) Materials of construction should be available in the vicinity of the site.
(h) Road or railway communication to the site should be easy.

10.4 SELECTION OF SITE FOR DIVERSION HEAD WORKS

The following points should be remembered while selecting the site for the diversion head works

1. At the site, the river should be straight and narrow.
2. The river banks should be well defined.
3. The valuable land should not be submerged when the weir or barrage is constructed.
4. The elevation of the site should be much higher than the area to be irrigated.
5. The site should be easily accessible by roads or railways.
6. The materials of construction should be available in vicinity of the site.
7. The site should not be far away from the command area of the project, to avoid transmission loss.

10.5 COMPONENTS PARTS OF DIVERSION HEAD WORKS

The following are the component parts of the diversion head works (Fig. 10.1)

1. Weir or barrage.
2. Divide wall.
3. Scouring sluices or under sluices.
4. Fish ladder.
5. Canal head regulator.
6. Silt excluder.
7. Guide bank.
8. Marginal embankment or Dyke.

10.6 WEIR OR BARRAGE

(a) Weir Normally, the water level of any perennial river is such that it cannot be diverted to the irrigation canal. The bed level of the canal may be higher than

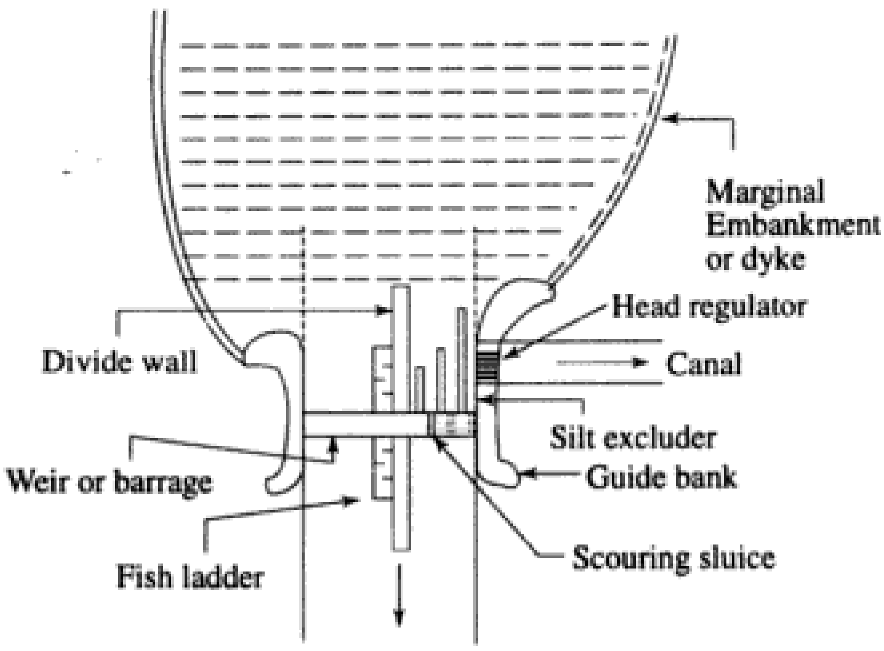


Fig. 10.1 Diversion head works

the existing water level of the river. In such a case, a weir is constructed across the river to raise the water level from H_1 to H_2 (Fig. 10.2). Then, the water can be easily diverted to the canal. The surplus water passes over the crest of the weir. Sometimes, adjustable shutters are provided on the crest to raise the water level to some required height, if necessary. But these shutters are dropped down during the flood. The weir may be constructed with masonry or concrete. The details of weir are given in Chapter 11.

(b) Barrage When the water level on the up stream side of the weir is required to be raised to different levels at different time, then the barrage is constructed. Practically a barrage is an arrangement of adjustable gates or shutters at different tiers over the weir. The water level can be adjusted at H_1, H_2, H_3 ..., etc. by operating the adjustable gates (Fig. 10.3). (The details of barrage are given in Chapter 11).

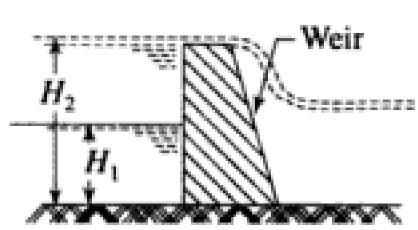


Fig. 10.2 Weir

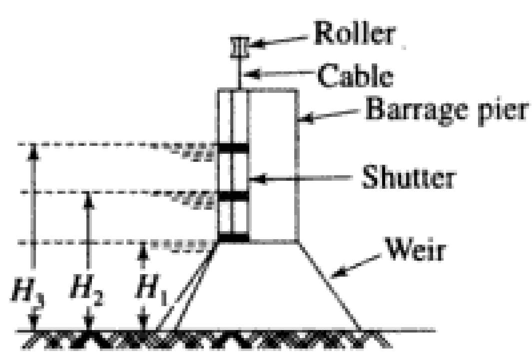


Fig. 10.3 Barrage

10.7 DIVIDE WALL

The divide wall is a long wall constructed at right angles to the weir or barrage, it may be constructed with stone masonry or cement concrete (Fig. 10.4). On the up stream side, the wall is extended just to cover the canal head regulator and on

For the free movement of the fishes along the course of the river, the fish ladder is essential. In the fish ladder, the baffle walls are constructed in a zig-zag manner so that the velocity of flow within the ladder does not exceed 3m/sec. The width, length and height of the fish ladder depends on the nature of the river and the type of the weir or barrage (Fig. 10.6).

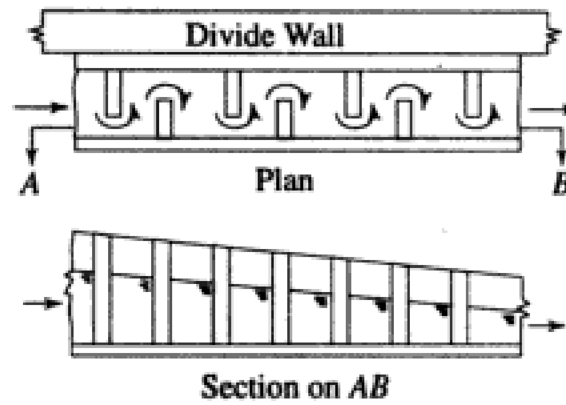


Fig. 10.6 Fish ladder

10.10 CANAL HEAD REGULATOR

A structure which is constructed at the head of the canal to regulate flow of water is known as canal head regulator. It consists of a number of piers which divide the total width of the canal into a number of spans which are known as bays. The piers consist of a number tiers on which the adjustable gates are placed. The gates are operated from the top by suitable mechanical device. A platform is provided on the top of the piers for the facility of operating the gates. Again some piers are constructed on the down stream side of the canal head to support the roadway (Fig. 10.7).

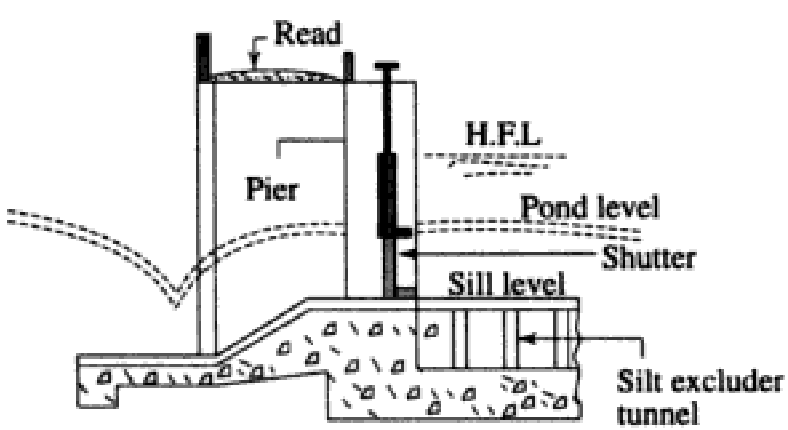


Fig. 10.7 Canal head regulator

10.11 SILT EXCLUDER

When still water pocket is formed in front of the canal head by constructing the divide wall, then it is found that the lower layer of water contains heavy silt and the upper layer contains very fine silt. The fine silt is very fertile and it may be allowed to enter the canal. But the heavy silt causes sedimentation in the pocket. To eliminate the suspended heavy silt, the silt excluder is provided. It consists of a series of tunnels starting from the side of the head regulator up to the divide wall. The tunnel nearest to the head regulator is longest, and the successive tunnels decrease in length, the tunnel nearest to the divide wall is shortest. The tunnels are covered by R.C.C. slab. The top level of the slab is kept below the Sill level of the head regulator. So, the comparatively clear water (containing fine silt) is allowed to flow in the canal through the head regulator. The suspended heavy silt carried by the water enters the silt excluder tunnels and passes out through the scouring sluices (Fig. 10.8).

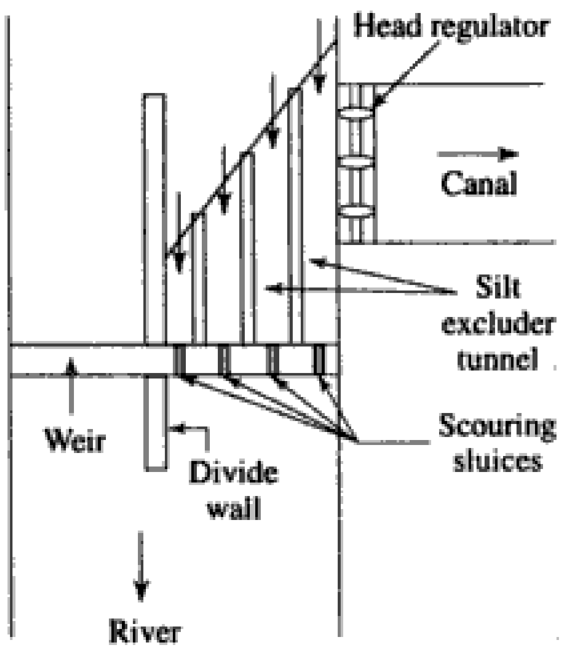


Fig. 10.8 Silt Excluder

10.12 MARGINAL EMBANKMENT OR DYKE

The marginal embankments or dykes are earthen embankments which are constructed parallel to the river bank on one or both the banks according to the condition. The top width is generally 3 m to 4 m. The side slope on the river side is generally $1\frac{1}{2} : 1$ and that on the country side is 2:1. The height of the embankment depends on the highest flood level. A suitable margin is provided between the toe of the embankment and the bank of the river. To resist the effect of erosion on the embankment, wooden piles (generally, sal ballah) are driven along the river banks throughout the length of dyke. The length of the dyke depends on the area to be protected. The river side slope of the dyke is protected by boulder pitching with cement grouting and the country side slope is protected by turfing Fig. 10.9. The dyke serves the following purposes.

(a) It prevents the flood water or storage water from entering the surrounding area which may be submerged or may be water logged.
(b) It retains the flood water or storage water within a specified section.
(c) It protects the towns and villages from devastation during the heavy flood.
(d) It protects valuable agricultural lands.

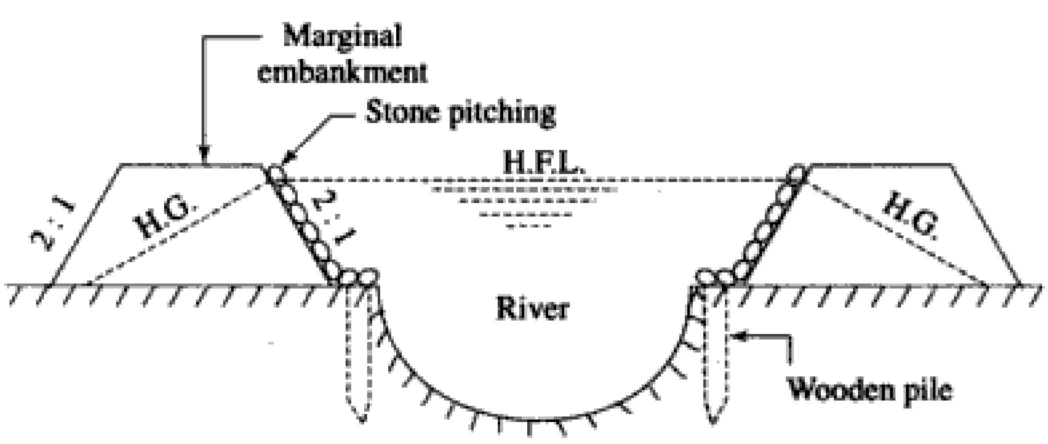


Fig. 10.9 Marginal embankment

10.13 GUIDE BANK

When a barrage is constructed across a river which flows through the alluvial soil, the guide banks must be constructed on both the approaches to protect the structure from erosion. It is an earthen embankment with curved heads on both the ends. The upstream curved head extends up to 1.5 L and downstream curved head extends up to 0.25 L from the centreline of the barrage or bridge, where *L* is the distance between the abutments. The intermediate straight portion is known as shank. The crest width of the bank varies from 3 m to 4 m and its height depends on the highest flood level. A free board of about 1.5 m should be provided. The side slope on the river side is generally $1\frac{1}{2}:1$ and that on the country side is 2 : 1. The river side slope is protected by boulder pitching and the country side slope is protected by turfing. As the guide bank was first designed by Bell, it is sometimes known as Bell's Bund (Fig. 10.10). The Guide bank serves the following purposes,

(a) It protects the barrage from the effect of scouring and erosion.
(b) It provides a straight approach towards the barrage.
(c) It controls the tendency of changing the course of the river.
(d) It controls the velocity of flow near the structure.

Component Parts of the Guide Bank

(a) Upstream curved head.
(b) Downstream curved head.
(c) Shank.
(d) Sloping apron.
(e) Launching apron with sausage work.
(f) Pile protection.

(a) Upstream Curved Head This head extends up to a distance of 0.45 *L* from the upstream end of the shank. The tangent at the nose of the curve subtends an angle of 120° –145° with the perpendicular drawn at the upstream end of the shunk. The function of this head is to resist the thrust of the water during the flood and to protect the structure from damage. The river side of the head is protected by boulder pitching, launching apron and piling.

(b) Downstream Curved Head This head extends up to a distance of 0.25 *L* from the downstream end of the shank. The tengents at the nose of the curve and the perpendicular at the down stream end of shank subtend an angle of 45°– 60°. The river side of this head is protected by boulder pitching, launching apron and piling.

(c) Shank The straight portion of guide bank is known as shank. The guide bank may be parallel to the river bank or may be divergent or convergent according to the site condition. So, the shank may be parallel or oblique to the river bank. The river side of the shank is protected by boulder pitching, launching apron and piling. The country side is protected by turfing only.

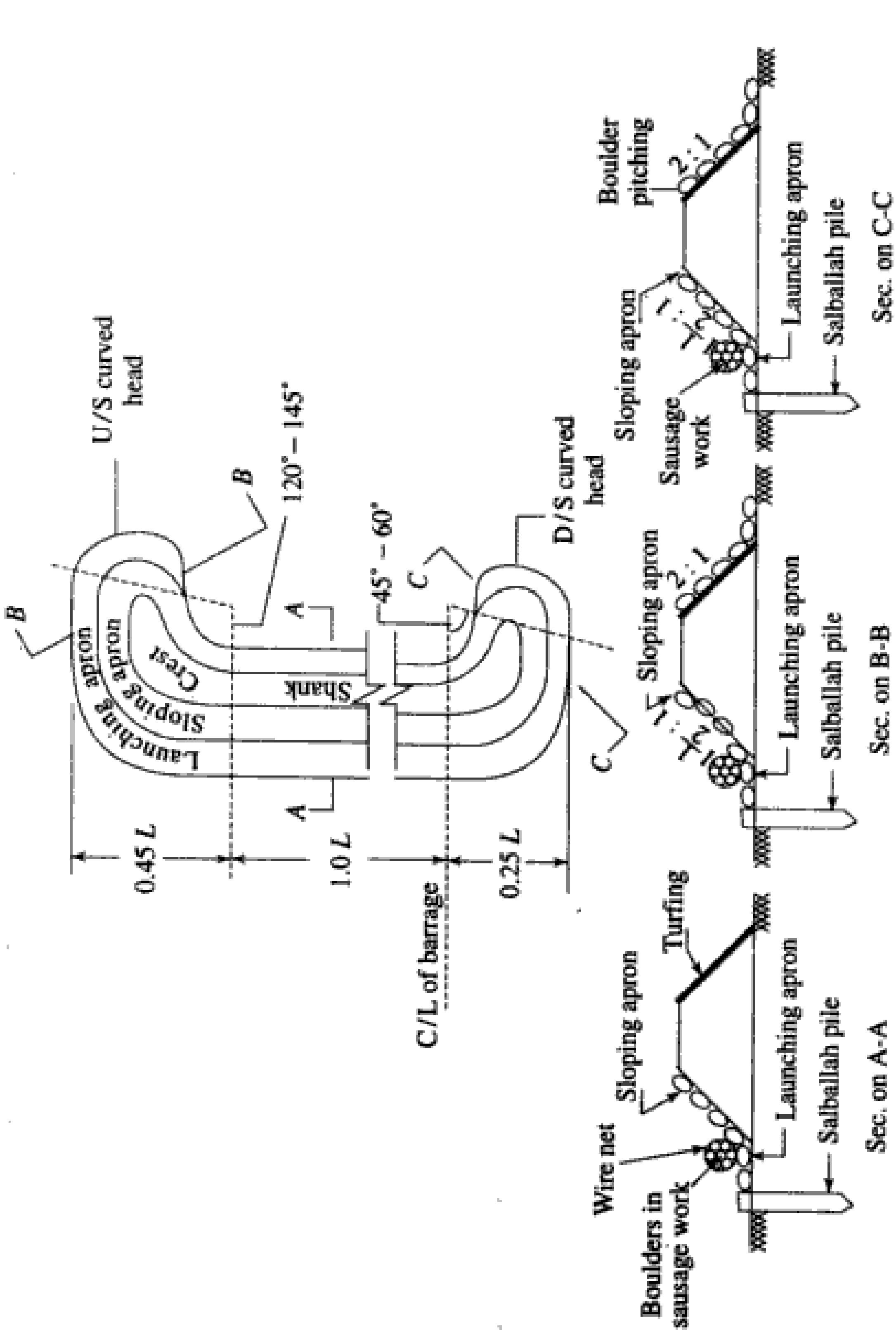


Fig. 10.10 Guide bank

(d) Sloping Apron This apron consists of boulder pitching with cement grouting. The slope is generally $1\frac{1}{2} : 1$, but it may vary according to the nature of the soil. Generally, single layer pitching is provided. But for the rivers in hilly region, double layer apron is recommended. The boulders weighing 30–50 kg should always be recommended for the protective work of the guide bank.

(e) Launching Apron with Sausage Work The Launching apron consists of double layer boulder (30–50 kg) pitching. The width of the apron varies from 2 to 3 m. At the toe of the guide bank, the sausage work is done. In this work, the boulders (10–20 kg) are enclosed in a wire net approximately circular in shape and laid along the toe of the bank covering the entire river side. The diameter of the sausage may be about 1 to 1.5 m.

(f) Pile Protection The salballah piles are driven along the edge of the launching apron for the stabilisation of soil and to protect the guide bank from scouring and erosion. The piles are coated with tar and the spacing is generally 1 m centre to centre. The spacing may vary according to the site condition.

REVIEW QUESTIONS

1. Fill up the blanks with appropriate word/words.
   (i) When a dam is constructed across a river valley to form storage reservoir then it is known as __________.
   (ii) When a barrage is constructed across a river to raise the water level then it is known as __________.
   (iii) To form a still water pocket in front of the canal head a __________ is constructed.
   (iv) For the movement of fishes the __________ is provided by the side of divide wall.
   (v) The __________ is provided to eliminate the deposited silt in front of the head regulator.
   (vi) The __________ bank is provided to protect the hydraulic structure from scouring and erosion.
   (vii) The guide bank is also known as __________ bund.
2. Distinguish between storage head works and diversion head works.
3. Name the component parts of the diversion head works and state their functions.
4. Distinguish between the following,
   (a) Weir and barrage
   (b) Guide bank and marginal bank
   (c) Silt excluder and scouring sluice.
5. Enumerate the points to be considered while selecting the site for diversion head works.

ANSWERS

1. (i) Storage head works (ii) Diversion head works
(iii) Divide wall (iv) Fish ladder
(v) Silt excluder or scouring sluice (vi) Guide
(vii) Bell's

11

WEIR AND BARRAGE

11.1 INTRODUCTION

The sub-surface flow of water plays an important role for the stability of hydraulic structures like weir or barrage on alluvial soil. The seepage water exerts uplift pressure on the foundation and on the protective aprons. To counter balance this uplift force, necessary measures should be taken. The characteristics of seepage flow as established by Bligh's creep theory and Khosla's theory should be taken into account. These two theories recommend the adoption of sheet piles below the upstream and downstream breast wall of weir and barrage. The sheet piles lengthen the path of seepage water and destroys the effect of uplift pressure on the structures, The causes of failure should be investigated so that necessary precautions can be taken during the designing stage of hydraulic structures.

11.2 THEORY OF SEEPAGE FLOW

The steady seepage flow through an isotropic and homogeneous soil mass can be represented by the laplacian equation,

$$\frac{d^2\phi}{dx^2} + \frac{d^2\phi}{dy^2} = 0$$

Here, ϕ = flow potential = Kh,
where, K = coefficient of permeability,
h = seepage head at any point in the soil.

The above equation represents two sets of curve which intersect each other orthogonally. One set of curves is known as equipotential lines and the other set of curves is known as stream lines.

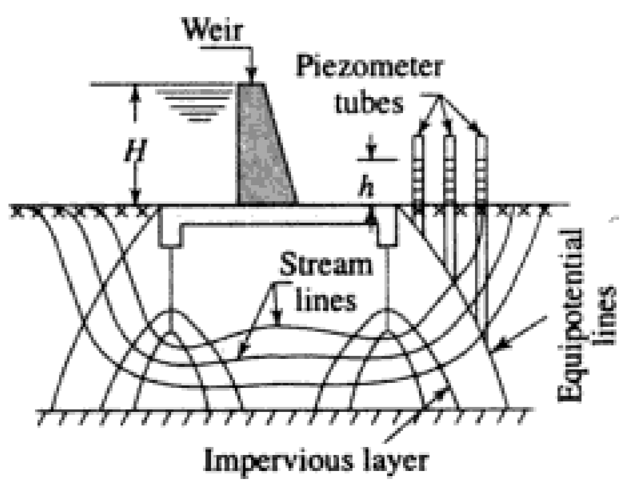


Fig. 11.1 Flow lines

Streamlines The path along which the sub-surface water flows through the soil indicates the streamline. Every water particle traces out its own streamline while moving from upstream to downstream below the foundation of the hydraulic structure. The first streamline just follows the bottom surface of foundation of the structure. The other streamlines trace out semi-elliptical path (Fig. 11.1).

Equipotential Lines Every streamline possesses a certain head H (i.e. the depth of water on upstream side), when it just enters the soil. This head goes on decreasing as it travels towards the downstream and ultimately it becomes zero. By piezometer tubes, it may be found that at some intermediate point in the streamline, there could be certain residual head 'h' which gets dissipated in the remaining length of its path. Thus, on different streamlines there may be a point of equal residual head 'h'. If these points are joined, then a curve is obtained which is known as equipotential line (Fig. 11.1).

11.3 HYDRAULIC GRADIENT OR EXIT GRADIENT

Let AA, BB, CC, represent the seepage flow. When the seepage water flows through the soil it exerts some force which is known as seepage force. The force acts in the direction of flow.

Let us consider an element of cross-sectional area as dA and its length as dl. The intensity of hydrostatic pressure on upstream face is assumed as p and that on the downstream face as $(p + \mathrm{d}p)$ (Fig. 11.2). Seepage force in the direction of flow $= p\,\mathrm{d}A$ and seepage force against the flow $= (p + \mathrm{d}p)\,\mathrm{d}A$.

Net seepage force $= p\,\mathrm{d}A - (p + \mathrm{d}p)\,\mathrm{d}A = -\mathrm{d}p \cdot \mathrm{d}A$

The volume of element $= \mathrm{d}A \cdot \mathrm{d}l$

Force per unit volume $= \dfrac{-\mathrm{d}p \cdot \mathrm{d}A}{\mathrm{d}A \cdot \mathrm{d}l} = -\dfrac{\mathrm{d}p}{\mathrm{d}l}$

The force $\left(-\dfrac{\mathrm{d}p}{\mathrm{d}l}\right)$ acts tangentially to the streamline. Here, – ve sign indicates that the pressure decreases in the direction of flow. This force is vertical at the exit end and it is proportional to $\dfrac{\mathrm{d}h}{\mathrm{d}l}$.

This expression $\dfrac{\mathrm{d}h}{\mathrm{d}l}$ is known as the hydraulic gradient or exit gradient. The soil at the exit end will remain stable if the downward weight of the submerged soil is greater than the upward seepage force. When the two forces just balance each other, then it is known as critical hydraulic gradient or critical exit gradient. The critical condition will be achieved as follows,

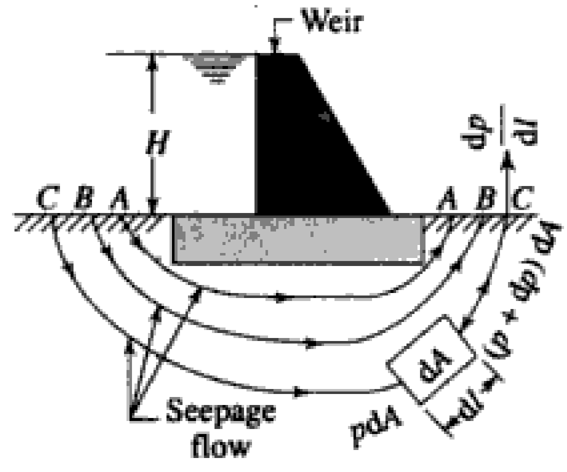


Fig. 11.2 Exit gradient

Submerged wt. per unit volume of soil mass, $\omega_s = \omega(1-n)(G-1)$
where, ω_s = Submerged wt. per unit vol, ω = sp. wt. of water, n = porosity of soil, G = sp. gr. of soil.

For critical condition, the upward force $\left(-\frac{dp}{dl}\right)$ should be equal to ω_s, i.e.

$$-\frac{dp}{dl} = w(1-n)(G-1) \text{ or } -\frac{dp}{dl\,x\,w} = (1-n)(G-1)$$

or

$$-\frac{dh}{dl} = (1-n)(G-1), \text{ we know, } dp = w\,dh \therefore dh = \frac{dp}{w}$$

Here, $\frac{dh}{dl}$ represents the hydraulic gradient at the exit end. When this gradient is equal to the value given by $(1-n)(G-1)$, then the gradient will be termed as critical hydraulic gradient or critical exit gradient. For safety, the actual exit gradient should be within 1/4 and 1/6.

11.4 CAUSES OF FAILURE OF WEIR OR BARRAGE ON PERMEABLE FOUNDATION

The combined effect of subsurface flow and surface flow may cause the failure of the weir or barrage.

1. Failure due to Subsurface Flow

(a) By piping or undermining The water from the upstream side continuously percolates through the bottom of the foundation and emerges at the downstream end of the weir or barrage floor. The force of percolating water removes the soil particles by scouring at the point of emergence. As the process of removal of soil particles goes on continuously, a depression is formed which extends backwards towards the upstream through the bottom of the foundation. A hollow pipe like formation thus develops under the foundation due to which the weir or barrage may fail by subsiding. This phenomenon is known as failure by piping or undermining.

(b) By uplift pressure The percolating water exerts an upward pressure on the foundation of the weir or barrage. If this uplift pressure is not counterbalanced by the self weight of the structure, it may fail by rapture.

2. Failure by Surface Flow

(a) By hydraulic jump When the water flows with a very high velocity over the crest of the weir or over the gates of the barrage, then hydraulic jump develops. This hydraulic jump causes a suction pressure or negative pressure on the downstream side which acts in the direction of uplift pressure. If the thickness of the impervious floor is not sufficient, then the structure fails by rapture.

(b) By scouring During floods, the gates of the barrage are kept open and the water flows with high velocity. The water may also flow with very high velocity over the crest of the weir. Both the cases can result in scouring effect on the downstream and on the upstream side of the structure. Due to scouring of the soil on both sides of the structure, its stability gets endangered by shearing.

11.5 PRECAUTIONS AGAINST FAILURE

The following precautions can be taken to prevent failure,

(a) The length of the impervious layer should be carefully designed so that the path of the percolating water is increased consequently reducing the exit gradient.

(b) Sheet piles should be provided on the upstream and downstream side of the impervious floor to increase to length of percolating water so that the uplift pressure is considerable reduced.

(c) The thickness of the impervious floor should be such that the weight of floor is a sufficient to counterbalance the uplift pressure.

(d) Energy dissipator blocks like friction block, impact block, etc should be provided.

(e) Inverted filter should be provided with concrete blocks on the top so that the percolating water does not wash out the soil particles.

(f) Loose talus should be provided for a sufficient length with heavy boulders (40 to 50 kg).

(g) Deep foundation like well foundation should be provided for the barrage piers.

11.6 BLIGH'S CREEP THEORY

Bligh's creep theory states that the percolating water creeps along the profile of the bottom of the hydraulic structure which is in contact with the subsoil. The path traced by the percolating water is known as creep length. The loss of head per unit creep length is known as hydraulic gradient, which is constant throughout its passage. Again, the loss of head is proportional to creep length (Fig. 11.3).

Let, H = depth of water on up stream side of weir, S = length of impermeable floor, x_1, x_2, x_3 = length of sheet piles.

Then, creep length = $S + 2x_1 + 2x_2 + 2x_3$.

Hydraulic gradient per unit creep length $= \frac{H}{L} = \frac{H}{S + 2x_1 + 2x_2 + 2x_3}$

The reciprocal of hydraulic gradient (i.e. $\frac{L}{H}$ is known as Bligh's creep coefficient, C, i.e.

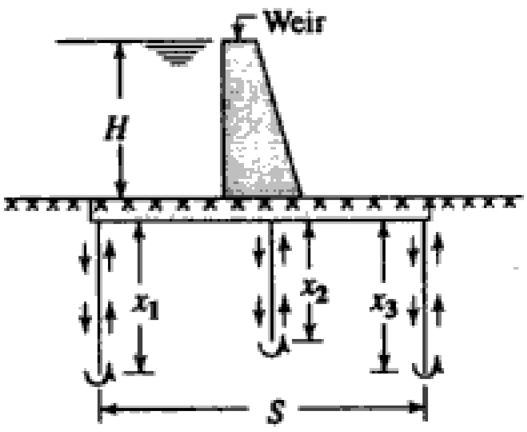


Fig. 11.3 Creep length

(d) Design of protective works like launching apron, loose talus, inverted filter etc.
(e) Exit gradient.

The permissible exit gradient for different types of soils are given below,

Type of soil	*Exit gradient*
Fine sand	1/6 to 1/7
Medium sand	1/5 to 1/6
Coarse sand	1/4 to 1/5
Sand mixed with gravels	1/4 to 1/5

11.9 TYPES OF WEIRS

The following are the different types of weirs,

(a) Masonry weir.
(b) Rock-fill weir.
(c) Concrete weir.

(a) Masonry Weir Masonry weir wall is constructed over the impervious floor. Cut-off walls are provided at both ends of the floor. Sheet piles are provided below the cut off walls. The crest shutters are provided to raise the water level, if required. The shutters are dropped down during flood. The masonry weir wall may be vertical on both face or sloping on both face or vertical on downstream face and sloping in upstream face (Fig. 11.4).

(b) Rock-fill Weir It consists of masonry breast wall which is provided with adjustable crest shutters. The upstream rock-fill portion is constructed with boulders forming a slope of 1 in 4. The boulders are grouted with cement mortar. The downstream sloping apron consists of core walls. The intermediate spaces between the core walls are filled up with boulders maintaining a slope of 1 in 20. The boulders are grouted properly with cement mortar (Fig. 11.5).

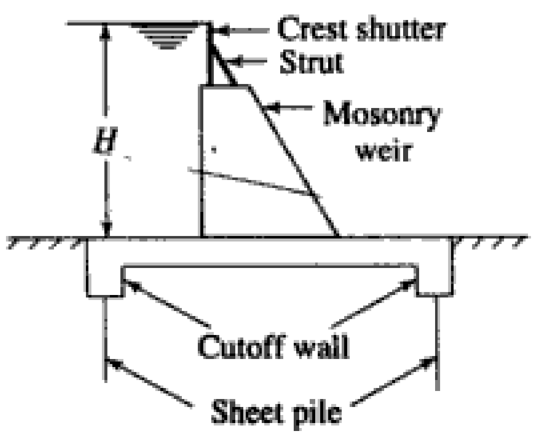


Fig. 11.4 Masonry weir

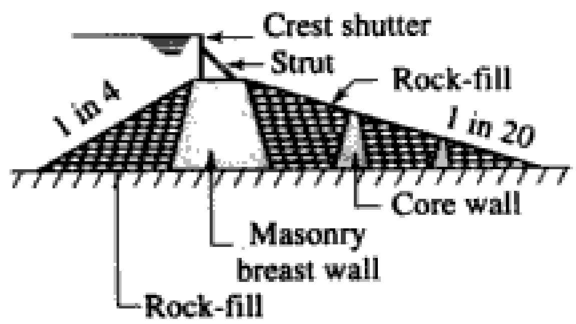


Fig. 11.5 Rock-fill weir

(c) Concrete Weir Now-a-days, the weir is constructed with reinforced cement concrete. The impervious floor and the weir are made monolythic. The cut off walls are provided at the upstream and downstream end of the floor and at the toe of the weir. Sheet piles are provided below the cut-off walls. The creat shutters are also provided which are dropped down during the flood (Fig. 11.6).

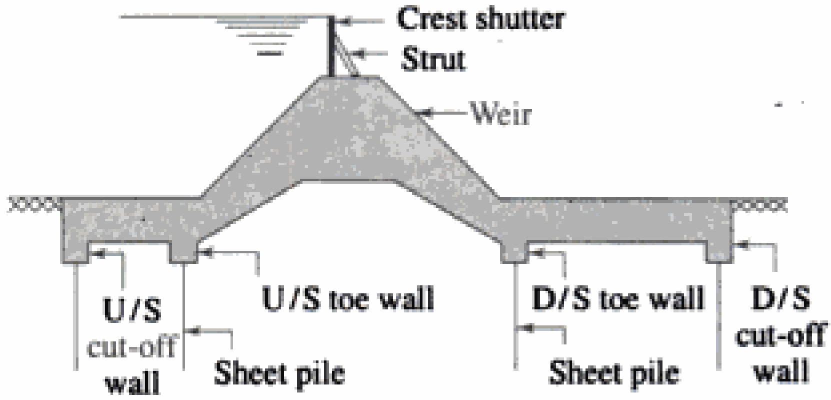


Fig. 11.6 Concrete weir

11.10 COMPONENT PARTS OF WEIR

The component parts of the weir and their functions are described in the following passages (Fig. 11.7).

The following are the functions of each component:

(1) Weir Breast It is the main body of the structure. It may be constructed with masonry work or concrete. The height and the section of the breast wall depends on the depth of water to be retained and the nature of the foundation. The function of this component is to raise the water level on the upstream side so that the water can be diverted to the irrigation canal through the head regulator.

(2) Crest Shutters These are adjustable gates or shutters provided on the crest of the weir. The bottom ends of these shutters are hinged with the crest and the top ends are free. The shutters may be raised or dropped by a mechanical device. Struts are provided on the downstream side of the shutters to resist the thrust of the water. The function of these shutters is to raise the water level on the upstream side. If required the shutters are dropped during the flood.

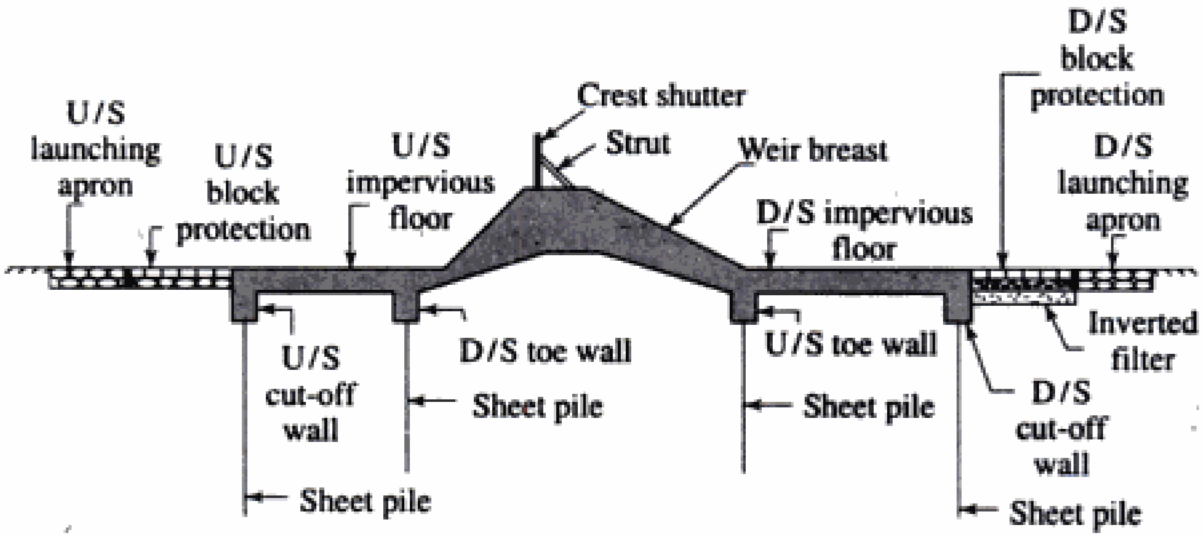


Fig. 11.7 Component parts of weir

(3) Upstream Impervious Floor or Apron Impervious floor or apron is provided to protect the main body of the weir from the scouring effect. The floor is constructed with reinforced cement concrete. In case of masonry weir, this floor covers the total designed length of upstream and downstream apron. It acts as a base plate of the weir but in case of concrete weir, the floor is made monolythic with the main body by providing reinforcement. The floor is made sloping on both sides. So, it is also known as sloping glacis.

(4) Cut-off and Toe Walls The cut-off walls are provided at the upstream end and the downstream end of the impervious floor. Walls are also provided at the upstream and the downstream toe of the weir. The function of the cut-off and toe walls are to provide proper anchorage to the impervious floor and to provide sufficient bearing to the sheet piles.

(5) Upstream Block Protection This block protects the impervious floor from the effect of scouring. This is constructed with concrete blocks or dressed stone blocks over a bed of loose stone packing. The joints are finished with cement morter. The width of this protection work is taken equal to the length of sheet piles.

(6) Upstream Launching Apron This apron is constructed with boulders or stones (not less than 30 kg) arranged in layers without any joint. It protects the impervious floor and the sheet piles from the scour holes which may develop and proceed towards the weir. The size of stones and the depth of apron depend on the velocity of flow and the probable scour depth.

(7) Sheet Piles The sheet piles are provided on the upstream and downstream cut-off walls and on the intermediate toe walls. The function of the sheet piles is to lengthen the path of the seepage flow. Thus the uplift pressure of the seepage water on the foundation is reduced considerably and the scouring effect on the exit gradient is also reduced.

(8) Downstream Impervious Floor or Apron The function of this impervious floor is to protect the weir from the scouring effect which is caused by the formation of hydraulic jump. Again, it protects the weir from the effect of piping or undermining which may occur on the downstream side due to the seepage flow.

(9) Downstream Block Protection This protection block is constructed with cement concrete blocks or dressed stones by placing them with open joints. The joints are filled up with small gravels or bajri. The seepage water can escape through the joints. Below these concrete or stone blocks, the inverted filter is provided.

(10) Inverted Filter It consists of layers of materials having the increasing grade or permeability from the bottom towards the top for example, medium sand—coarse sand—bajri—gravels—ballast are arranged from the bottom to the top, layer by layer. Thus, it is similar to a filter, but in inverted position. The

13. Downstream Launching Apron

Same as weir.

14. Deep Foundation

Deep foundation for the pier may be of two types,

(A) Well Foundation

Depending upon the base of the piers, the shape of the well foundation is ascertained. The common shapes are circular, twin-circular, dumb well, double-D, etc. The following items are calculated as usual way (vide soil mechanics and foundations engineering),

(a) Depth of well (b) Well curb, (c) Cutting edge, (d) Steining thickness, (e) Bottom plug, (f) Reinforcement.

The following procedure is adopted for sinking the well.

(1) Construction of Coffer Dam Coffer dam is constructed with sheet piles around the site where the well foundation is to be done, and an island is formed. The shape and size of coffer dam depends upon the site condition. The water inside the coffer dam is pumped out.

(2) Construction of Well Curb The centre point of the well is accurately marked and the cutting edge is placed in level bed. Then well curb is cast over the cutting edge according to the size as per design. The curing of the curb is done properly.

(3) Construction of Well Steining The well steining is constructed over the curb to a height of 2 m with designed reinforcement and grade of concrete. The curing is done for at least a week.

(4) Sinking Operation After curing of the first stage of steining, the excavation is started from inside. The excavation is done by mechanical device. Due to the excavation, the well steining goes on sinking by its self weight. Care should be taken so that the well sinks vertically. If any tilt or shift is detected, it should be rectified immediately. When it comes to the ground level, the excavation is stopped and the second stage of steining is constructed, to a height of 2 m. After curing, the excavation is again started and the well goes on sinking. In this way, stage by stage the sinking of the well is continued. After third or fourth stage, it may be noticed that the well does not sink due to its own weight. This is due to the skin friction developed between the earth and the side of the well. In that

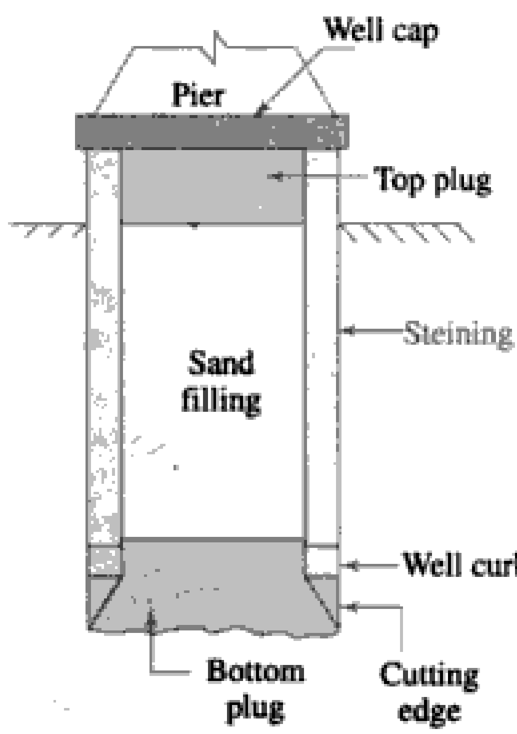


Fig. 11.9 Well foundation

case, the application of additional loading over the well should be started. The loading is done by sand bags placing systematically on the platform provided over the well. However, the process of construction (steining), sinking and loading is continued until the designed depth is reached or stable foundation is available.

(5) Plugging and Sand Filling After the completion of the process of sinking, the water inside the well is pumped out the the bottom plugging is done with cement concrete with the help of compressed air. The hollow space of the well is filled up with sand. The top portion of the well is also plugged with cement concrete. Then, a cap is provided on the well over which the pier is constructed (Fig. 11.9).

(B) Pneumatic Caisson Foundation

When the depth of water in the river is more than 10 m, then the open caisson or well foundation is not feasible due to various difficulties. In such a case the pneumatic caisson foundation is adopted. It is a box like compartment having opening at the bottom. The top end consists of lock chambers. The caisson consists of the following components,

(1) Two lock chambers First lock chamber is meant for the workers and second chamber is meant for removing the excavated materials.

(2) Working chamber In this chamber, the excavation is carried out by the workers. This chamber is air tight and compressed air is sent through the air pipe so that sub-soil water may not enter the chamber. For the physical comfort of the workers, the air pressure is limited to 3.5 kg/cm^2. Arrangements for light and oxygen are also made.

(3) Two shafts One shaft carries a ladder or lift for the workers to get down to the working chamber. Other shaft is for the removal of excavated materials.

(4) Air pipe It is provided for sending the compressed air. This pipe is connected to the air compressor. Procedure for sinking the pneumatic caisson is described here.

(i) The initial procedure is same as in the case of well foundation. The normal excavation is continued until the sub-soil water creates any problem.

(ii) When caisson operation is started, the water inside the

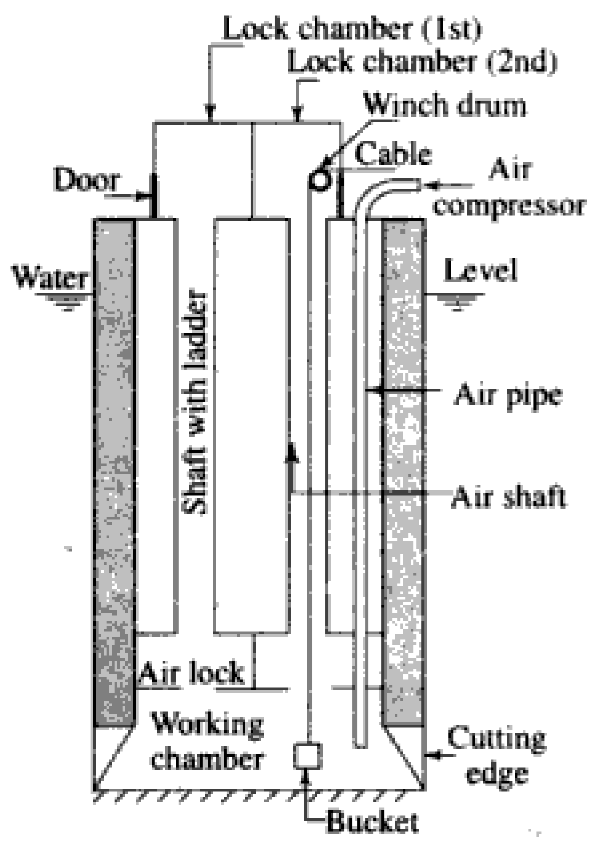


Fig. 11.10 Pneumatic caisson

working chamber is pumped out and compressed air is sent so that sub-soil water may not enter the chamber.

(iii) The workers enter the first lock chamber and get down to the working chamber through the ladder or lift. The height of the working chamber should be such that the workers can work comfortably. The excavation is done uniformly along the bottom of the cutting edge.

(iv) The excavated materials are removed by buckets operated by winch drum which is located in the second lock chamber.

(v) Construction of steining wall, excavation and sinking proceed in usual manner.

(vi) When the caisson has reached the desired depth, the bottom is plugged with cement concrete. The caisson is withdrawn and the well is filled up with sand. The top of the well also is plugged with cement concrete. Then reinforced cement concrete cap is provided on the top of the well over which the required pier is constructed (Fig 11.10).

REVIEW QUESTIONS

1. Fill up the blanks with appropriate word/words.
   (i) The path along which the sub surface water flows through the soil is known as ___________.
   (ii) On every steamline there may be a point of equal residual head. If all these points are joined, then the line is known as __________.
   (iii) When the pipe like formation is developed under the foundation of the weir or barrage due to the sub-surface flow, then it is known as ___________.
   (iv) To lengthen the path of seepage flow __________ are provided on the cut-off walls of the weir or barrage.
   (v) To allow the seepage water to escape without dislocating the soil particles on the downside of weir the __________ is provided.
2. Distinguish between streamlines and equipotential lines.
3. Enumerate the causes of failure of a barrage on permeable foundation.
4. Explain the methods to be adopted to check the failure of the barrage or weir.
5. Name the different types of weirs and describe each type with a neat sketch.
6. Name the component parts of the barrage and state their functions.
7. Explain the methods of construction of deep foundation under the barrage piers.

ANSWERS

1. (i) streamline (ii) equipotential line
   (iii) undermining (iv) sheet piles
   (v) inverted filter

12

SPILL WAYS

12.1 NECESSITY OF SPILL WAYS

The spill ways are openings provided at the body of the dam to discharge safely the excess water or flood water when the water level rises above the normal pool level (Fig. 12.1).

The spill ways are provided on the dam for the following reasons,

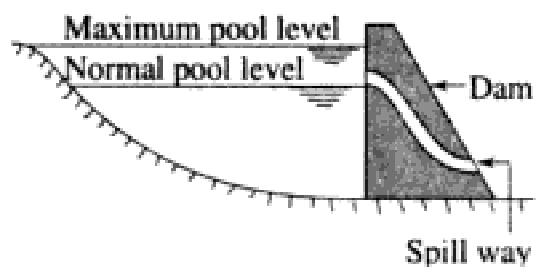


Fig. 12.1 Spill way

(a) The height of the dam is always fixed according to the maximum reservoir capacity. The normal pool level indicates the maximum capacity of the reservoir. The water is never stored in the reservoir above this level. The dam may fail by over turning so, for the safety of the dam the spill ways are essential.

(b) The top of the dam is generally utilised by making road. The surplus water in not be allowed to over top the dam, so to stop the over topping by the surplus water, the spill ways become extremely essential.

(c) To protect the downstream base and floor of the dam from the effect of scouring and erosion, the spillways are provided so that the excess water flows smoothly.

12.2 LOCATION OF SPILL WAY

Generally, the spill ways are provided at the following places

(a) Spill ways may be provided within the body of the dam (Fig. 12.1).

(b) Spill ways may sometimes be provided at one side or both sides of the dam (Fig. 12.2).

(c) Sometimes by-pass pill way is provided which is completely separate from the dam (Fig. 12.3).

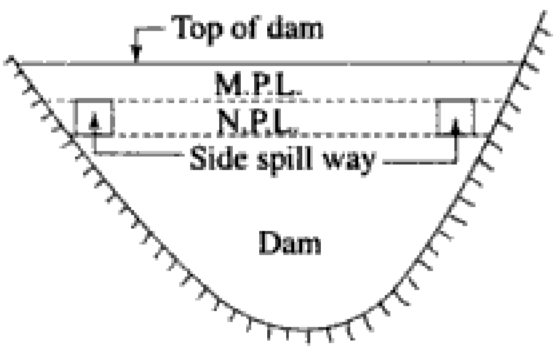


Fig. 12.2 Side spill way

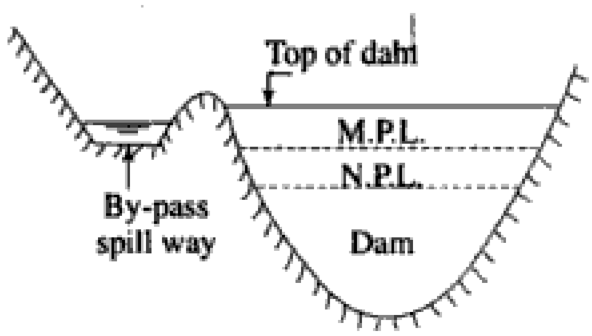


Fig. 12.3 By-pass spill way

12.3 DETERMINATION OF DISCHARGE CAPACITY AND NUMBER OF SPILL WAYS

The maximum discharge capacity and the number of spill ways are determined by studying the following factors,

(a) By studying the flood hydrography of past ten years, the maximum flood discharge may be computed which is to be disposed off completely through the spill ways.

(b) The water level in the reservoir should never be allowed to rise above the maximum pool level and should remain in normal pool level. So, the volume of water collected between maximum pool level and minimum pool level is computed, which indicates the discharge capacity of spillways.

(c) The maximum flood discharge may also be computed from other investigations like, rainfall records, flood routing, empirical flood discharge formulae, etc.

(d) From the above factors the highest flood discharge is ascertained to fix the discharge capacity of spill ways.

(e) The natural calamities are beyond the grip of human being. So, an allowance of about 25% should be given to the computed highest flood discharge which is to be disposed off.

(f) The size and number of spill ways are designed according to the design discharge.

12.4 TYPES OF SPILL WAYS

The following are the common types of spill ways.

Drop Spill Way

In drop spill way, the over flowing water falls freely and almost vertically on the downstream side of the hydraulic structure. This type of spill way is suitable for weirs or low dams. The crest of the spill way is provided with nose so that the water jet may not strike the downstream base of the structure. To protect the structure from the effect of scouring horizontal impervious apron should be provided on the downstream side (Fig. 12.4a).

Sometimes a basin is constructed on the downstream side to form a small artificial pool which is known as water cushion. This cushion serves the purpose of energy dissipator (Fig. 12.4 b).

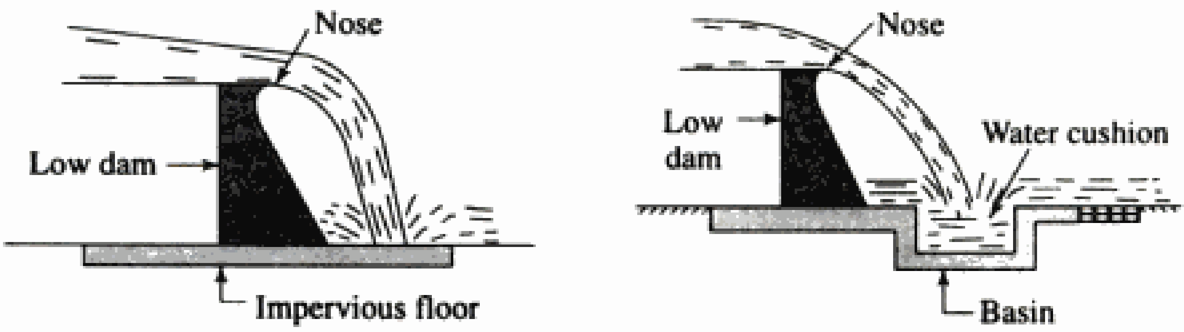


Fig. 12.4 (a) Drop spillway with impervious apron
(b) Drop spillway with water cushion

The drop spill ways is not suitable for a high dam, because the downstream apron will be subjected to high impact force for which massive protection works will be necessary. Again, high impact on the downstream apron may cause vibration in the structure which may create cracks in the foundation. Thus, the stability of the structure will be in danger due to undermining.

Ogee Spill Way

The ogee spill way is a modified form of drop spill way. Here, the downstream profile of the spill way is made to coincide with the shape of the lower nappe of the free falling waterjet from a sharp crested weir. In this case, the shape of the lower nappe is similar to a projectile and hence downstream surface of the ogee spill way will follow the parabolic path where '0' is the origin of the parabola. The downstream face of the spill way forms a concave curve from a point '*T*' and meets with the downstream floor. This point '*T*' is known as point of tangency. Thus the spill way takes the shape of the letter '*S*' (i.e. elongated form). Hence, this spill way is termed as ogee spillway. (Fig. 12.5).

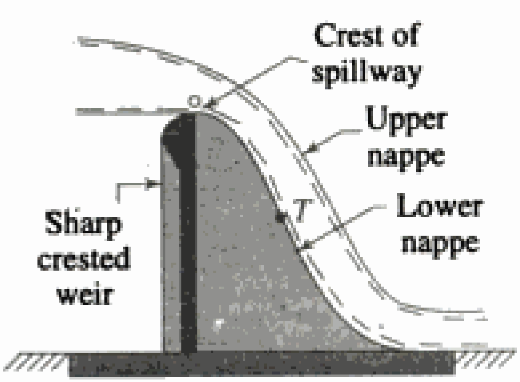


Fig. 12.5 Ogee spillway

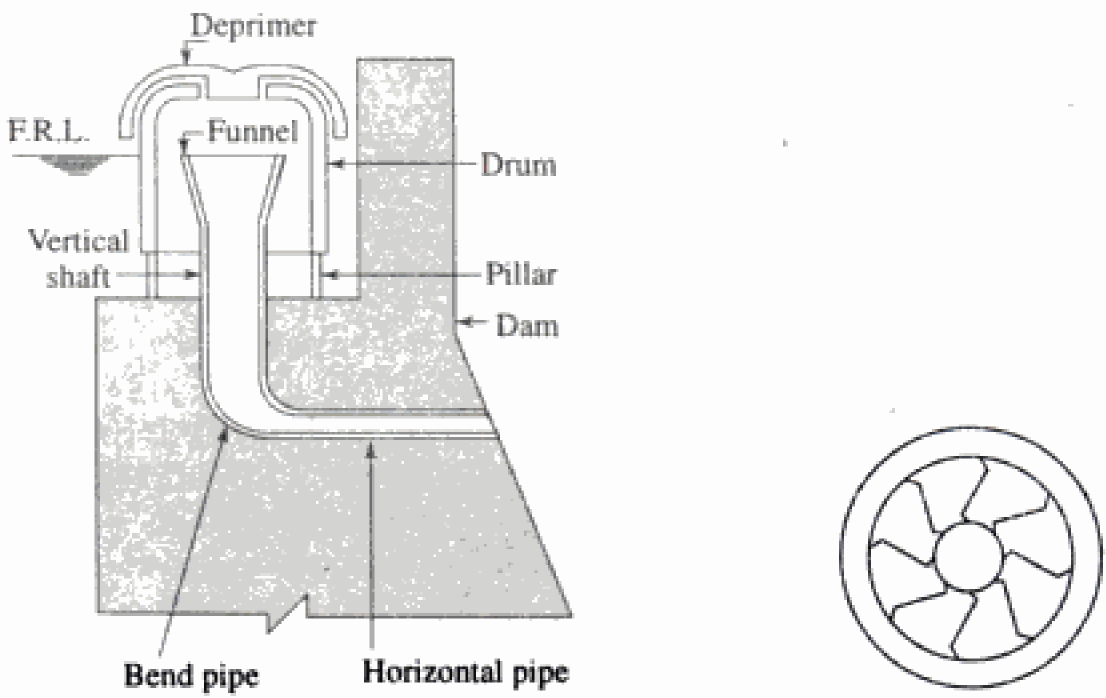
The shape of the lower nappe is not same for all the head of water above the crest of the weir. It differs with the head of water. But for the design of the ogee spill way the maximum head is considered. If the spill way runs with the maximum head, then the overflowing water just follows the curved profile of the spill way and there is no gap between the water and the spill way surface and the discharge is maximum.

When the actual head becomes more than the designed head, the lower nappe does not follow the ogee profile and gets separated from the spill way surface. Thus a negative pressure develops at the point of separation. Due to the negative

is connected to a horizontal pipe which carries the flowing water away from the base of the dam. The top level of the funnel is kept just at the full reservoir level. The funnel consists of several volutes (curved vanes or blades). Thus the water has a spiral motion while passing through the funnel. A circular drum is placed over the funnel. The drum is supported on pillars. Its bottom end is completely open and the top end consists of a small opening, which acts as air inlet. The water enters the drum through the bottom. A deprimer is provided over the top opening of the drum leaving on air space between the two.

When the water rises above the full reservoir level, it spills over the circumference of the funnel and flows with a spiral motion through it. Thus a vortex is formed in the vertical shaft. This vortex induces a suction pull and a vacuum is created inside the drum. Thus, the siphonic action is started and the shaft starts running full.

When the water comes back to the full reservoir level, the air enters the drum through the space between the drum and deprimer and the siphonic action is stopped automatically (Fig. 12.9 a and b).



(a) Section of siphon (b) Plan of funnel

Fig. 12.9 Volute siphon spill way

Chute or Trough Spill Way

This spill way is simply a rectangular open channel or trough (known as chute) provided on the dam to discharge the surplus water from the reservoir to the same river on the downstream side. The spill way may be provided along the abutment of the dam or along the edge of the reservoir at the full supply level. The chute is constructed by joining pre-cast R.C.C. channels in a longitudinal slope of 1 in 4 or 1 in 6. The channels are supported on pillars. The section of the channel is designed according to the volume of surplus water or flood discharge. This spill way may be provided at one side or both sides of the dam. Apron should be provided at the downstream end of the chute. The apron is made of boulder pitching with cement grouting (Fig. 12.10 a and b).

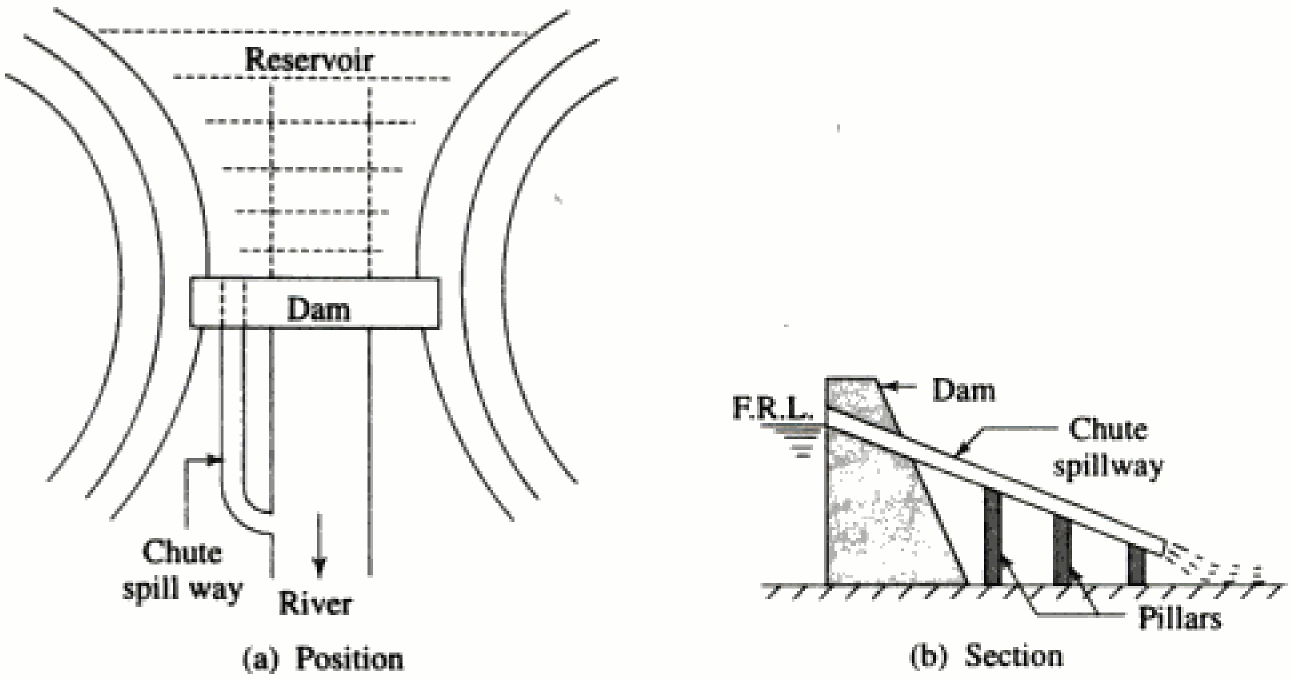


Fig. 12.10 Chute spill way

Shaft Spill Way

It consists of a vertical shaft which is constructed with masonry work or plain cement concrete or reinforced cement concrete on the bed of the reservoir just at the upstream side of the dam. The inlet mouth of the vertical shaft is conical shaped. The vertical shaft is connected with horizontal shaft. The horizontal shaft again may be taken through the body of the dam (in case of gravity dam) or through the base of the dam (in case of earthen dam) or may be connected to a tunnel outside the dam. The inlet mouth is kept at the normal pool level of the reservoir. So, when the water rises above the N.P.L. it enters the shaft from all directions and flows out through the shaft. In order to arrest the floating debris, a net protection is provided on the inlet mouth (Fig. 12.11 a and b).

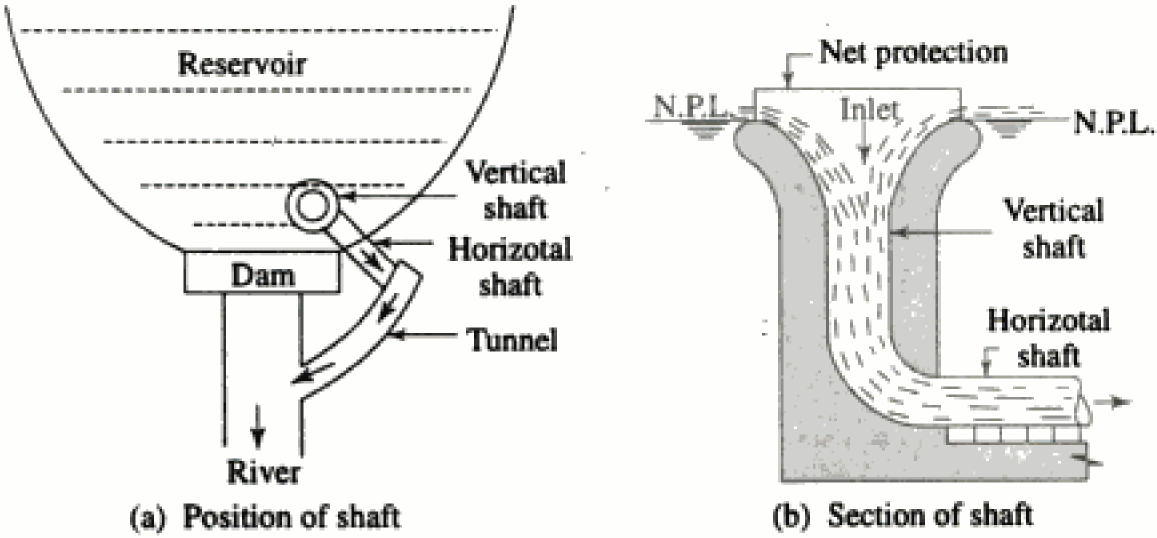


Fig. 12.11 Shaft spill way

Side Channel Spill Way

The side channel spill way is completely separate from the main body of the dam. The spill way is constructed at right angle to the dam and at any side according to the site condition. The crest of the spill way is kept at the normal pool level of the reservoir. When the water rises above the N.P.L. it spills over the crest of the spill way and flows through the side channel and ultimately meets the same river on the downstream side. This type of spill way is recommended for the sites where other types of spill ways are found unsuitable. The side walls of the channel may be constructed with brick masonry or stone masonry. The longitudinal slope of the channel depends on the available space or length (Fig. 12.12).

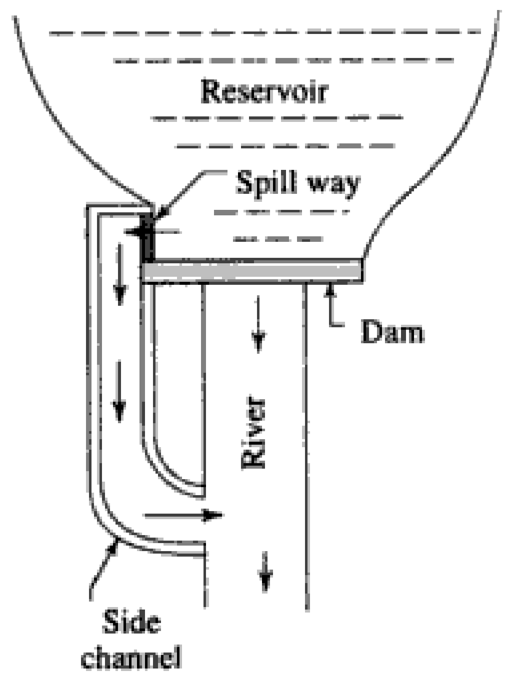


Fig. 12.12 Side channel spill way

12.5 ENERGY DISSIPATION

Definition When water spills and flows over the spillways, then it acquires a very high velocity, as the whole potential energy (due to potential head) is transformed into kinetic energy. The process of destruction of this kinetic energy is known as energy dissipation.

Purpose of Energy Dissipation The high velocity of flow over the spill way has a tendency to erode the bed of the river at the base of the spill way and thus the stability of the spill way may be in danger by forming piping or undermining. Hence, to destroy the kinetic energy and to resist the erosion of the river bed, some devices are adopted which are known as energy dissipators.

Types of Energy Dissipator The following are the different types of energy dissipator,

1. Hydraulic jump with stilling basin The hydraulic jump is defined as a turbulent passage of water from super-critical to sub-critical state. The hydraulic jump forms the turbulent roller which causes the dissipation of kinetic energy. This is the most suitable method as the energy loss is due to the impact of water against water.

For the formation of hydraulic jump a minimum depth of water is always required at the toe of the spill way. When the depth of water is insufficient to form the jump, the stilling basin is formed by horizontal apron and low obstruction. The basin acts as a water cushion, and it gives the requisite depth for the formation of jump (Fig. 12.13a).

When the depth of tail water is more then that required to create the jump, the sloping apron is provided (Fig. 12.13b).

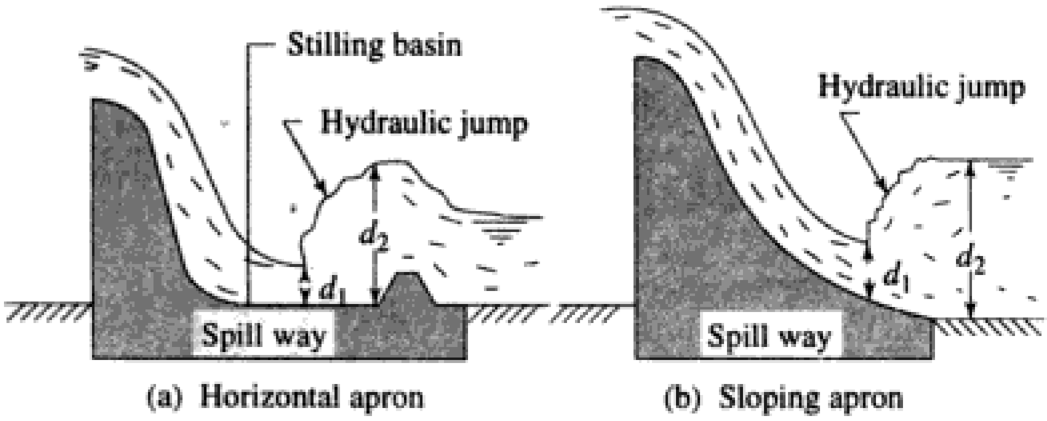


Fig. 12.13 Hydraulic jump

2. Buckets The following buckets are generally provided for the dissipation of energy.

(a) Solid roller bucket It is a solid bucket with concave circular profile and a deflector lip. When water flows over the bucket, the sheet of water is deflected by the lip and two rollers are formed. One roller moves in anticlockwise direction on the surface of the bucket which is known as bucket roller. The other roller moves in clockwise direction on the downstream side of the bucket which is known as ground roller. The movement of these two rollers cause the dissipation of energy. The solid roller bucket is generally provided when the depth of tail water is too large as compared to the depth required for the formation of hydraulic jump (Fig. 12.14).

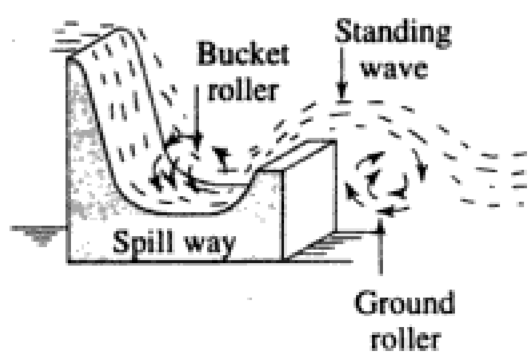


Fig. 12.14 Solid roller bucket

(b) Slotted roller bucket The slotted roller bucket is similar to solid bucket in shape, the only difference is that it consists of slotted deflector lip. This type of bucket is suitable at low tail water. In this case also the bucket roller and ground roller are formed which are effective in destroying the kinetic energy. The advantage of this type of bucket is that the debris which might get into the bucket is washed out through the slots (Fig. 12.15).

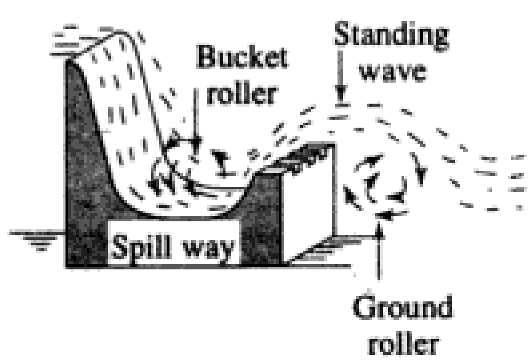


Fig. 12.15 Slotted roller bucket

(c) Ski-Jump Bucket The ski-jump bucket is suitable when the tail water is less than the depth required for the formation of hydraulic jump. Here, the shape of the bucket is such that the lip deflects the sheet of water in the form of free jet which falls in the river bed at a safe distance away from the spill way. For precaution, the river bed should be protected by boulder pitching at the zone of outfall of water jet (Fig. 12.16).

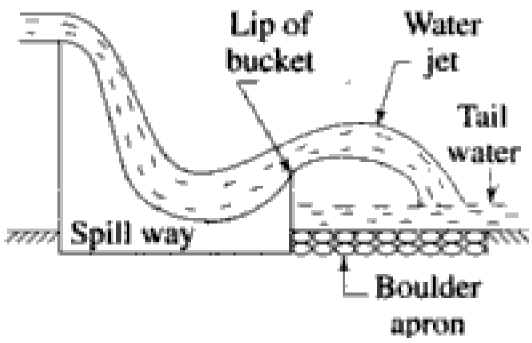


Fig. 12.16 Ski-jump bucket

Condition for the Formation of Hydraulic Jump Let H = height of spill way crest, h_e = potential energy head, V_1 = velocity before hydraulic jump, V_2 = velocity after hydraulic jump, d_1 = depth of water before jump, d_2 = depth of water after jump, h_L = head lost, q = discharge per unit length, c = coefficient of discharge

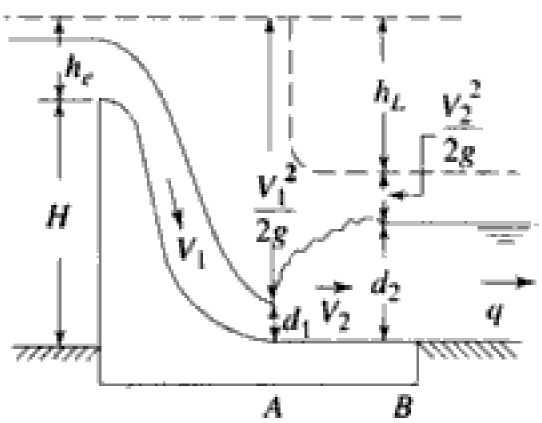


Fig. 12.17 Condition for hydraulic jump

Refer Fig. 12.17

We know, unit discharge, $q = C\,(h_e)^{3/2}$

or $$h_e = \left(\frac{q}{c}\right)^{2/3}$$

The total energy of flow (E) is given by, $E = H + h_e$

The total energy is same at all sections. Considering section A and B.

$$E = d_1 + \frac{V_1^2}{2g} = d_2 + \frac{V_2^2}{2g} + h_L$$

The conjugate depths d_1 and d_2 are related by,

$$d_2 = -\frac{d_1}{2} + \sqrt{\frac{2q^2}{g d_1} + \frac{d_1^2}{4}}$$

Again, the critical depth (d_c) is given by the relation, $d_c = \left(\frac{q^2}{g}\right)^{1/3}$

Now the condition for the formation of hydraulic jump is, (a) $d_1 < d_c$, (b) $d_2 > d_c$

The formation of hydraulic jump is also indicated by Froude number (F).

The number is given by,

$$F = \frac{V}{\sqrt{gD}}$$

where, V = velocity of flow, D = depth of flow.

If F_1 be the Froude number at the beginning of hydraulic jump, then conjugate depths d_1 and d_2 are related as

$$d_2 = \frac{d_1}{2}\left[\sqrt{1 + 8F_1^2} - 1\right]$$

Length of jump = 5 $(d_2 - d_1)$.

when F_1 varies from 4.5 to 9, the hydraulic jump is called steady jump and this jump has the best performance.

Review Questions

1. Fill up the blanks with appropriate word/words.
   - (i) The opening provided at the body of the dam to discharge excess water is known as ________.
   - (ii) The water is stored in a storage reservoir up to ________.
   - (iii) For weirs or low dams ________ spill way is suitable.
   - (iv) When the surface of the spill way is made to coincide with the shape of the lower nappe of free falling water jet, then it is known as ________.
   - (v) The priming of the saddle siphon spill way is done by ________.
2. Why are spill ways provided in a dam?
3. Distinguish between drop spill way and ogee spill way.
4. Describe an ogee spill way with a neat sketch.
5. Describe volume siphon spill way with a neat sketch.

Answers

1. (i) Spillways (ii) Normal pool level
   (iii) drop (iv) Ogee spill way
   (v) baby siphon

13

CROSS-DRAINAGE WORKS

13.1 INTRODUCTION

In an irrigation project, when the network of main canals, branch canals, distributories, etc are provided, then these canals may have to cross the natural drainages like rivers, streams, nallahs, etc at different points within the command area of the project. The crossing of the canals with such obstacles cannot be avoided. So, suitable structures must be constructed at the crossing point for the easy flow of water of the canal and drainage in the respective directions. These structures are known as cross drainage works. But the nature of cross drainage works may be different at different places. Sometimes, the bed level of canal may be below the bed level of drainage and sometimes, it may be higher than that of the drainage. The bed levels of canal and drainage may be nearly same also. So the structures are different at different places and the designation of the structures also are different. The details of these various structures will be dealt with later on.

13.2 NECESSITY OF CROSS-DRAINAGE WORKS

The following factors justify the necessity of cross drainage works,

(a) The water shed canals do not cross natural drainages. But in actual orientation of the canal network, this ideal condition may not be available and the obstacles like natural drainages may be present across the canal. So, the cross drainage works must be provided for running the irrigation system.

(b) At the crossing point, the water of the canal and the drainage get intermixed. So, far the smooth running of the canal with its design discharge the cross drainage works are required.

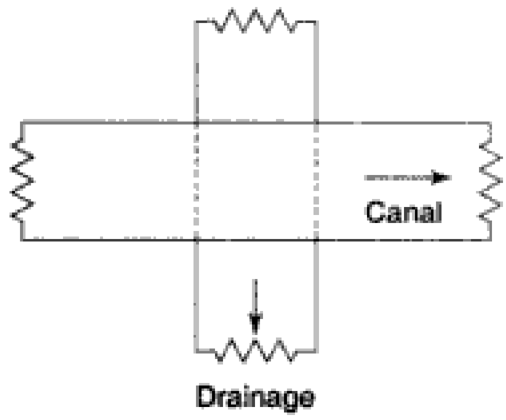
(c) The site condition of the crossing point may be such that without any suitable structure, the water of the canal and drainage cannot be diverted to their natural directions. So, the cross drainage works must be provided to maintain their natural direction of flow.

13.3 TYPES OF CROSS-DRAINAGE WORKS

According to the relative bed levels, maximum water levels and relative discharges of the canals and drainages the cross drainage works may be of the following types,

Type–I Irrigation Canal Passes Over the Drainage This condition involves the construction of following:

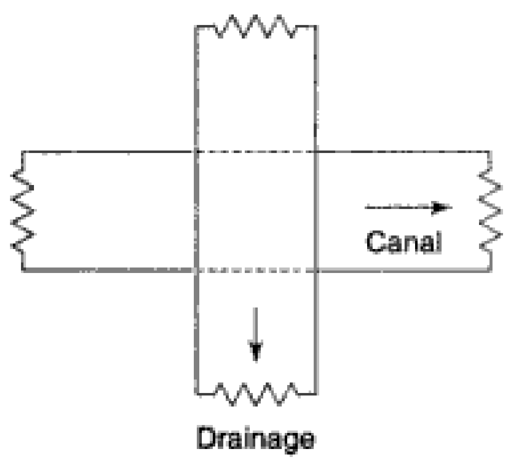
(a) Aqueduct The hydraulic structure in which the irrigation canal is taken over the drainage (such river, stream, etc.) is known as aqueduct. This structure is suitable when bed level of canal is above the highest flood level of drainage. In this case, the drainage water passes clearly below the canal.



(b) Siphon Aqueduct In a hydraulic structure where the canal is taken over the drainage, but the drainage water cannot pass clearly below the canal. It flows under siphonic action. So. it is known as siphon aqueduct. This structure is suitable when the bed level of canal is below the highest flood level of the drainage.

Type–II Drainage Passes Over the Irrigation Canal This condition involves the construction of the following:

(a) Super Passage The hydraulic structure in which the drainage is taken over the irrigation canal is known as super passage. the structure is suitable when the bed level of drainage is above the full supply level of the canal. The water of the canal passes clearly below the drainage.



(b) Siphon Super Passage The hydraulic structure in which the drainage is taken over the irrigation canal, but the canal water passes below the drainage under siphonic action is known as siphon super passage. This structure is suitable when the bed level of drainage is below the full supply level of the canal.

Type–III Drainage and Canal Intersection Each Other at the Same Level This condition involves the construction of the following:

(a) Level Crossing When the beds of the drainage and canal are practically at the same level, then a hydraulic structure is constructed which is known as level crossing. This is suitable for the crossing of large drainage with main canal.

13.5 AQUEDUCT

The aqueduct is just like a bridge where a canal is taken over the deck supported by piers instead of a road or railway. Generally, the canal is in the shape of a rectangular trough which is constructed with reinforced cement concrete. Sometimes, the trough may be of trapezoidal section. An inspection road is provided along the side of the trough. The bed and banks of the drainage below the trough is protected by boulder pitching with cement grouting. The section of the trough is designed according to the full supply discharge of the canal. A free board of about 0.50 m should be provided. The height and section of piers are designed according to the highest flood level and velocity of flow of the drainage. The piers may be of brick masonry, stone masonry or reinforced cement concrete. Here, deep foundation (like well foundation) is not necessary for the piers. The concrete foundation may be done by providing the depth of foundation according to the availability of hard soil.

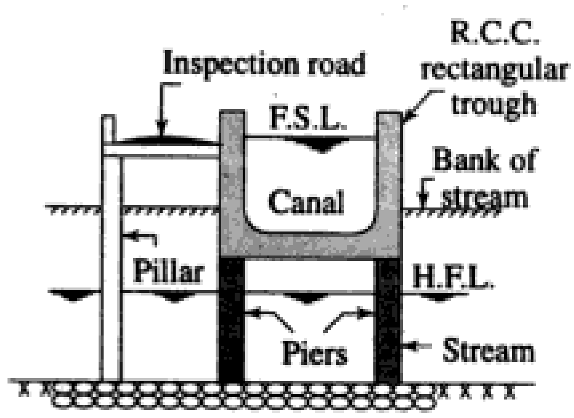


Fig. 13.1 Aqueduct

13.6 SIPHON AQUEDUCT

The siphon aqueduct, the bed of the drainage is depressed below the bottom level of the canal trough by providing sloping apron on both sides of the crossing. The sloping apron may be constructed by stone pitching or cement concrete. The section of the drainage below the canal trough is constructed with cement concrete in the form of tunnel. This tunnel acts as a siphon. Cut off walls are provided on both sides of the apron to prevent scouring. Boulder pitching should be provided on the upstream and downstream of the cut-off walls. The other components like canal trough, piers, inspection road, etc. should be designed according to the methods adopted in case of aqueduct (Fig. 13.2).

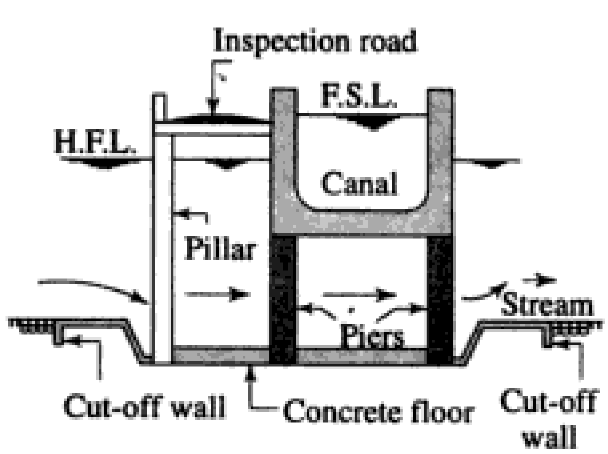


Fig. 13.2 Siphon aqueduct

(3) Canal regulator It is provided across the canal just at the downstream side of the crossing point. This regulator also consists of adjustable shutters at different tiers.

Operation In dry season, when the discharge of the drainage is very low, the drainage regulator is kept closed and the canal water is allowed to flow as usual. In rainy season, when the discharge of the drainage is very high, the drainage regulator is kept completely open and the canal regulator is adjusted according to requirement. The level crossing is recommended for the crossing of main canal with large drainage (Fig. 13.5).

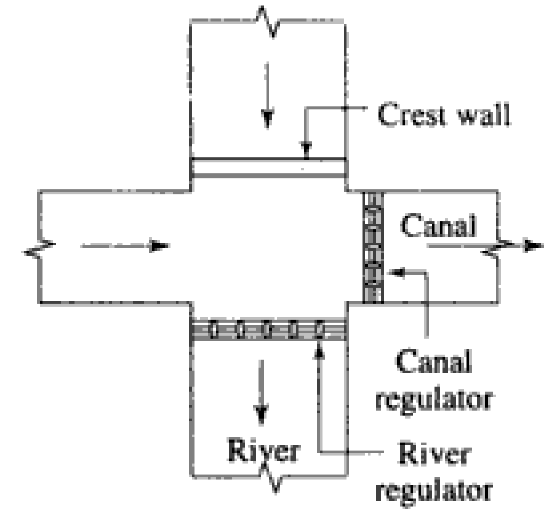


Fig. 13.5 Level crossing

13.10 INLET AND OUTLET

In case of crossing of a small irrigation channel with a small drainage, no hydraulic structure is constructed. Because, the discharges of the drainage and the channel are practically low and these can be easily tackled by easy system like inlet and outlet arrangement. In this system an inlet is provided in the channel bank simply by open cut and the drainage water is allowed to join the channel. Then at a suitable point on the down stream side of the channel an outlet is provided by open cut and the water from the irrigation channel is allowed to flow through a leading channel towards the original course of the drainage. At the points of inlet and outlet the bed and banks of the drainage are protected by stone pitching. The bed and banks of the irrigation channel between inlet and outlet points should also be protected by stone pitching (Fig. 13.6).

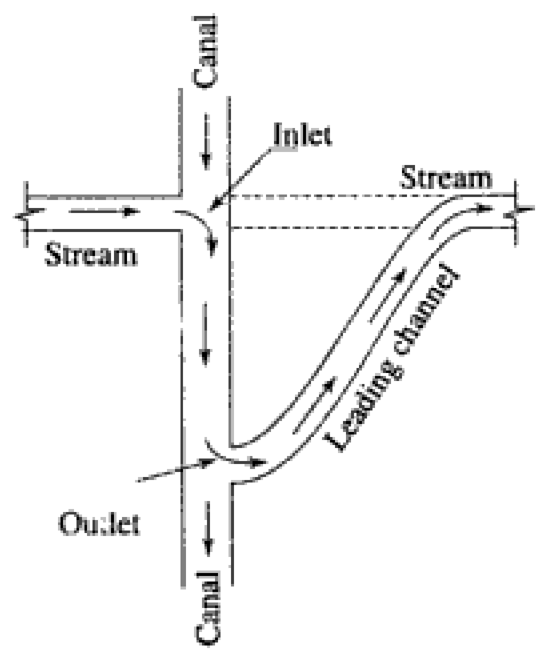


Fig. 13.6 Inlet and outlet

REVIEW QUESTIONS

1. (i) When a canal crosses a river __________ works are done at the crossing point.

14

CANAL FALLS

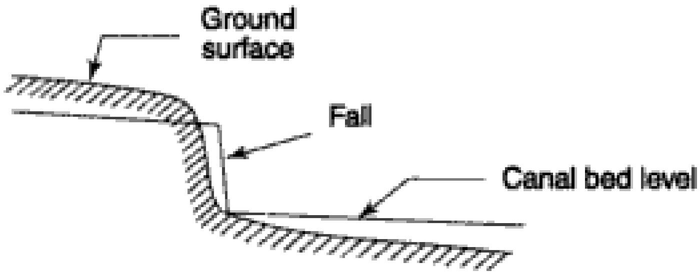
14.1 INTRODUCTION

Irrigation canals are constructed with some permissible bed slopes so that there is no silting or scouring in the canal bed. But it is not always possible to run the canal at the desired bed slope throughout the alignment due to the fluctuating nature of the country slope. Generally, the slope of the natural ground surface (i.e. country slope) is not uniform throughout the alignment. Sometimes, the ground surface may be steep and sometimes it may be very irregular with abrupt change of grade. In such cases, a vertical drop is provided to step down the canal bed and then it is constinued with permissible slope until another step down is necessary. This is done to avoid unnecessary huge earth work in filling. Such vertical drops are known as canal falls or simply falls.

14.2 NECESSITY OF CANAL FALLS

The canal falls are necessary in case the following conditions occur:

(a) When the slope of the ground suddenly changes to steeper slope, the permissible bed slope cannot be maintained. It requires excessive earthwork in filling to maintain the slope. In such a case canal falls are provided to avoid excessive earth work in filling (Fig. 14.1(a)).



(a)

Fig. 14.1(a)

(b) When the slope of the ground is more or less uniform and the slope is greater than the permissible bed slope of canal. In that case also the canal falls are necessary (Fig. 14.1(b)).

(c) In cross-drainage works, when the difference between bed level of canal and that of drainage is small or when the F.S.L. of the canal is above the bed level of drainage then the canal fall is necessary to carry the canal water below the stream or drainage (i.e. in case of siphon super passage) (Fig. 14.1(c)).

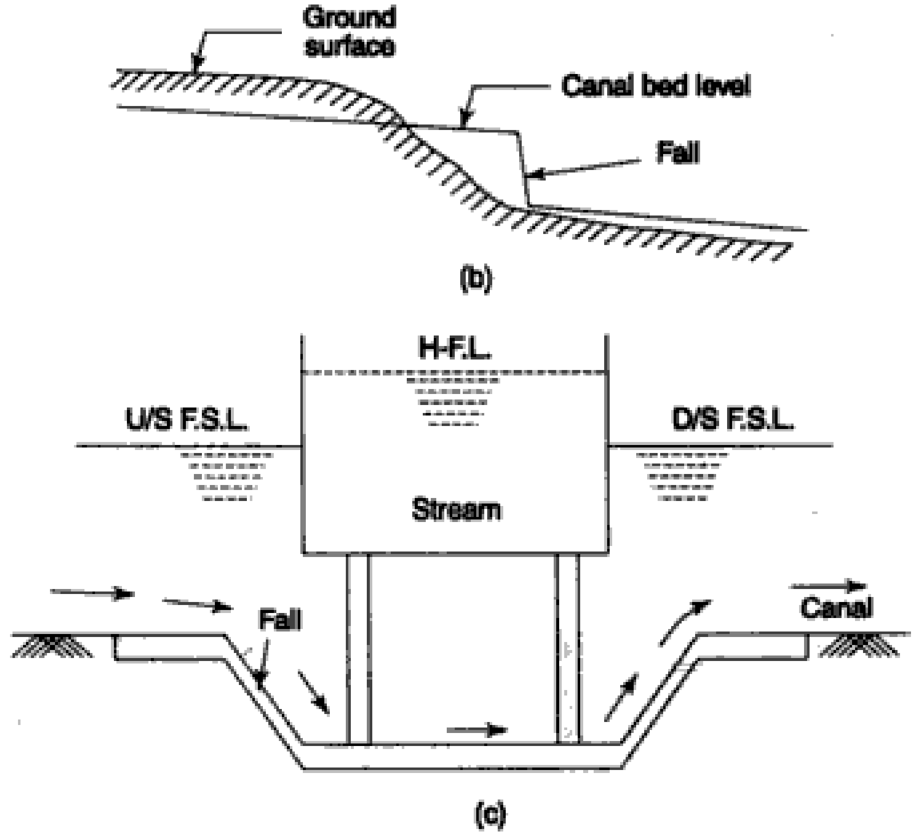


Fig. 14.1 (b) and (c)

14.3 TYPES OF CANAL FALLS

The following are the different types of canal falls that may be adopted according to the site condition.

Ogee Fall

In this type of fall, an ogee curve (a combination of convex curve and concave curve) is provided for carrying the canal water from higher level to lower level. This fall is recommended when the natural ground surface suddenly changes to a steeper slope along the alignment of the canal. The fall consists of a concrete vertical wall and concrete bed. Over the concrete bed the rubble masonry is provided in the shape of ogee curve (inverted oblong 'S'). The surface of the masonry is finished with rich cement mortar (1:3). The upstream and

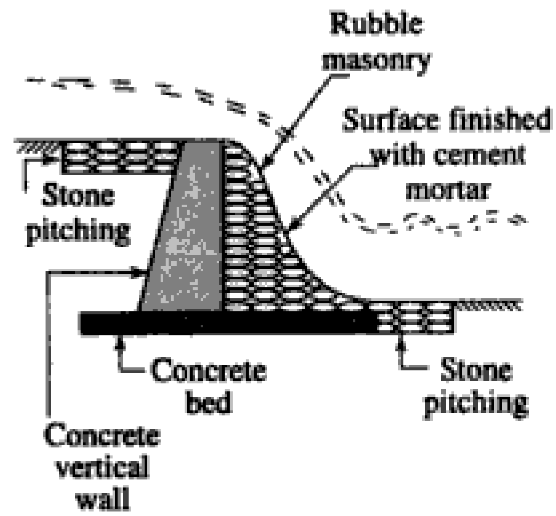


Fig. 14.2 Ogee fall

impervious floor is provided to resist the scouring effect of the falling water. The upstream and downstream side of the fall is protected by stone pitching finished by cement grouting. The size and number of notches depends upon the full supply discharge of the canal (Fig. 14.5).

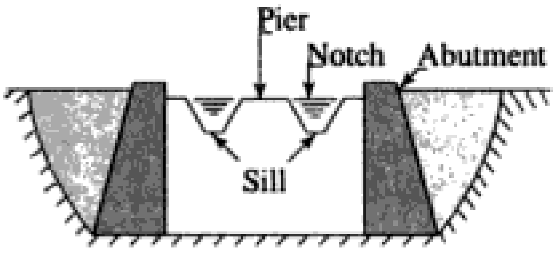


Fig. 14.5 Trapezoidal notch fall

Vertical Drop Fall or Sarda Fall

It consists of a vertical drop wall which is constructed with masonry work. The water flows over the crest of the wall. A water cistern is provided on the downstream side which acts as a water cushion to dissipate the energy of falling water. A concrete floor is provided on the downstream side to control the scouring effect of the flowing water. Curtain walls are provided on the upstream and downstream side. Stone pitching with cement grouting is provided on the upstream and downstream side of the fall to protect it from scouring. This type of falls were provided on the Sarda canal in Utter Pradesh. Hence, it is sometimes known as Sarda fall (Fig. 14.6).

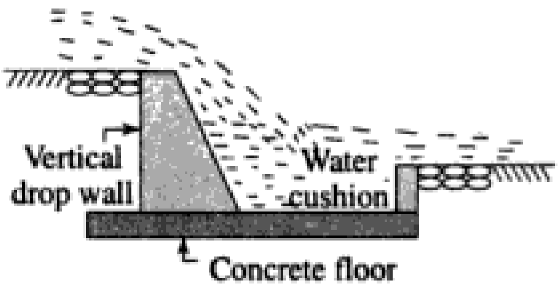


Fig. 14.6 Vertical drop fall

Glacis Fall

It consists of a straight sloping glacis provided with a crest. A water cushion is provided on the downstream side to dissipate the energy of flowing water. The sloping glacis is constructed with cement concrete. Curtain walls and toe walls are provided on the upstream and downstream side. The space between the toe walls and curtain walls is protected by stone pitching. This type of fall is suitable for drops up to 1.5 m (Fig. 14.7). For the improvement in energy dissipation, the glacis falls have been modified as follows.

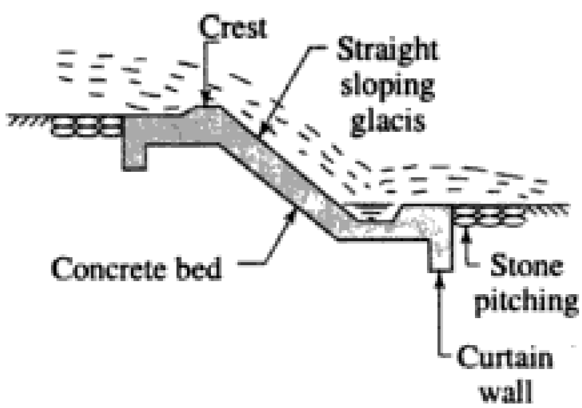


Fig. 14.7 Glacis fall

(a) Montague Type Fall In this type of fall, the straight sloping glacis is modified by giving parabolic shape which is known as Montague profile. Taking '0' as the origin, the Montague profile is given by the equation,

$$X = v\sqrt{\frac{4y}{g}} + y,$$

where, x = distance of point P from OX axis, Y = distance of point P from OY axis, V = velocity of water at the crest, g = acceleration due to gravity.

The main body of the fall is constructed with cement concrete. Toe walls and curtain walls are same as in the case of straight sloping glacis. The bed protection by stone pitching is also same (Fig. 14.8).

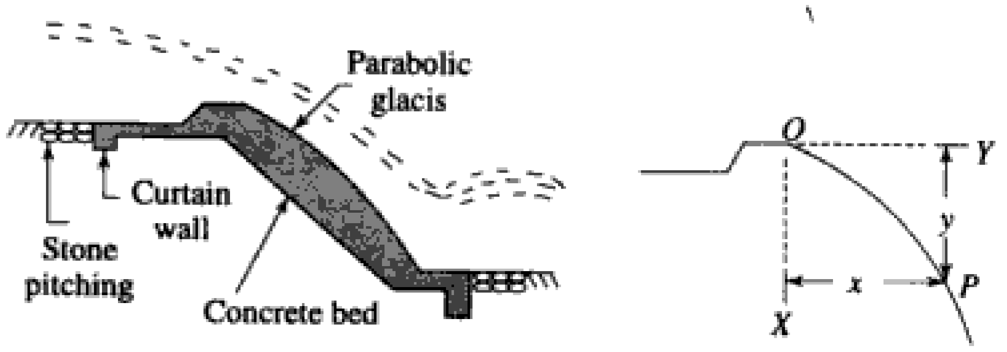


Fig. 14.8 Montague type fall

(b) Inglis Type Fall In this type of fall, the glacis is straight and sloping, but buffle walls are provided on the downstream floor to dissipate the energy of flowing water. The height of buffle depends on the head of water on the upstream side. The main body of the fall is constructed with cement concrete. The toe walls and curtain walls are same as straight glacis. The protection works with stone pitching are also same. Sometimes, this fall is known as buffle fall (Fig. 14.9).

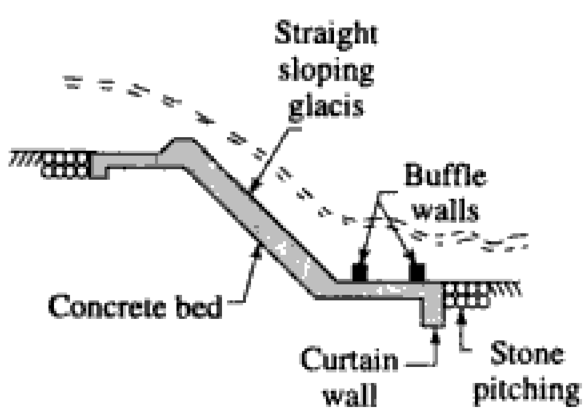


Fig. 14.9 Inglis type fall

REVIEW QUESTIONS

1. Fill up the blanks with appropriate word/words.
   (i) Along the alignment of any irrigation canal if the ground surface consists of abrupt change of grade, then _________ should be provided.
   (ii) When the slope of the natural ground surface is even and long, then _________ fall is provided in an irrigation canal.
   (iii) To dissipate the energy of falling water a _________ is provided in vertical drop fall.
   (iv) Montague type fall is a special type of _________ fall.
   (v) Vertical drop fall is also known as _________ fall.
2. What is a canal fall? Why is it necessary?
3. Name the different types of falls, and state the suitability of their application.
4. Describe an ogee fall and rapid fall with neat sketch.
5. Distinguish between Montague fall and Inglish fall with sketch.

15

INLAND NAVIGATION

15.1 INTRODUCTION

The communication or the transport of goods through the perennial rivers or big canals is known as inland navigation. In ancient days, the rivers were the main source of communication system between the cities or towns (known as nagar) which developed by the banks of big rivers. The different types of goods were transported from one place to other by the merchants. In present days, the highways, railways and airways play an important role in communication and transportation of goods. But still, the inland navigation systems are found necessary for various reasons.

In British period and just after independence, the waterway communication from Bengal to Assam was established via Calcutta port, Bay of Bengal and Chittagang port (Chattagram). But during the period 1950–60, the Government of Pakistan occasionally closed the said route for various reasons. The Government of India worried about the matter decided to open an inland nagivation route from Farakka (West Bengal) to Dhubri (Assam). In 1959 the Central Water and Power Commission took up a project titled "Ganga—Bramhaputra navigation cum irrigation project". Farakka barrage, Tista barrage and Mahananda barrage were the component parts of this project. The project report was prepared after long investigation works which was accepted by the government and the constructional work got started. Now, the Farakka barrage and Tista barrage have been completed. The Mahananda barrage and the main canal are now under construction. The navigation through the rivers Ganga, Bramhaputra, Godavari, Krishna, Kaveri, etc. are on at present. The navigation canals like Godavari and Krishna canal, Orissa coast canal, Ganga canal from Hardwar to Kanpur are also in use at present.

However, the following are the disadvantages of inland navigation,

(a) The transportation of goods through the waterway requires much time as the motion of the water craft is slow in comparison to the other communication system, i.e. roads, railways.

(b) It is limited to transport goods to the towns or cities by the bank of the river or canal only.
(c) Large area cannot be covered by this system.
(d) Occasional dredging is necessary to maintain the required depth of water in the river.
(e) Lock gates on the river or canal waste much time for the movement of the water craft like barge, steamer, etc. from upstream to downstream and vice versa.

15.2 REQUIREMENTS OF NAVIGABLE WATER WAYS

The following are the requirements of the navigable water ways,

1. Depth of Water The depth of water in the waterway, i.e. river or canal is the primary consideration for navigation. For navigation the minimum depth of water should be 3 m to 4 m. Though the water flows in the perennial river throughout the year the depth of water varies according to the season. In summer and winter the depth of water may not be sufficient for the water craft. So, to maintain the required depth of water sometimes the construction of barrage may be necessary. However, the depth of water may be ascertained by the following consideration:

(a) The type and capacity of water craft which is to be allowed in the waterway.
(b) The immerged depth of water craft when fully loaded.

2. Width of Water Way The minimum width of the water way should be 80 m. The width should be decided according to the size and volume of water craft recommended for navigation purposes. The width also depends on the wave action which is generated by the moving ship or steamer. The wave action causes the erosion of the bank of the waterway. It is a common theory that lesser the width, the more will be the effect of wave action. Again, the width should be such that it should not cause any difficulty in the up and down movement of water craft.

3. Alignment The water way should not consist of many zig-zag course, sharp bends, hair pin bends, etc. These may cause difficulties in the movement of water craft. In such cases, cut-off channels should be provided to make the water way straight or some suitable measure should be taken to overcome such positions.

4. Lock Gates To maintain the required depth of water, sometimes barrages may be constructed in some places. For the movement of the steamers across the barrage, the lock gates should be provided with the barrage. In case of navigation canal, when the canal fall cannot be avoided, the lock gate has to be provided. In navigation canal, the cross-drainage work like super passage cannot be provided. The level crossing system is always preferred.

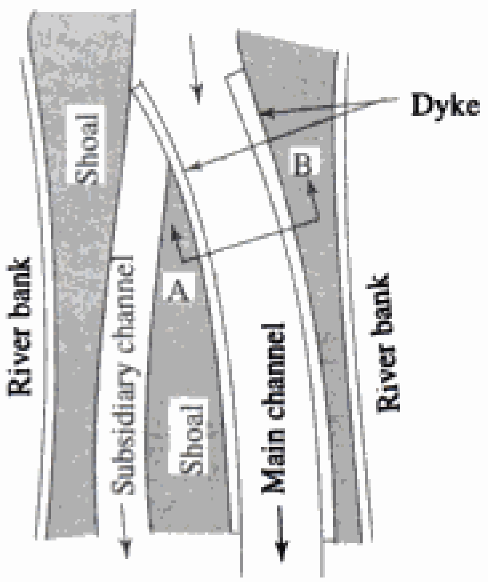
5. Over Bridges When road or railway bridges are constructed across the navigable waterway, the optimum height should be kept above the highest flood

occasional dredging becomes necessary to maintain the navigability of the river. The dredging may be of the following types,

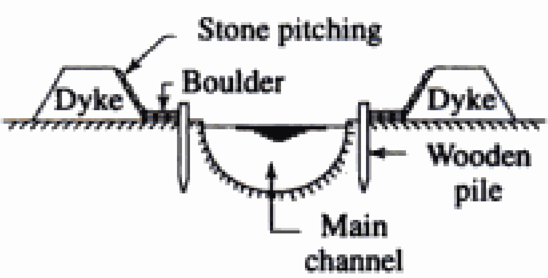
(i) Mechanical Dredging This method consists of an endless chain of buckets which lift the sediments from the bottom to the surface. The buckets are carried by conveyor belt and the materials are discharged at the stack yard continuously. The mechanical dredging may be done by dipper dredging and ladder dredging.

(ii) Hydraulic Dredging In hydraulic dredging the sediment and water (in the form of slurry) are drawn by suction pipe and the mixture is discharged through the delivery pipe of a pump to the spoil area. The hydraulic dredging may be done by a dustpan dredger and cutter head dredger.

(c) Contraction Works When a river is very wide with shallow section and a number of bifurcated shallow channels are developed, then contraction works may be recommended to form a narrower, deeper and well defined channel section through which navigation may be possible. The contraction works consists of the construction of dykes on both sides of the deeper channel. This is to guide the flow of water through the developed channel. On the river side of the dyke, the wooden piles are driven in two rows with their heads projected about 1 m above the ground level. A box like compartment is formed by fixing wooden battens with the piles. The box is filled with boulders. The section of the dyke depends on the highest flood level is the river (Fig. 15.1).



(a) Bifurcated channel



(b) Section on A B

Fig. 15.1 Contraction works

(d) Bank Stabilisation At the zone of river bend, the concave side is liable to be affected by scouring and the formation of shoal takes place on the other side. The affected bank should be made stable by protection works. The bank stabilisation consists of wooden piling or sheet piling along the bottom of the

3. Canalization Method

In this method, a link canal is constructed between two points of the same navigable river to shorten the route. A barrage is constructed at the take-off point to regulate the water level in the link canal. A lock gate should be provided at the head of the link canal. Sometimes, a feeder canal may be taken to maintain the navigability of a port for example, feeder canal of Farakka barrage. The width and depth of the canal depends on the water craft that will be allowed in the canal. Such canal should always be lined and the velocity of flow should be critical so that no sitting can take place in the canal bed (Fig. 15.5).

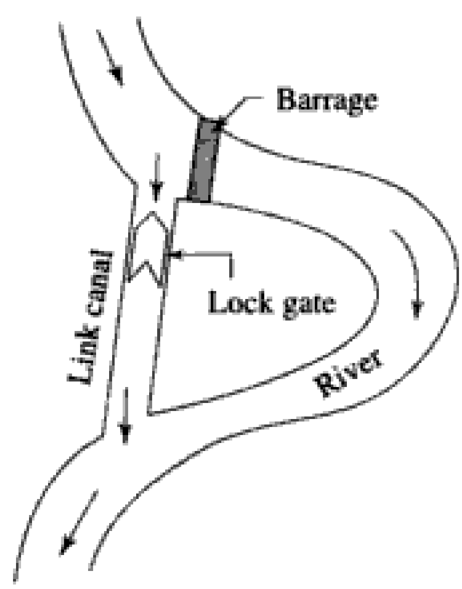


Fig. 15.5 Link canal

15.5 METHODS OF CANAL NAVIGATION

In navigation canal system, two or more big perennial rivers are inter linked by canals to transport agricultural or commercial goods from one state to other. As for example, The Ganga-Bramhaputra navigation cum irrigation project is meant for connecting three states Assam, West Bengal and Behar via Tista barrage and Farakka barrage. The following are the component parts of the navigation canal system (Fig. 15.6).

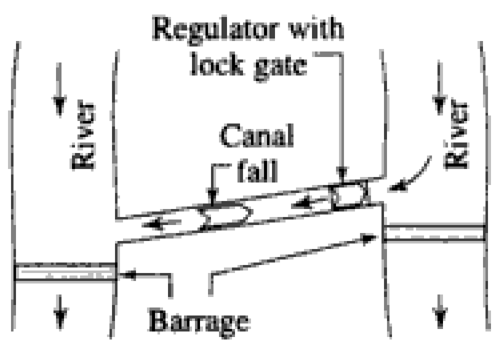


Fig. 15.6 Navigation canal

(a) Barrage This is constructed to form a water pool infront of the canal head so that the required depth of water can be maintained in the canal throughout the year.

(b) Head Regulator with Lock Gate Head regulator is provided at the head of the canal so that the water level can be regulated. The lock gate should be provided with the head regulator for the movement of the ships across the regulator.

(c) Lock Gate at Canal Fall In place of canal fall, the lock gate must be provided for the easy movement of the ships from upstream to downstream and vice-versa.

(d) Level Crossing Where cross-drainage work becomes unavoidable, the level crossing system is preferred. The alignment of the navigation canal should be in such a way that the condition for the level crossing is available.

(e) Over Bridges In places of road or railway crossings the over bridges should be constructed at a height, so that the required clearance is available below the bridge for the movement of the ships.

The width of the navigable canal should be sufficient for the ships to move freely. Generally, the width varies from 80 m to 100 m.

The depth of the canal should be sufficient for a loaded ship, free movement. Generally, the depth varies from 4 m to 6 m. But the depth is fully dependant on the carrying capacity of the ships and the submerged depth of the fully loaded ship.

The navigation canal should always be lined and the bed slope should be such that the velocity of flow is maintained. This means, there should be no silting in the canal bed.

15.6 NAVIGATION LOCKS OR LOCK GATES

The movable gates (horizontal swing) which are provided with the barrage or head regulator or fall, for the movement of the water craft through the navigation canal are known as navigation locks or lock gates. The following are the main criteria which govern the design of navigation lock,

(a) Depth The overall depth of the lock chamber should be equal to:

$$D = \text{Max. difference of pool elevation between u/s and } D/S + \text{required draft} + \text{free board.}$$

The bottom level of the lock chamber is fixed with respect to the downstream pool level and the required draft. The draft is the optimum depth of water for the movement of the ship.

(b) Width The width of the lock gate depends on the width of vessels that will pass through the lock chamber. Generally, the width varies from 50 m to 100 m.

(c) Length The length of the lock chamber also depends on the maximum length of the vessels which are allowed to cross. Generally, the length varies from 200 m to 300 m.

(d) Filling and emptying system At the time of transit operation, the lock gates are not opened instantaneously, because this will lead to tremendous velocity of flow and abrupt waves across the lock chamber which may cause damage to the vessel, lock chamber and other components. Hence, some filling and emptying device are employed with the lock chamber. Generally, tunnels or conduits are provided on one side or both sides of the lock chamber with inlet and outlet valve. The tunnels or conduits consist of number of openings corresponding to the lock chamber which are meant for filling the chamber. The tunnels or conduits should be designed in such a way so that the lock chamber can be filled quickly to avoid unusual delay in crossing.

Components of Navigation Lock The following are the main components of navigation lock,

(1) Lock gates Two lock gates are provided at the two ends of the lock chamber. The gates consist of pairs of shutters which are pivoted at one end and the other ends are free to move towards each other. The opening and closing of the shutters is done by a mechanical device. When the shutters are closed, they form an angle of 120°. For opening and closing the shutters, the movement is shown by arrows (Fig. 15.7).

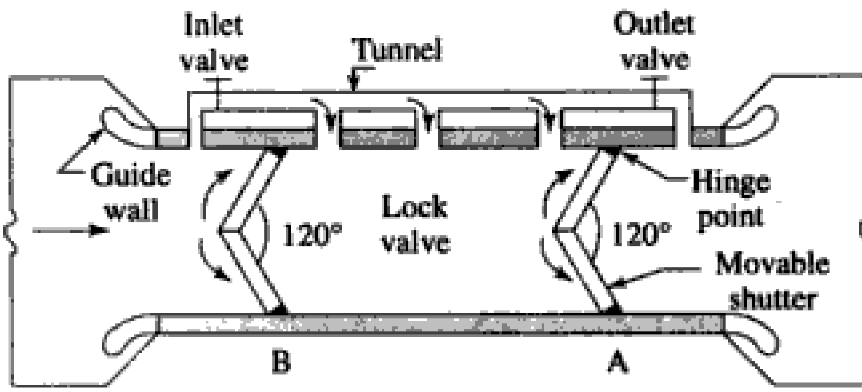


Fig. 15.7 Navigation lock

(2) Lock chamber The compartment between the upstream and downstream lock gates is known as the lock chamber. It is a rectangular chamber where the vessels are detained for sometime during crossing the lock gates.

(3) Side tunnel or conduit The tunnel or conduit may be provided on one or both sides of the lock chamber. It consists of inlet and outlet valves and number of openings corresponding to the lock chamber. This is provided for filling or emptying the lock chamber during the period of crossing.

(4) Guide walls Sometimes guide walls are provided at the upstream and downstream approach of the lock to protect the structure from the expected damage by the moving vessels. The guide walls may be constructed with concrete or sheet piling or wooden piling.

Operation of Lock Gate

1. When a vessel is moving from downstream to upstream, and it approaches near the downstream lock gate, the gate A is opened and the vessel is allowed to enter the lock chamber. Then the gate A is closed.
2. The inlet valve of the tunnel is opened and the outlet valve is kept closed. The water enters the lock chamber through openings and the water level goes on rising gradually. The vessel also rises up with the rise of water level. When the water level of the chamber reaches the required height, gate B is opened and the vessel is allowed to move. Gate B is then closed and by opening the outlet valve the water in the lock chamber is brought back to its original level.
3. During the movement of the vessel from upstream to downstream, the lock chamber is first filled with water by opening the inlet valve (gates A and B are kept closed). Then the gate B is opened and the vessel is allowed to

enter the lock chamber. The outlet valve is opened and the inlet valve is kept closed. Thus, the water level in the lock chamber goes on reducing gradually and the vessel also sinks downwards. When the water comes to the required level, the gate A is opened and the vessel is allowed to move further towards its journey.

Review Questions

1. Fill up the blanks with appropriate word/words.
   - (i) The communication through the rivers or canals is termed as __________.
   - (ii) In navigable river __________ must be provided with the barrage or weir.
   - (iii) The canalisation method is adopted to improve the __________ navigation.
   - (iv) To maintain the navigability of river __________ should be done ocassionally.
   - (v) At the sharp bend of navigation river a __________ channel should be provided.
2. What are the requirements of navigation waterway?
3. Name the methods of river navigation. Describe each method with a sketch where even necessary.
4. Describe the method of operation of a lock gate showing its component parts by a neat sketch.
5. Write short notes on the following,
   - (a) Snag removal.
   - (b) Lock and dam method.
   - (c) Contraction work.
   - (d) Dredging.

Answers

1. (i) Inland navigation (ii) Lock gate
   (iii) River (iv) Dredging
   (v) Cut-off

16

DAM

16.1 INTRODUCTION

An impervious high barrier which is constructed across a river valley to form a deep storage reservoir is known as dam. It is suitable in hilly region where a deep gorge section is available for the storage reservoir. The dam is meant for serving multipurpose functions such as, (a) Irrigation, (b) Hydroelectric power generation, (c) Flood control, (d) Water supply, (e) Fishery, (f) Recreation.

Weir and Barrage are also impervious barriers across the river. Which are suitable in plain terrain but not in hilly region. The purpose of weir is only to raise the water level to some desired height and the purpose of barrage is to adjust the water level at different levels when required. These two hydraulic structures are suitable for irrigation only.

16.2 SELECTION OF SITE FOR DAM

While selecting the site for a dam, the following points should be considered,

1. Good rocky foundation should be available at the dam site. The nature of the foundation soil should be examined by suitable method of soil exploration.
2. The river valley should be narrow and well defined so that the length of the dam may be short as far as possible.
3. Site should be in deep gorge section of the valley so that large capacity storage can be formed with minimum surface area and minimum length of dam.
4. Valuable property and valuable land should not be submerged due to the construction of dam.
5. The proposed river or its tributaries should not carry large quantity of sediment. If unavoidable, the sources of sediments should be located and necessary measures should be recommended to arrest the sediment.
6. The site should be easily accessible by road or railway for the transport of construction materials, equipments, etc.

7. The construction materials should be available in the vicinity of the dam site.
8. Sufficient space should be available near the site for the construction of labour colony, godowns and staff quarters for the personnel associated with the constructional activities.
9. The basin should be free from cracks, fissures, etc. to avoid percolation loss. It is done by physical verification and other observations. If unavoidable, the area should be located and necessary measures should be recommended to make the area leakproof.
10. From the rainfall records in the catchment area or empirical formulae the maximum discharge of the river should be computed. From the computed value, it should be ascertained whether the required quantity of water shall be available or not.

16.3 INVESTIGATION WORKS FOR DAM SITE

The following investigation works should be done before final selection of dam site and the preparation of the project report,

1. Preliminary Survey The preliminary survey involves the following steps,

(a) Reconnaissance survey The reconnaissance survey should be conducted for the dam site and surrounding area to gather information regarding the natural features of the area, nature of dam site, location of labour colony and staff quarters, stack yard, godowns, etc. The nature of the land and the localities in the basin area should also be recorded. An index map should be prepared.

(b) Topographical survey A topographical map is to be prepared for the proposed project area by traverse surveying. The traverse survey may be conducted by any suitable method depending on the nature of the area.

(c) Contour Survey A contour map should be prepared for the basin area to determine the capacity of the reservoir.

(d) Longitudinal and Cross-sectional levelling Longitudinal levelling and cross-sectional levelling should be done at the dam site at least one km upstream and downstream of the proposed centreline of the dam. This is done to select the most suitable dam site.

2. Geological Survey The geological survey involves the following steps,

(a) Soil survey To know the nature of the foundation at the dam site, soil exploration should be done by suitable method. The sub-soil formation should be throughly studied to determine the type of foundation for the dam.

(b) Study of formation in basin area Soil exploration should be done at different spot in the basin area to ascertain the nature of sub-soil. This is done to calculate the probable percolation loss.

(c) Study of source of sediment The sources of sediments carried by the river or its tributaries should be studied and located. If slips or areas of loose soil with mica particles are found, then the stabilisation of those areas should be done.

3. Hydrological Survey It involves the following steps.

(a) Gauge and discharge site The gauge and discharge stations should be established near the dam site to record the discharge of the river throughout the year.

(b) Silt analysis In rainy season the river carries heavy silt or sediment. The analysis of the silt should be carried out throughout the season for some specific period to determine grade of silt. This is done to ascertain the possible sedimentation in the reservoir and thus suitable methods can be employed to reduce the sedimentation.

4. Communication Survey The route survey for the possible communication of the dam site with the nearest highway or railway station should be done. It involves the preparation of longitudinal section and cross-sections along the proposed alignment. It is done to estimate the cost of construction of this connecting road or railway line. The possible route for telephone communication and electrical connections should also be located.

5. Construction Materials Survey The availability of construction materials like stone, sand, etc. should be located in the topographical map of the concerned district or state. The possible route for carrying these materials should also be located in the map.

6. Compensation Report A detailed report should be prepared for the compensation which is likely to be paid by the government during the implementation of the project. This will include the dam site, area of labour colony and staff quarters, area required for stack yards and godowns, valuable lands and properties that may be submerged by the reservoir, etc.

7. Project Report The project report involves the following steps:

(a) Design and estimate of dam and other allied structures.
(b) Detailed drawings of dam section with foundation and other buildings or structures.
(c) Detailed estimate for the road or railway communication.
(d) Comprehensive report for compensation.
(e) Specification of works and materials of construction.
(f) The project is forwarded to the higher authority with recommendation for approval.

16.4 CLASSIFICATION OF DAM

Dams may be classified on the following basis,

16.5 SOLID GRAVITY DAM

The solid gravity dam may be constructed with rubble masonry or concrete. The rubble masonry is done according to the shape of the dam with rich cement mortar. The upstream and downstream faces are finished with rich cement mortar. Now-a-days, concrete gravity dams are preferred, because they can be easily constructed by laying concrete, layer by layer with construction joints. But good rocky foundation must be available to bear the enormous weight of the dam. The distance between the heel and toe is considered as the base width. It depends, on the height of the dam. Again, the height depends on the nature of foundation. If good rocky foundation is available, the height may be above 200 m. If hard foundation is not available, the height of the dam should be limited to about 20 m. The upstream and downstream base of the dam is made sloping. The horizontal trace (or line) passing through the upstream top edge is known as axis of the dam or the base line. The layout of the dam is done corresponding to this base line. Drainage gallery is provided at the base of the dam. Spill ways are provided at the full reservoir level to allow the surplus water to flow to the downstream. The solid gravity dam resists all the forces acting on it by its self weight (Fig. 16.1).

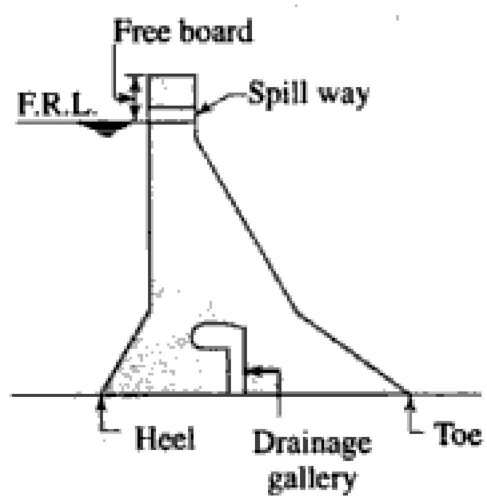


Fig. 16.1 Solid gravity dam

16.6 FORCES ACTING ON GRAVITY DAM

The following forces act on a gravity dam:

1. Weight of the Dam

The weight of the dam is the main stabilising force which counter balances all the external forces acting on the dam. So, the dam should be constructed with heavy materials of high specific gravity. For the construction of dam the specific weight of concrete and stone masonry should not be less than 2400 kg/m^3 and 2300 kg/m^3 respectively. The weight of the dam acts through its centre of gravity. For design purpose, the weight per unit length should be calculated. The centre of gravity of the dam is calculated with respect to the vertical upstream face or with some reference line.

2. Water Pressure

On the upstream face the pressure is exerted by the water stored up to full reservoir level and on the downstream face the pressure is exerted by the tail water. Again, the upstream face of the dam may be completely vertical or partly vertical and partly inclined. But the downstream face is always inclined.

(a) Water pressure on U/S face (when completely vertical)

Total pressure, $P = \frac{W H^2}{2}$, Where H = height of water (F.R.L.)

This pressure acts horizontally at a height $H/3$ from base (Fig. 16.2).

(b) Water pressure on U/S face (when partly vertical and partly inclined) In this case, the water pressure is calculated as horizontal component and vertical component.

Horizontal component, $P_H = \frac{WH^2}{2}$

This pressure acts on the vertical plane ED and at a height $H/3$ from base. The vertical component P_v is the weight of water per unit length contained in the area $E\ F\ G\ D$. This pressure acts through the centre of gravity ($\overline{X}$ from line ED), (Fig. 16.3).

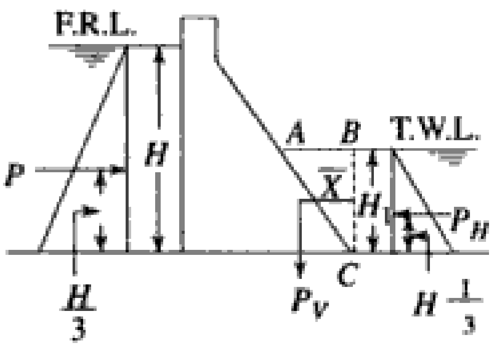


Fig. 16.2 When U/S face vertical

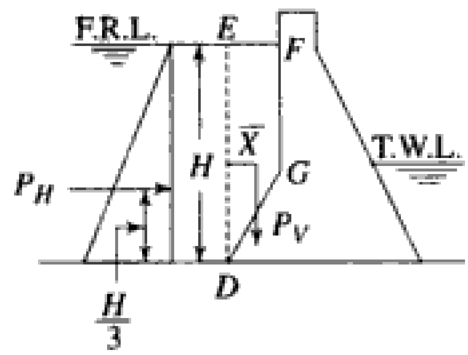


Fig. 16.3 When U/S face inclined

(c) Water pressure on D/S face As the downstream face is inclined, the water pressure is calculated in terms of horizontal component and vertical component.

Horizontal component, $P_H = \frac{WH_1^2}{2}$, This acts horizontally at a height $\frac{H_1}{3}$ from base. The vertical component of pressure P_v is the weight of water per unit length contained in the area $A\ B\ C$, and acts at $\bar{x}$ from BC (Fig. 16.2).

3. Uplift Pressure The stored water on the upstream side of the dam has a tendency to seep through the soil below the foundation. While seeping, the flowing water exerts uplift pressure on the base of the dam which depends on the head of water.

This uplift pressure reduces the self weight of the dam. If the depth of water on the upstream side be H and that on the downstream side be H_1, then the intensity of pressure on U/S base is WH and on the D/S, base is WH_1 (Fig. 16.4).

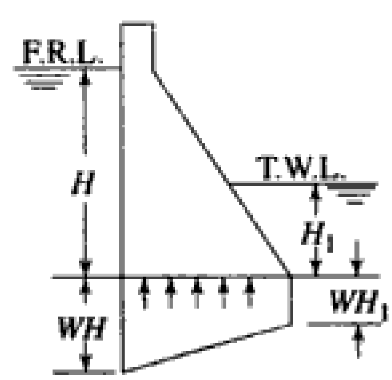


Fig. 16.4 Uplift pressure

(b) Effect of horizontal component The horizontal component imparts following two forces:

(1) Hydrodynamic force Due to horizontal acceleration (f_h), the water pressure is increased momentarily. This extra pressure caused by the earthquake waves is known as hydrodynamic pressure. The expression given by Von-Karman for the hydrodynamic force is as follows (Fig. 16.5):

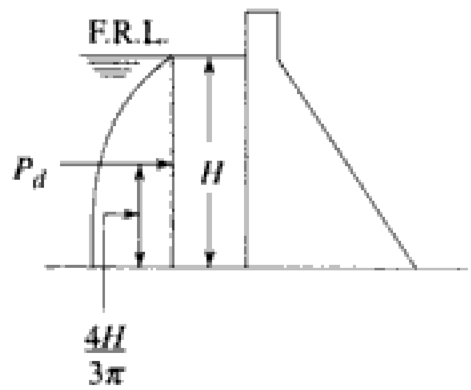


Fig. 16.5 Hydrodynamic force

$$P_d = 0.555 \times f_h \times W \times H^2$$

Where, P_d = hydrodynamic force, f_h = horizontal acceleration, W = sp. wt. of water, H = depth of water in reservoir.

This force acts at a height $\frac{4H}{3\pi}$ from the base of dam.

The moment of this force = $P_d \times \left(\frac{4H}{3\pi}\right) = 0.424 \times P_d \times H$.

(2) Inertia force Due to the horizontal acceleration 'f_h' an inertia force will be developed on the body of the dam. This force is given by, $I = \left(\frac{W}{g}\right) \times f_h$.

but $$f_h = C_h \times g$$

or $$I = \frac{W}{g} \times C_h \times g = W \times C_h$$

Where, I = inertia force, W = total weight of dam, g = acceleration due to gravity, f_h = horizontal acceleration, C_h = a coefficient adopted for horizontal acceleration 0.1 g to 0.3 g.

5. Silt Pressure The silt carried by the river and its tributaries gets deposited against the upstream base of the dam year after year. After considerable deposition of silt, it exerts pressure on the dam. So, provisions should be made to resist this silt pressure. The upstream face of the dam may be completely vertical or partly vertical and partly sloping. So, the pressure of silt will differ accordingly.

(a) When the upstream face is completely vertical, the silt pressure is given by Rankine's formula,

$$P = \frac{W_s\,h^2}{2} \times \frac{(1 - \sin\phi)}{(1 + \sin\phi)}$$

Where, W_s = submerged sp.wt of silt, h = depth of silt deposit, ϕ = angle of internal friction.

This force acts horizontally at a distance 2h/3 below the surface of silt deposit (Fig. 16.6).

(b) When the upstream face is partly vertical and partly sloping, the silt pressure gets resolved in two components—horizontal and vertical. The horizontal component (P_H) is given by,

$$P_H = \frac{W_s\, h^2}{2} \times \frac{(1 - \sin \phi)}{(1 + \sin \phi)}$$

It acts at a distance $\frac{2h}{3}$ from the surface of silt deposit.

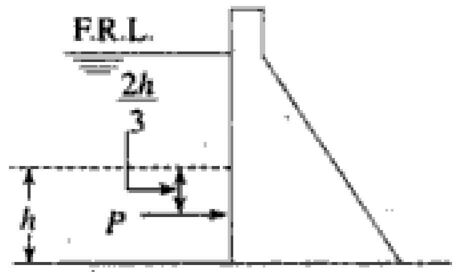


Fig. 16.6 Silt pressure when U/S face vertical

The vertical component P_v is equal to the submerged weight of the silt deposit contained in the area *abcd* for unit length. This force acts at a distance $\bar{x}$ from the line *ab* (Fig. 16.7).

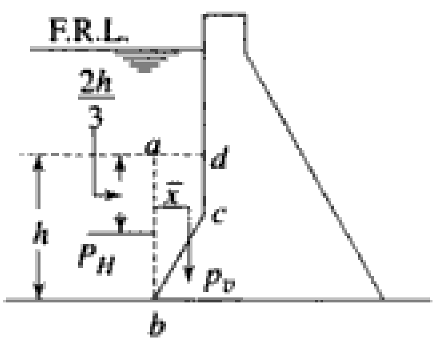


Fig. 16.7 Silt pressure when U/S face inclined

6. Wave Pressure When very high wind or tornado flows over the water surface of the reservoir, waves are formed which exert pressure on the upper part of the dam. The magnitude of the wave depends on the velocity of wind, depth of reservoir and the area of water surface. The wave pressure is calculated by Moliter's formula as follows (Fig. 16.8):

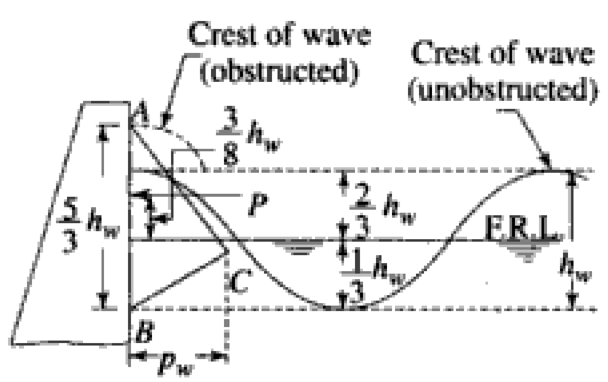


Fig. 16.8 Wave pressure

(i) *Height of wave,*

$$h_w = 0.032\sqrt{VF} + 0.763 - 0.271.\,(F)^{1/4} \text{ (when } F \text{ is less than 32 km)}$$

$$h_w = 0.032\sqrt{VF} \text{ (when } F \text{ is greater than 32 km)}$$

Where, H_w = height of wave, v = velocity of wind km/hr, F = straight length of water surface in km.

(ii) Maximum intensity of pressure, $P_w = 2400 \times h_w$ km/m^2.

(iii) Wave pressure diagram is represented by the area of triangle *ABC*. Total pressure is given by the area of triangle per unit length of dam.

$$P = 1/2 \times p_w \times 5/3\, h_w = 2000\,(h_w)^2 \text{ kg/m}$$

This force acts at a height 3/8 h_w above F.R.L.

7. Ice Pressure This pressure should be counted only in places where the formation of ice is expected on the reservoir surface. When the sheet of ice is formed on the entire water surface of the reservoir, then it exerts pressure on the dam at the point of contact during the process of contraction and expansion with the change of temperature.

In India, ice pressure is generally not required to be considered. However, if required, the intensity of ice pressure may be assumed as 25,000 kg/m^2 which is applicable for the contact area between the ice and the face of dam.

8. Wind Pressure The top exposed portion of the dam is not much and the wind pressure on the surface area of this portion is negligible. But still an allowance should be made for the wind pressure at the rate of about 150 kg/m^2 for the exposed surface area of the upstream and downstream faces.

16.7 CAUSES OF FAILURE OF GRAVITY DAM

The solid gravity dam may fail because of the following reasons,

1. By Over Turning The solid gravity dam may fail by over turning at its toe when the total horizontal forces acting on the dam are greater than the total vertical force (i.e. its, self weight). In such a case, the resultant force passes through a point outside the middle-third of the base of the dam. The overturning may be caused at the downstream edge of any horizontal section.

2. By Sliding The total horizontal forces acting on a dam tend to slide the entire dam at its base or along any horizontal section of the dam. The sliding may take place when the total horizontal forces acting on the dam are greater than the combined resistance offered by shearing resistance of the joint and the static friction.

3. By Over Stressing If the permissible working compressive stress of concrete or masonry exceeds due to some adverse conditions, then the dam may fail by crushing due to overstressing of the concrete or masonry.

4. By Cracking The tensile stresses should not be allowed to develop on the upstream face of the dam. If due to some reasons, the tension is developed in the dam section, crack will form in the body of the dam and ultimately this will cause the failure of the dam.

16.8 PRECAUTIONS AGAINST FAILURE

To avoid failure of the dam, the following precautions should be taken while designing the dam section,

1. To avoid overturning, the resultant of all forces acting on the dam should remain within the middle-third of the base width of the dam. This condition should be achieved in both the cases, when the reservoir is full and also when it is empty.

(b) When Full When the reservoir is full, the resultant R should pass through the point D, the extreme right end of the middle-third. Now, for stability the base width B is to be determined in terms of height 'H'.

Taking moment about D,

$$W \times \frac{B}{3} = P \times H/3$$

or $$\frac{W}{P} = \frac{H}{B} \quad (1)$$

Again, we know, $P = \dfrac{WH^2}{2}$ and $W = \omega \times \rho \times \dfrac{1}{2} \times B \times H$

Where, ω = density of water, ρ = sp. gr. of material of dam.

therefore, $$\frac{W}{P} = \frac{\frac{\omega\rho\, BH}{2}}{\frac{WH^2}{2}}$$

or $$\frac{W}{P} = \frac{\rho B}{H} \quad (2)$$

from (1) and (2)

$$\frac{H}{B} = \frac{\rho B}{H}$$

or $$\rho B^2 = H^2$$

or $$B = \frac{H}{\sqrt{\rho}}$$

So, to keep the resultant force in the middle-third, the base width B should be equal to $\dfrac{H}{\sqrt{\rho}}$.

Thus, the elementary profile of a gravity dam is a right angled triangle with base width equal to $\dfrac{H}{\sqrt{\rho}}$.

16.10 PRACTICAL PROFILE OF A GRAVITY DAM

In elementary profile, we have seen that the maximum pool level is just at the apex of the dam. But, in actual practice the water level may rise above M.P.L. due to various reasons such as heavy wind, waves, peak flood, etc. So, some safe margin should be provided at the top so that the water may not spill over the top of the dam. This margin is known as free board. The amount of free board depends on the peak flood, intensity of wind, height of waves, etc. In normal

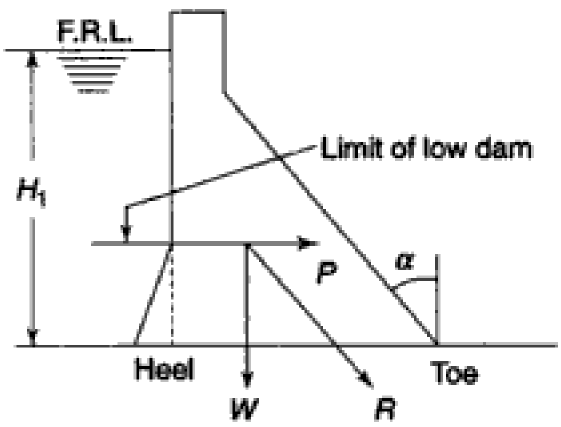


Fig. 16.13 High dam

Example 1 To form a storage reservoir of required capacity a solid gravity dam of height 150 m is to be constructed. Comment whether the said dam will be designed as low dam or high dam. Taking permissible working stress as 40 kg/cm^2 sp. gr. of the materials of dam as 2.5.

Solution We know, the limiting height of low dam is,

$$H = \frac{f}{\omega(\rho + 1)} = \frac{40 \times 10^4}{1000(2.5 + 1)} = 114.29 \text{ m}$$

Here,

$$f = 40 \text{ kg/cm}^2 = 40 \times 10^4 \text{ kg/m}^2,\ W = 1000 \text{ kg/m}^3\ \rho = 2.5.$$

So, the height of the proposed dam is greater than the limiting height of the low dam.

Hence, the dam should be designed considering it as a high dam.

Example 2 Find the maximum height of the low dam, having the following data.

Cement concrete = 1 : 2 : 4, factor of safety = 4, sp. gr. of materials = 2. 4. Draw the section of the dam.

Solution The ultimate compressive strength of cement concrete (1 : 2 : 4) may be assumed as 150 kg/m^2.

$$\text{Allowable compressive stress } (f) = \frac{150}{4} = 37.5 \text{ kg/cm}^2$$

$$= 37.5 \times 10^4 \text{ kg/m}^2$$

Here, $\rho = 2.4,\ W = 1000 \text{ kg/m}^3$

$$H = \frac{f}{\omega(\rho + 1)} = \frac{37.5 \times 10^4}{1000(2.4 + 1)} = 110.30 \text{ m}$$

So, the height of low dam should be 110.3 m (Fig. S-1). Assuming free board = 2.3 m.

Depth of water = 110.3 – 2.3 = 108 m.

Top width of dam = 0.552 $\sqrt{H}$ = 0.552$\sqrt{108}$ = 10.39 m.

Single Block System In this system the Zones I, II, and III are provided by graphical system as usual. After this zone, the upstream face is made sloping and the same slope is maintained in the downstream face. Thus the resultant is made to pass through the middle-third for both conditions-when the reservoir is empty and when it is full. This condition is achieved by trial-and-error method. The stability is then checked by computation method.

This system of fixing the dam profile is uneconomical for very high dams, because it give enormous base width. So, it is economical for dams of lesser heights (Fig. 16.15).

2. Mathematical Method or Analytical Method The procedure of this method is as follows (Fig. 16.16).

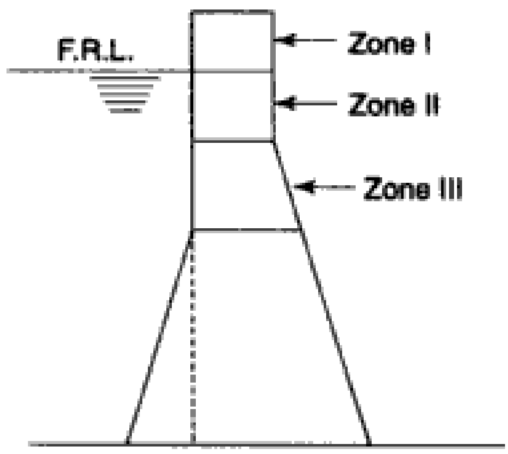


Fig. 16.15 Single block system

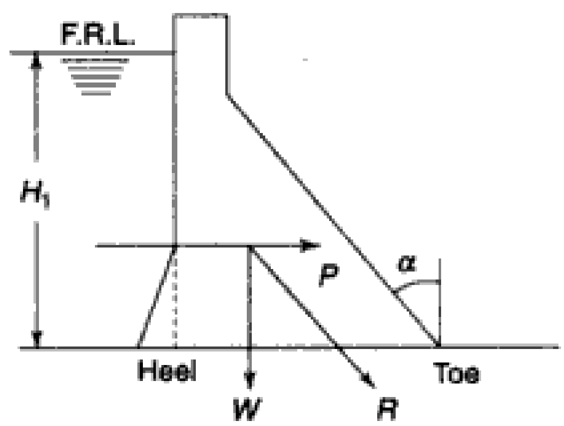


Fig. 16.16 Mathematical method

1. At every critical horizontal joint, the following factors are calculated for both the conditions—reservoir empty and reservoir full.
2. All the vertical forces acting on the dam are calculated which include self weight of dam, weight of water on sloping face, uplift pressure, etc. The algebraic sum of these forces is calculated, say, ΣW.
3. All horizontal forces are calculated which include water pressure, hydrodynamic force, inertia force, etc. The algebraic sum of these forces also is determined, say, P.
4. The overturning moment (M_0) and moment of resistance M_R are calculated at the toe and the heel. Let the algebric sum is

   $$\Sigma M = \Sigma M_R - \Sigma M_0$$

5. The lever arm $\bar{x}$ of the resultant force is calculated by $\bar{x} = \dfrac{\Sigma M}{\Sigma W}$.
6. The eccentricity of the resultant is found out by,

   $$e = \frac{B}{2} - \bar{x}$$

   where, B = base width of dam.
7. The normal stresses at the toe and heel are found out by,
   (a) Normal stress at toe,

$$N_s = \frac{\Sigma W}{B}\left(1 + \frac{6 \cdot e}{B}\right)$$

(b) Normal stress at heel,

$$N_s = \frac{\Sigma W}{B}\left(1 - \frac{6 \cdot e}{B}\right)$$

8. Principal stress at the toe and heel is found out by,

$$\sigma = N_s \cdot \sec^2 \alpha - P \tan^2 \alpha$$

where, σ = principal stress, N_s = normal stress, P = total horizontal force α = angle made by the toe with normal.

9. Shear stress at the toe and heel is found out by,

$$\pi = (N_s - P) \tan^2 \alpha$$

10. Factor of safety for different conditions,

(a) Factor of safety for overturning = $\dfrac{\Sigma M_R}{\Sigma M_0}$

(b) Factor of safety for sliding = $\dfrac{\mu \Sigma M}{P}$

(c) Factor safety for shear = $\dfrac{\mu \Sigma W + A \cdot q}{P}$

Where, A = Area of section under consideration, q = average shear strength of the materials, μ = coefficient of friction.

16.15 ARCH DAM

Definitions of some terms related to arch dam are:

(1) Arch Dam A dam which is constructed in the form of an arch supported an abutments is called the arch dam. It transfers the major water pressure to the abutments by the arch action. A part of the water pressure is transferred to the foundation by cantilever action.

The arch dam may be constructed in masonry or concrete. The abutments of the arch should be very strong because the major thrust developed by the water pressure is carried by it. The arch dam is suitable for '*V*-shaped' valley.

(2) Extrados The outside curved surface (i.e. upstream surface) of the arch dam is known as extrados.

(3) Intrados The inside curved surface (i.e. downstream surface) of the arch dam is known as intrados.

(4) Slenderness Ratio The ratio of the curved length to the thickness of the curved ring is known as slenderness ratio. At the top, this ratio should not exceed 75 and the ratio for the middle and base should be about 25.

$$\text{S.R.} = \frac{\text{Length of arch}}{\text{Thickness of each}} = \frac{r, \theta}{t}$$

Where, r = radius of the arch, θ = angle subtended by the arch at the centre, t = thickness of the arch (distance between extrados and intrados).

(5) Length of Arch Dam It is the distance measured along the axis of the dam (i.e. centre line of the arch ring) at the top level between the abutments.

(6) Thickness of Arch Dam It is the distance between the extrados and intrados at any point. The thickness varies from the bottom (maximum) to the top (minimum) according to the type of the arch.

Again, according to the ratio of B (base width of dam) and H (height of dam), the dam is designated as thin, medium and thick.

Thin arch dam $\frac{B}{H} \leq 0.2$,

Medium arch dam $0.2 > \frac{B}{H} < 0.3$,

Thick arch dam $\frac{B}{H} > 0.3$.

(7) Length-height Ratio The economic limit of the arch dam is defined by the ratio of length and height. When this ratio varies from 4 to 6 the arch dam is considered economical.

16.16 FORCES ON ARCH DAM

The forces acting on the arch dam are similar to the forces acting on the solid gravity dam. But the following differences should be noted,

1. The uplift pressure is less important than solid gravity, because the base width of arch dam is narrow.
2. The ice pressure should be considered with much importance because it produces continuous concentrated load along the arch.
3. The change of temperature causes high internal stress, so, this factor should also be given importance.

16.17 TYPES OF ARCH DAMS

The arch dam is classified according to the method of construction. The following are three main types of arch dam,

(1) Constant Radius Arch Dam In constant radius arch dam, the radius of the extrados is kept constant at all elevations from top to bottom. But the radius of intrados goes on decreasing gradually, because the thickness of the arch goes on increasing towards the base. The upstream face is vertical and the downstream face is sloping. Again, in this type of arch dam the centres of extrados, and intrados and the centreline of the arch lie on the same vertical line. Sometimes, this arch is known as constant centre arch. Here the central angles of the intrados at different elevations go on decreasing. This type of arch is uneconomical in case of V-shaped valley, but, economical for 'U-shaped' valley (Fig. 16.17).

(2) Constant Angle Arch Dam In constant angle arch dam, the central angles of the arch rings at all elevations are kept constant. Theoretically, it was found that if the central angle is kept constant at 133°–134°, then the arch dam will be economical. But, in actual practice the central angle is kept between 100° to 150° according to the shape of the valley. The constant angle arch dam is suitable for both the 'V-shaped' and 'U-shaped' valleys (Fig. 16.18).

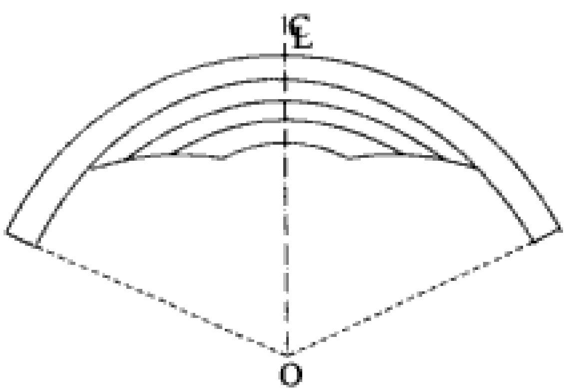


Fig. 16.17 Constant radius arch dam

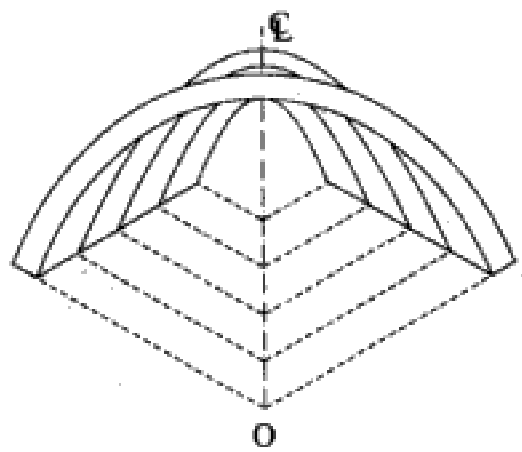


Fig. 16.18 Constant angle arch dam

(3) Variable Radius and Variable Angle Arch Dam In this type of arch dam, the radius of extrados and intrados of the arch vary at different elevations from top to bottom. The radius is maximum at the top and minimum at the bottom of the dam. The central angles of the arch rings also vary from top to bottom of the dam, the angle is maximum at the top and minimum at the base of the dam. Generally, the central angles vary from 80° to 150°.

This type of arch dam is suitable for both 'U-shaped' and 'V-shaped' valleys (Fig. 16.19).

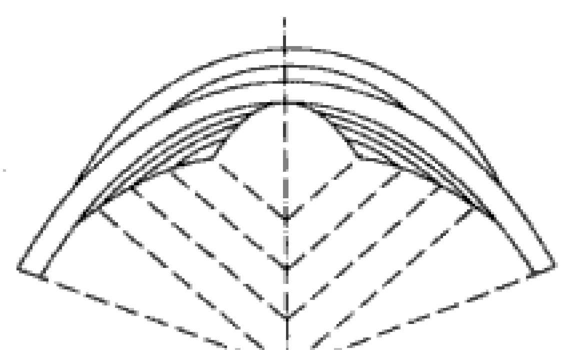


Fig. 16.19 Variable radius and variable arch dam

16.18 BUTTRESS DAM

In buttress dam an inclined deck or slab is supported by a series of triangular walls which are known as buttresses. The buttresses are constructed at right angles to the axis of the dam and they are equally spaced throughout the length of the dam. The upstream end of the buttress consists of haunch or corbel which

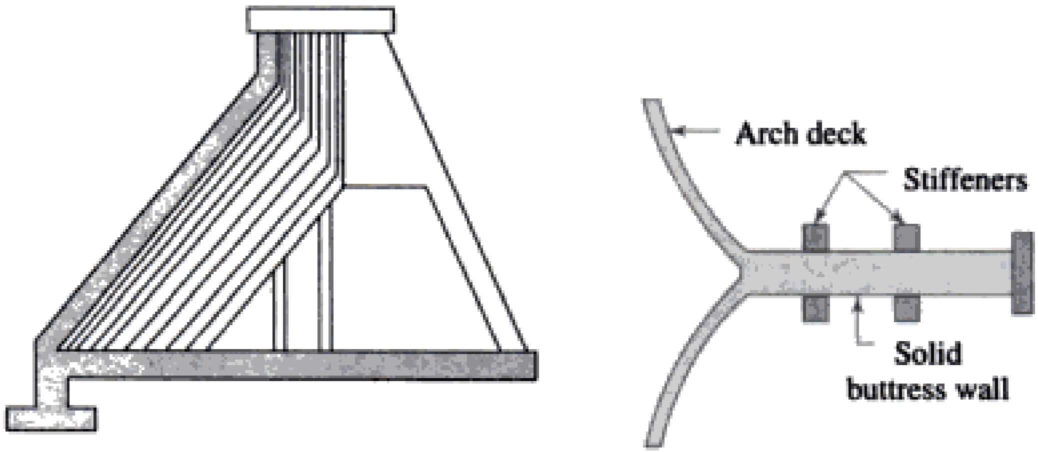


Fig. 16.21 Multiple arch type buttress dam

(4) Massive Head or Bulk Head Type Buttress Dam In this type of dam no separate deck is provided to retain water. The water retaining member is made with buttresses by enlarging the upstream ends like bulbs known as massive heads or bulk heads. The bulk heads are joined side by side to form a water tight continuous body. The joints of the bulk heads are provided with copper strip water stops and the entire joint is filled up with bituminous filling materials. So, each buttress behaves like an independent member (Fig. 16.23).

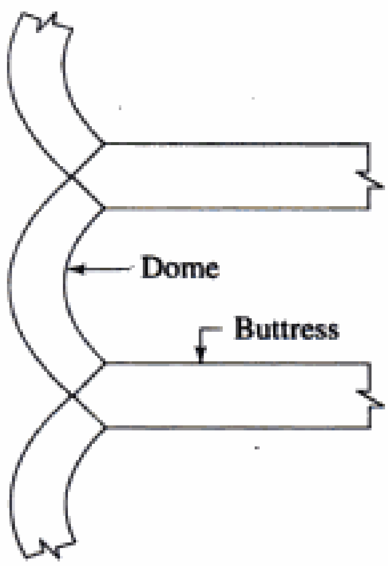


Fig. 16.22 Multiple dome type buttress dam

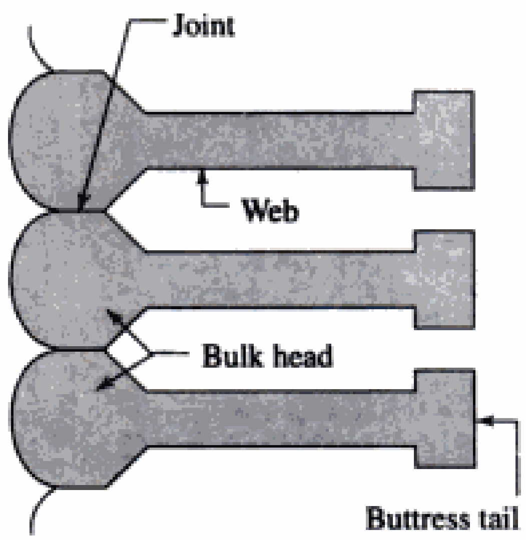


Fig. 16.23 Bulk head buttress dam

16.20 MERITS AND DEMERITS OF BUTTRESS DAM

Merits

1. With comparison to solid gravities dam, this dam requires less construction material.
2. As the water resisting deck is inclined, the vertical component of the water pressure helps in stabilising the dam.
3. The ice pressure has no significance in this dam, because it tends to slide over the upstream face.

4. This dam can be constructed on a weak foundation.
5. It is suitable for wide valley.
6. The hollow space between the buttresses can be utilised for storage.
7. The height of the buttress dam can be raised at any time.

Demerits

1. This type of dam can be easily damaged by sabotage.
2. The life of the dam gets reduced if the water retaining members get damaged due to rusting of reinforcements.
3. It requires extensive form work and a lot of skilled labour.
4. Constant maintenance and supervision is required.

16.21 EARTHEN DAM

Earthen dams are constructed purely by earth work in trapezoidal section. These are most economical and suitable for weak foundation. Earthen dams are classified as follows:

Based on Method of Construction

Rolled Fill Dam In this method, the dam is constructed in successive layers of earth by mechanical compaction. The selected soil is transported from borrowpits and laid on the dam section, to layers of about 45 cm. The layers are thoroughly compacted by rollers of recommended weight and type. When the compaction of one layer is fully achieved, the next layer is laid and compacted in the usual way. The designed dam saction hence is completed layer by layer.

Hydraulic Fill Dam In this method, the dam section is constructed with the help of water. Sufficient water is poured in the borrowpit and by pugging thoroughly, slurry is formed. This slurry is transported to the dam site by pipe line and discharged near the upstream and downstream faces of the dam. The coarser material gets deposited near the face and the finer material move towards the centre and get deposited there. Thus the dam section is formed with faces of coarse material and central core is of impervious materials like clay and silt. In this case, compaction is not necessary.

Semi-Hydraulic Fill Dam In this method the selected earth is transported from the borrowpit and dumped within the section of the dam, as done in the case of rolled fill dam. While dumping no water is used. But, after dumping the water jet is forced on the dumped earth. Due to the action of water the finer materials move towards the centre of the dam and an impervious core is formed with fine materials like clay. The outside body is formed by coarse material, In this case also compaction is not necessary.

Homogeneous Type Dam This type of dam is constructed purely with earth in trapezoidal section having the side slopes according to the angle of repose of the soil. The top width and height depends on the depth of water to be retained and the gradient of the seepage line. The phreatic line (top level of seepage line) should pass well within the body of the dam. This type of dam is completely pervious. The upstream face of the dam is protected by stone pitching. Now-a-days, the earthen dam is modified by providing horizontal drainage blanket or rock toe (Fig. 16.24).

Zoned Type Dam This type of dam consists of several materials. the impervious core is made of puddle clay and the outer previous shell is constructed with the mixture of earth, sand, gravel, etc. The core is trapezoidal in section and its width depends on the seepage characteristics of the soil mixture on the upstream side. The core is extended below the base of the dam to control the sub-soil seepage. Transition filters are provided on both sides of the impervious core to control the seepage. The transition filter is made of gravel and coarse sand. The upstream face of the dam is protected by stone pitching (Fig. 16.25).

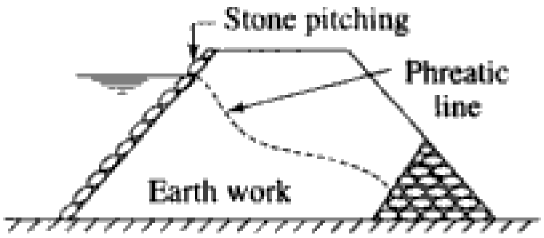


Fig. 16.24 Homogeneous type dam

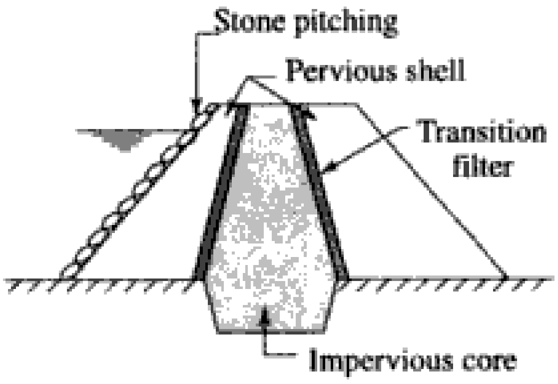


Fig. 16.25 Zoned type dam

Diaphragm Type Dam In this type of dam, a thin impervious core or diaphragm is provided which may consist of puddle clay or cement concrete or bituminous concrete. The upstream and downstream body of the dam is constructed with pervious shell which consists of the mixture of soil, sand, gravel, etc. The thickness of the core is generally less than 3m. A blanket of stones is provided on the toe of the dam for the drainage of the seepage water without damaging the base of the dam. the upstream face is protected by stone pitching. The side slope of the dam should be decided according to the angle of repose of the soil mixture (Fig. 16.26).

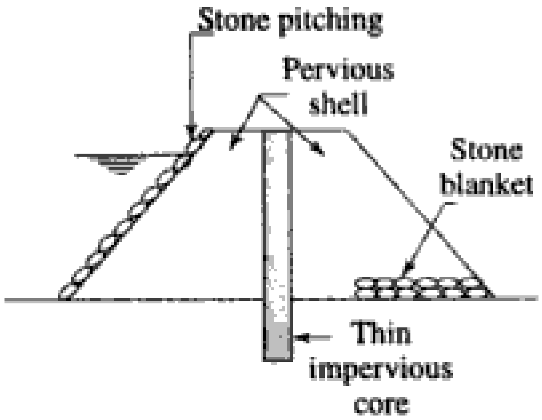


Fig. 16.26 Diaphragm type dam

16.22 CAUSES OF FAILURE OF EARTHEN DAM

The failure of the earthen dam may be caused due to the reasons,

(1) Hydraulic Failure This type of failure may be caused by:

(a) Overtopping If the actual flood discharge is much more than the estimated flood discharge or the free board is kept insufficient or there is settlement of the dam or the capacity of spill way is insufficient, then it results in the overtopping of the dam. During the overtopping the crest of the dam may be washed out and the dam may collapse.

(b) Erosion If the stone protection of the upstream side is insufficient, then the upstream face may be damaged by erosion due to wave action. The downstream side also may be damaged by tail water, rainwater, etc. The toe of the dam may also get damaged by the water flowing through the spill ways.

(2) Seepage Failure This type of failure may be caused by:

(a) Piping or undermining Due to the continuous seepage flow through the body of the dam and through the sub-soil below the dam, the downstream side gets eroded or washed out and a hollow pipe like groove is formed which extends gradually towards the upstream through the base of the dam. This phenomenon is known as piping or undermining. This effect weakens the dam and ultimately causes the failure of the dam.

(b) Sloughing The crumbling of the toe of the dam is known as sloughing. When the reservoir runs full, for a longer time, the downstream base of the dam remains saturated. Due to the force of the seepage water the toe of the dam goes on crumbling gradually. Ultimately the base of the dam collapses.

(3) Structural Failure This type of failure may be caused by

(a) Sliding of the side slopes Sometimes, it is found that the side slope of the dam slides down to form some steeper slope. The dam goes on depressing gradually and then overtopping occurs which leads to the failure of the dam.

(b) Damage by burrowing animals Some burrowing animals like craw-fish, snakes, squirrel, rats, etc cause damage to the dam by digging holes through the foundation and body of the dam.

(c) Damage by earthquake Due to earthquake cracks may develop on the body of the dam and the dam may eventually collapse.

16.23 ROCK-FILL DAM

This type of dam is constructed by dumping stones (i.e. boulders) in a trapezoidal section. No mortar is used while dumping the stones. The stones are dropped from some height so that the edges of the stones are broken and they are well set with each other. Rubble measonry is done on the upstream side which is grouted by cement. Finally an impervious membrane is provided over the rubble masonry by concrete or asphalt to make the surface water tight. This type of dam is suitable when plenty of stones are available from a nearby quarry (Fig. 16.27).

16.24 COMPOSITE DAM

This type of dam consists of rock-fill on the downstream side and earth-fill on the upstream side. The rock-fill section is constructed in the usual way. The earth-fill section is constructed with selected earth which is compacted properly. The upstream side is protected by stone pitching which is grouted with cement mortar. An impervious membrane of cement concrete or asphalt is provided over the stone pitching to make the surface water tight. The height of the dam depends on the depth of water to be retained and the side slope is fixed according to the site condition (Fig. 16.28).

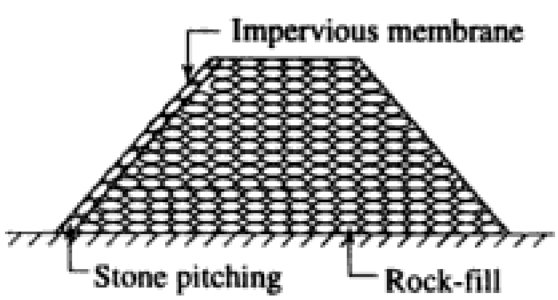


Fig. 16.27 Rock-fill dam

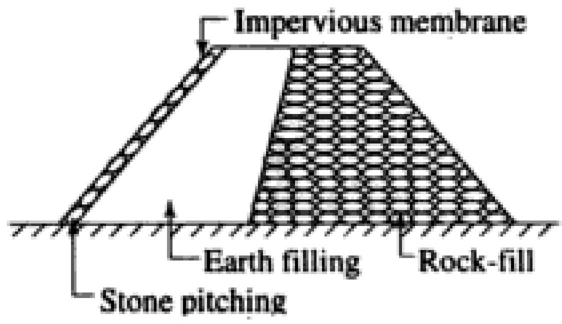


Fig. 16.28 Composite dam

16.25 STEEL DAM

Steel dam is constructed with R.S. joists, iron angles and steel plates. R.S. joists are driven into the ground with which struts (iron angles) are fixed. Rafters (iron angle) are fixed with the struts to form the inclined dam profile. Purlines are fixed on rafters at suitable interval. Then steel plates are fixed with the purline to form the water retaining member. This dam is mainly used for coffer dam (Fig. 16.29).

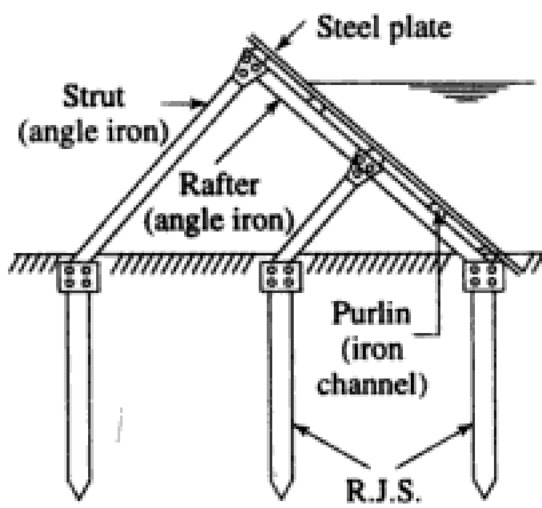


Fig. 16.29 Steel dam

16.26 TIMBER DAM

Timber dam is a temporary structure constructed to retain water at a depth of about 10 m. When timber is available in large quantities, this type of dam is constructed at a moderate cost. Anchor posts are driven into the ground and a tie beam is fixed over the posts. The tie beam consists of inclined posts and rafter. The posts are stiffened with struts. The rafter consists of purlins. Then planks are fixed on the purlins which form the membrane for retaining water. This type of dam is mainly used for coffer dam (Fig. 16.30).

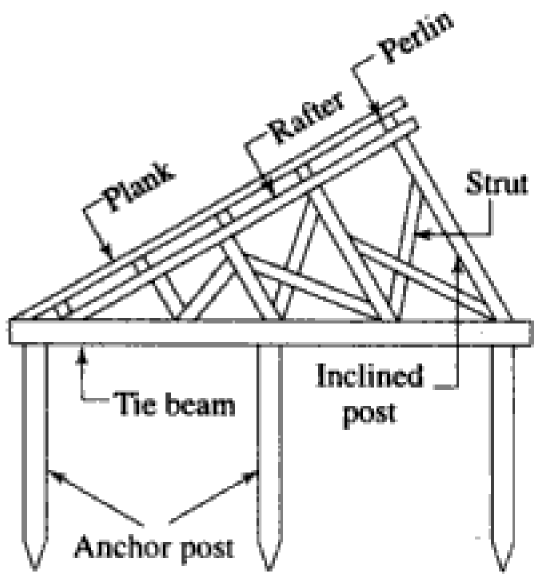


Fig. 16.30 Timber dam

REVIEW QUESTIONS

1. Fill up the blanks with appropriate word/words
   (i) The river valley should be _________ and well defined at the dam site.
   (ii) The nature of the foundation soil should be examined by soil _________ method.
   (iii) When the dam is constructed with masonry or concrete, then it is known as _________ dam.
   (iv) When the dam is constructed with earth or rock-fill, then it is known as _________ dam.
   (v) When the total horizontal force acting on the dam is greater than the total vertical force, then the dam may fail by _________
   (vi) The dam may fail by sliding along its _________ or any _________ section.
   (vii) There will be no tension in the dam section if the resultant passes through the _________
   (viii) The elementary profile of a gravity dam is right angled triangle with base width _________
   (ix) A low dam is designed on the basis of _________
   (x) A dam is considered high when its height is _________ than the expression $\frac{f}{\omega(\rho+1)}$.
2. Name the forces acting on a gravity dam. Enumerate any four with sketches wherever necessary.
3. (a) How can a gravity dam fail?
   (b) What precautions should be taken against the factors leading to the failure?

17

RESERVOIR PLANNING, WATER MANAGEMENT AND WATER LAWS

17.1 PURPOSE OF RESERVOIR

The storage reservoir is formed for the following purposes:

1. Flood control
2. Irrigation
3. Water supply.
4. Hydroelectric power generation
5. Development of fishery
6. Navigation
7. Soil conservation.

Again the reservoir may be designated by,

(a) Single Purpose Reservoir This type of reservoir is formed mainly to serve a single purpose, such as irrigation, flood control, water supply, etc.

(b) Multipurpose Reservoir This type of reservoir is formed to serve many purposes such as,

(i) Irrigation and water supply.
(ii) Irrigation, water supply and flood control.
(iii) Irrigation, water supply, flood control, hydroelectric power generation, fishery, etc.

17.2 CLASSIFICATION OF RESERVOIR

(1) Storage Reservoir The storage reservoir is formed by constructing a dam across a river valley. The idea of constructing such a reservoir is to store the excess water which flows through the river during the high floods or rainy season. This stored water is then utilised for various purposes, such as irrigation, water supply, fishery, hydroelectric power generation, etc. Again, the storage reservoir may be named as single purpose reservoir or multipurpose reservoir according to its utility.

(2) Flood Control Reservoir Flood control reservoirs are formed by constructing dams at suitable places in the catchment area or river valley to arrest the flood water temporarily so that the downstream area may not get damaged by sudden high flood discharge. The arrested water is then allowed to flow or released gradually without causing any harm to the low lying areas on the downstream side. This type of reservoir is designated as single purpose reservoir. Again the flood control reservoir may be of two types,

(a) Retarding Reservoir In this type of reservoir spill ways (without adjustable gates) are provided with the dam at such level and capacity so that the flood discharge through them is safe for the downstream areas. That means, the high flood discharge is retarded and it takes long time for the flood water to flow completely towards the downstream area. The discharge stops when the water level falls below the crest of the spill ways.

(b) Detention Reservoir In this type of reservoir, the spill ways with adjustable gates are provided with the dam so that the flood water may be detained for sometime and then released according to the situation of the downstream area by operating the gates of the spill ways.

(3) Distribution Reservoir The distribution reservoir is not formed by constructing a dam across a river valley or river. It is constructed by masonry work or concrete work in the form of a rectangular or circular tank at suitable places near the town or city. The water from the river or lake is pumped into this reservoir and stored there for supplying to the consumers of the town or city. The water may be supplied to consumers by pumping system or gravity system.

17.3 SELECTION OF SITE FOR RESERVOIR

The following points should be remembered while selecting a site for a reservoir.

1. Stable foundation for the dam should be available where the reservoir basin is proposed to be formed.
2. At the selected site the river valley should be narrow and well defined so that the length of the dam may be short.
3. The proposed reservoir basin should be watertight and free from cracks, fissures, etc. so that there is no loss of water due to percolation.
4. The reservoir basin should not submerge valuable land and properties.
5. The tributaries which carry high content of sediment should be avoided.
6. The reservoir site should be such that the optimum storage capacity is available for the desired purpose.
7. The site should be such that a deep reservoir may be formed with minimum surface area to control the loss of water due to evaporation.
8. The site should be easily accessible by road or railway.

9. The construction materials for the dam and other allied works should be available in the vicinity of the site.
10. Suitable area should be available for labour colonies, staff quarters, godowns, stack yards, etc.

17.4 INVESTIGATION WORKS FOR RESERVOIR

The following investigations should be carried out for planning the storage reservoir:

(a) Engineering Survey The area of the dam site and reservoir basin should be surveyed throughly to prepare topographical map and contour map. From the contour map the storage capacity of the reservoir should be fixed by ascertaining the height of the dam. Thus, the valuable land and properties which may get submerged in the reservoir can be found out. So, the amount of compensation to be paid to the owners can be estimated. A detailed statement may be prepared for the land acquisition. Road and railway communications should be located in the topographical map.

(b) Geological Survey It should be carried out to determine the following informations:

(1) Dam foundation The sub-surface exploration at the dam site should be carried out to determine the properties of foundation soil at various depths. This soil exploration helps to fix the depth and type of foundation for the dam.

(2) Characteristic of reservoir basin The geological survey of the basin should be carried out to locate the cracks, fissures, etc which are responsible for the percolation loss. Necessary measures can then be recommended for percolating zones to control the losses.

(3) Availability of construction materials Huge quantity of materials like fine aggregate (i.e. sand) and coarse aggregate (i.e. stone chips) are required for the construction of dam and allied structures. The places where these materials are available should be located and they should preferably be near the site. If the quarries and sand pits are situated in the vicinity of the project, it reduces the cost of the project.

(c) Hydrological Survey This survey includes the collection of following data:

(1) River gauging River gauging (i.e. discharge observation) should be carried out for the main river and its tributaries in the catchment area and on the upstream side of the site. This is to estimate the probable discharge of the river throughout the year.

(2) Rain fall records The rainfall records from the raingauge stations in the catchment area for the past few decades are collected and the run-off is estimated.

represents the total quantity of flow during that period. Thus the ordinates of the mass curve corresponding to different time period are found out from the hydrograph. Then these ordinates are plotted against time to obtain the shape of mass curve (Fig. 17.2).

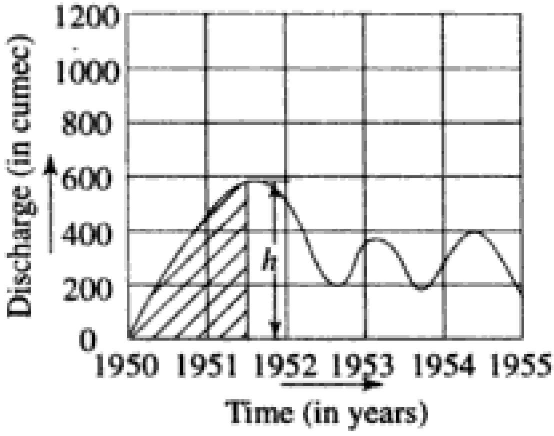


Fig. 17.1 Hydrograph

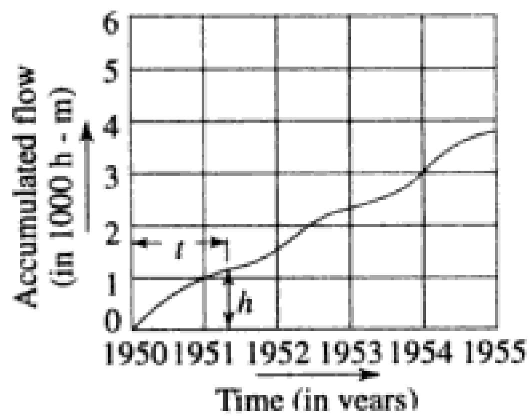


Fig. 17.2 Mass curve

Demand Curve A demand curve is a graph between the accumulated demand (as ordinate) and time (as abscissa). The demand curve shows the yearly consumption of water. Again, the demand may be constant or variable. When the demand is constant, the curve is a straight line (Fig. 17.3). When the demand is variable, the curve will be irregular (Fig. 17.4).

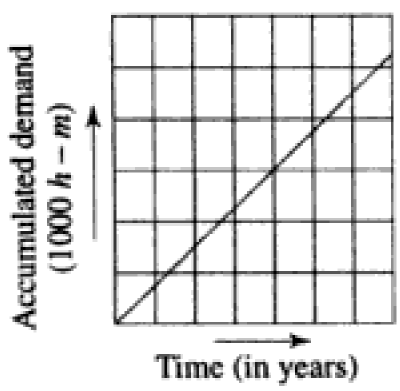


Fig. 17.3 Constant demand

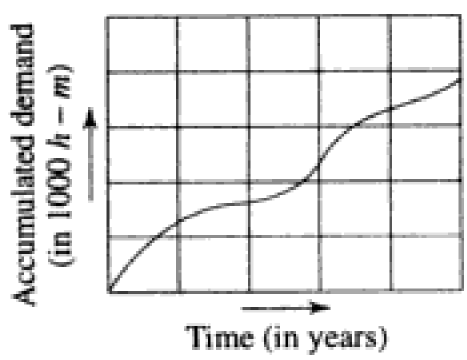


Fig. 17.4 Variable demand

17.7 ZONES OF STORAGE RESERVOIR

The storage capacity of a reservoir is designated by several zones which are demarkated by certain pool levels as stated below (Fig. 17.5).

Normal Pool Level (N.P.L.) It is the maximum elevation of the water surface which is to be stored in the reservoir during the normal working period. The water above this level will flow out over the spill way crest. This water level is also known as full reservoir level (F.R.L.).

Maximum Pool Level (M.P.L.) It is the maximum elevation to which the water surface is allowed to rise in the reservoir during the period of flood. This level can be controlled by providing spill way gate.

(b) The weeds from the surface of the reservoir should be removed.

3. Measure to Reduce Percolation Loss

(a) Geological investigations should be carried out to locate the zones of pervious formations, cracks and fissures in the bed and periphery of the reservoir basin.

(b) Suitable treatments should be adopted to stop the leakage of water through these zones.

(c) Soil stabilisation methods should be adopted if the basin is composed of permeable bed soil.

17.11 CAUSES OF SEDIMENTATION IN RESERVOIR

In the catchment area the soil may get eroded and rocks may get disintegrated due to various reasons. The disintegrated rocks and loose soil form the sediment which is carried by the river and gets deposited on the reservoir bed near the base of the dam. The sediment mainly consists of sand and silt. The process of deposition of sand and silt in the reservoir is designated as reservoir sedimentation.

The following are the causes of sedimentation:

(1) Characteristics of Soil in Catchment Area If the catchment area is composed of loose soil, then it may get easily eroded and get carried away by the river. On the other hand, if the soil of the catchment area is hard and rocky, the river would not be able to the carry sediment.

(2) Topography of the Catchment Area If the catchment area consists of steep slope, then it will develop high velocity of flow which will cause more erosion of the surface soil thereby making the river carry a lot of sediment.

(3) Intensity of Rainfall in Catchment Area If the intensity of rainfall in catchment area is high, then it will increase the rate of run-off and the river will carry more sediment.

(4) Cultivation in Catchment Area The intensive cultivation in the catchment area will make the soil loose and the rain water will carry a lot of sediment to the river.

(5) Vegetal Apron in Catchment Area If the catchment area is covered with vegetal apron like grass, plants, forest area, etc. then the erosion of the soil will be controlled and the river will not carry sediment. If there is no such vegetal apron or cover, the soil may be easily eroded and the river will carry much sediment.

17.12 EFFECT OF SEDIMENTATION

When the sediment laden water of the river approaches the zone of reservoir, the velocity of flow reduces gradually and thus the heavier particles are settled down at the head of the reservoir, i.e. starting zone of reservoir. This zone is termed as delta. Most of the sediments get deposited at this zone.

(5) Control of Deforestation The cutting of trees, i.e. deforestation in the catchment should be restricted by the government so that the vegetal cover is not destroyed. The deforestation may cause soil erosion and this may impart sediment load to the river water.

(6) Control of Cultivation The intensive cultivation in the catchment makes the soil loose which may then get easily carried by the rainwater to the river. So, the cultivation should be done in a planned manner and necessary measures should be taken to avoid the loose soil getting carried away by the rain water.

(7) Control of Grazing The grazing of cattle in the catchment should be restricted and they should not be allowed in the area where the soil can get easily eroded by their feet.

(8) Construction of Check Dams If the tributaries of a river are found to carry heavy sediment load, then check dams of low height are contructed across the tributaries at different stages to arrest the sediments just in the catchment area. Here, the check dams serve the purpose of detention reservoirs where the heavy sediments get settled down.

(9) Construction of Contour Bunds In hilly area contour bunds are constructed on the slope of the catchment at different elevations so that the heavy sediments are arrested at the base of the bunds and comparatively less turbid water passes over the bund (Fig. 17.7).

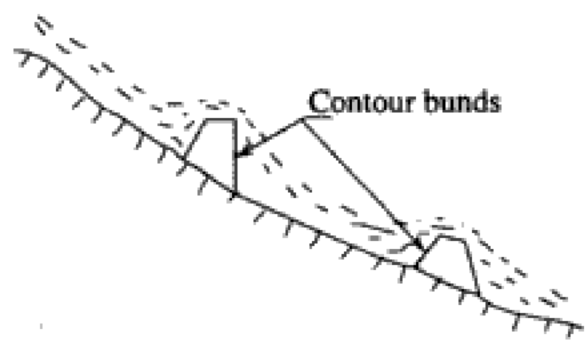


Fig. 17.7 Contour bunds

(10) Stabilisation of Slips (i.e. land slides) Most of the tributaries of a river obtain the sediments from the land slides which are caused due to the heavy rainfall in the hilly area. It is generally composed of loose soil or permeable rock formation. These slips should be stabilised by suitable methods so that the debris may not be carried by the flowing water.

17.14 FLOOD ROUTING

The flood routing is a process of determining the reservoir storage volume, out flow rates and the rise of water level in the reservoir corresponding to any inflow during any peak flood discharge. This is done to forecast the probable volume of storage and maximum water level in the reservoir and also the rate of outflow towards the downstream. Flood routing is a technique used for the flood control.

In flood routing, the following relation is studied thoroughly,

$$I = 0 + \Delta_v$$

where, I = average inflow during a specified period, 0 = average outflow during the same period, Δ_v = change in volume of storage during the same period.

When the water level rises in the reservoir, it indicates the increase in volume of storage and the change in storage (Δ_v) is positive. When the water level falls in the reservoir, it indicates the decrease in storage and the change in storage (Δ_v) is negative.

The flood routing can be done by graphical method. In this method, the inflow hydrograph (Fig. 17.8), storage curve (Fig. 17.9) and the outflow curve (Fig. 17.10) are prepared. By studying these graphs, the necessary controlling system should be adopted for the outflow so that the downstream areas may not be affected by the flood water.

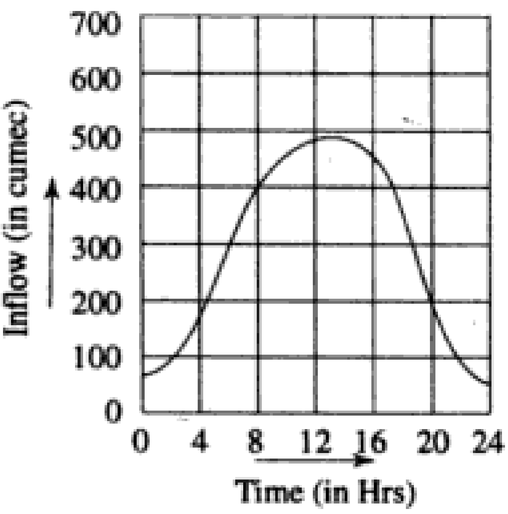


Fig. 17.8 Inflow hydrograph

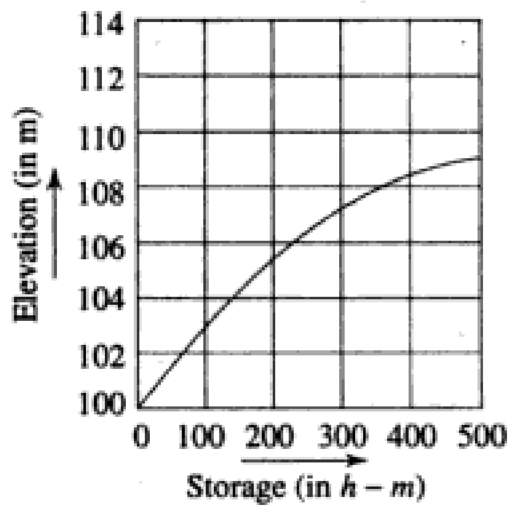


Fig. 17.9 Storage curve

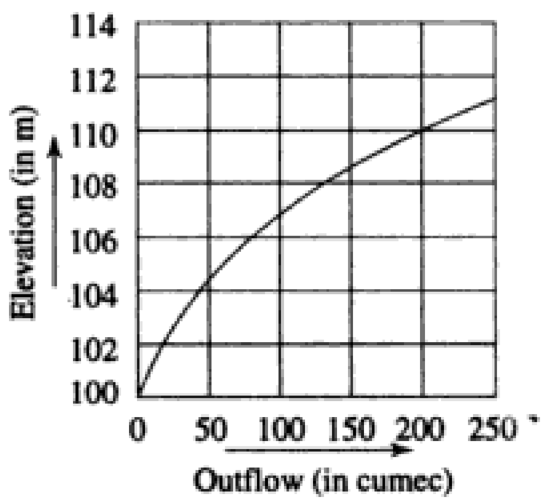


Fig. 17.10 Outflow curve

17.15 WATER MANAGEMENT

In rainy season, when plenty of water is available in the river or the reservoir, the restrictions for using water do not arise. In dry season, when the capacity of the reservoir is limited, the water should be used very carefully and economically to avoid unnecessary waste. Thus, the maximum benefit may be obtained by utilising minimum volume of water. So, the water management is the science for the proper utilisation of water by the cultivators.

Object of Water Management The followings are the objects of water management:

(a) Supply of water The irrigation water should be supplied to the cultivators at the right time for the maximum yield of crop.

(b) Equitable distribution The irrigation water should be equally shared by the cultivators.

(c) Irrigation efficiencies Optimum irrigation efficiencies should be achieved to enhance agricultural product.

(d) Maintenance of irrigation system The maintenance of the irrigation system should be done properly to make the project economical.

(e) Sense of justice The sense of justice and equity among the cultivator should be evoked to obey the water management rules.

Methods of Water Management The following are the various methods of water management:

(1) Reservoir management The reservoir management involves the methods of reducing evaporation loss, absorption and percolation loss. To reduce evaporation loss the methods applicable should be properly implemented. The plants, weeds, etc on the periphery and reservoir surface should be cleaned perfectly to avoid absorption loss. Necessary measures should be taken to reduce the percolation loss at the time of constructing the reservoir itself.

(2) Conveyance system management Maximum water is lost during the conveyance of water from the reservoir to the command area. In unlined canals, the water is lost mainly by seepage. So, the canals should be lined to reduce the loss.

(3) Land development If the agricultural land is undulating the water is lost by trapping. So, the agricultural land should be properly levelled to obtain uniform growth of plants with optimum quantity of water.

(4) Application of right amount of water Over irrigation or under irrigation reduces the yield of crop. So, the right amount of water should be applied to the land which is just sufficient to meet the water requirement of crops.

(5) Method of irrigation Correct method should be recommended for the surface irrigation, as for example,

Border strip method—for paddy, wheat, vegetables, etc.
Furrow method—for cotton, sugarcane, potato, etc.
Basin method—for orchards.

(6) Crop rotation The crop rotation should be practised for maintaining the fertility of the lands for obtaining maximum yield with minimum supply of water.

(7) Methods of ploughing The deep ploughing by tractors is much efficient to retain water and to get maximum yield with optimum supply of water than the ordinary shallow ploughing by animals. So, deep ploughing plays an important role in water management.

17.16 WATER LAWS

In areas where the water supply is inadequate to meet the needs of the users or cultivators, the water is considered as an essential commodity. So, a system of laws has been developed to specify the rights of the owners of the land to avoid conflict. Water laws play a major role in the economic aspects of water management. The following are the major laws

1. Riparian Rights The following are the clauses of this right,

(a) Under the concept of this right the owner of the land adjacent to a river (riparian land) is entitled to receive the full natural flow of the river without changing its quality and quantity.
(b) The riparian owner is protected against the diversion of water from the upstream side of his or her property.
(c) No upstream owner may lessen or increase the natural flow of a river to create disadvantage to the downstream owner.

2. Appropriate Rights The following are the clauses of this right,

(a) The right of the earliest appropriator is superior to any other claim.
(b) Appropriation is possible if the water is available in excess of earlier claims.
(c) The claimants of the earliest priority are entitled to their full share and the owners of later priority may not have the share.
(d) An appropriator may store water in reservoir for using during the period of shortage, but the amount of storage should be limited by the terms of storage appropriation.
(e) Waste water discharge, discharge from flood mitigation and hydroelectric storage reservoir should be properly appropriated.

REVIEW QUESTIONS

1. Fill up the blanks with appropriate word/words,
   (i) The reservoir which is formed for irrigation, water supply, flood control, etc is termed as ________ reservoir.
   (ii) The maximum water level which is to be stored in a reservoir is known as ________
   (iii) The lowest allowable water level in a reservoir is known as ________
   (iv) The volume of water which is stored below the minimum pool level is known as ________
   (v) The volume of water stored in a reservoir between normal pool level and minimum pool level is known as ________

(vi) The volume of water which flows through the river before the construction of dam is known as ________

2. Distinguish between the following
   (a) Single purpose reservoir and multipurpose reservoir.
   (b) Storage reservoir and flood control reservoir.
   (c) Safe yield and secondary yield.
   (d) Mass curve and demand curve.
3. What are the different pool levels in a reservoir? Explain with sketch.
4. How can water be lost from a reservoir? How can the losses be controlled?
5. How does sedimentation occurs in a reservoir? How can sedimentation be controlled?

ANSWERS

1. (i) multipurpose (ii) normal pool level
   (iii) minimum pool level (iv) dead storage
   (v) live storage (vi) valley storage

18

WATER LOGGING AND WATERSHED MANAGEMENT

18.1 INTRODUCTION

In agricultural land, when the soil pores within the root zone of the crops get saturated with the subsoil water, the air circulation within the soil pores gets totally stopped. This phenomenon is termed as water logging. The water logging makes the soil alkaline in character and the fertility of the land is totally destroyed and the yield of crop is reduced.

Due to the heavy rainfall for a longer period or due to the continuous percolation of water from the canals, the water table gets raised nearthe surface of the soil. Then, by capillary action the water rises to the root zone of the crops and goes on saturating the soil. If this condition goes on for a longer period, the soil becomes alkaline and is damaging to the crops.

The height to which the soil water rises above the watertable by capillary action is known as capillary fringe. Generally, the height of capillary fringe varies from 1 m to 1.5 m. When the watertable comes to 1.5 m below the surface of the soil, the land is said to be water logged.

Again, due to heavy rainfall and poor drainage system the water goes on collecting in low lying areas. This stagnant water makes the land marshy. The weeds and aquatic plants gradually cover the whole area and the area becomes unsuitable for cultivation.

18.2 CAUSES OF WATER LOGGING

The following are the main causes of water logging:

(1) Over Irrigation In inundation irrigation since there is no controlling system of water supply it may cause over irrigation. The excess water percolates and remains stored within the root zone of the corps. Again, in perennial irrigation system if water is supplied more than what is required. This excess water is responsible for the water logging.

(2) Seepage from Canals In unlined canal system, the water percolates through the bank of the canal and gets collected in the low lying areas along the course of the canal and thus the water table gets raised. This seepage is more in case of canal in banking.

(3) Inadequate Surface Drainage When the rainfall is heavy and there is no proper provision for surface drainage the water gets collected and submerges vast area. When this condition continues for a long period, the water table is raised.

(4) Obstruction in Natural Water Course If the bridges or culverts are constructed across a water course with the opening with insufficient discharge capacity, the upstream area gets flooded and this causes water logging.

(5) Obstruction in Sub-soil Drainage If some impermeable stratum exists at a lower depth below the ground surface, then the movement of the subsoil water gets obstructed and causes water logging in the area.

(6) Nature of Soil The soil having low permeability, like black cotton soil, does not allow the water to percolate through it. So, in case of over irrigation or flood, the water retains in this type of land and causes water logging.

(7) Incorrect Method of Cultivation If the agricultural land is not levelled properly and there is no arrangement for the surplus water to flow out, then it will create pools of stagnent water leading to water logging.

(8) Seepage from Reservoir If the reservoir basin consists of permeable zones, cracks and fissures which were not detected during the construction of dam, these may cause seepage of water. This sub-soil water will move towards the low-lying areas and cause water logging.

(9) Poor Irrigation Management If the main canal is kept open for a long period unnecessarily without computing the total water requirement of the crops, then this leads to over irrigation which shall result in water logging.

(10) Excessive Rainfall If the rainfall is excessive and the water gets no time to get drained off completely, then a pool of stagnant water is formed which might lead to water logging.

(11) Topography of the Land If the agricultural land is flat, i.e. with no country slope and consists of depressions or undulations, then this leads to water logging.

(12) Occasional Flood If an area gets affected by flood every year and there is no proper drainage system, the water table gets raised and this causes water logging.

18.3 EFFECTS OF WATER LOGGING

The following are the effects of water logging:

(1) Salinization of Soil Due to water logging the dissolved salts like sodium carbonate, sodium chloride and sodium sulphate come to the surface of the soil. When the water evaporates from the surface, the salts are deposited there. This process is known as salinization of soil. Excessive concentration of salt makes the land alkaline. It does not allow the plants to thrive and thus the yield of crop is reduced. This process is also known as salt efflorescence.

(2) Lack of Aeration The crops require some nutrients for their growth which are supplied by some bacteria or micro-organisms by breaking the complex nitrogeneous compounds into simple compounds which are consumed by the plants for their growth. But the bacteria requires oxygen for their life and activity. When the aeration in the soil is stopped by water logging, these bacteria cannot survive without oxygen and the fertility of the land is lost which results in reduction of yield.

(3) Fall of Soil Temperature Due to water logging the soil temperature is lowered. At low temperature of the soil the activity of the bacteria becomes very slow and consequently the plants do not get the requisite amount of food in time. Thus, growth of the plants is hampered and the yield also is reduced.

(4) Growth of Weeds and Aquatic Plants Due to water logging, the agricultural land is converted to marshy land and the weeds and aquatic plants are grown in plenty. These plants consume the soil foods in advance and thus the crops are destroyed.

(5) Diseases of Crops Due to low temperature and poor aeration, the crops get some diseases which may destroy the crops or reduce the yield.

(6) Difficulty in Cultivation In water logged area it is very difficult to carry out the operation of cultivation such as tilling, ploughing, etc.

(7) Restriction of Root Growth When the water table rises near to root zone the soil gets saturated. The growth of the roots is confined only to the top layer of the soil. So, the crops cannot be matured properly and the yield is reduced.

18.4 CONTROL OF WATER LOGGING (i.e. ANTI WATER LOGGING MEASURES)

The following measures may be taken to control water logging:

(1) Prevention of Percolation from Canals The irrigation canals should be lined with impervious lining to prevent the percolation of water through the bed and banks of the canals. Thus the water logging may be prevented.

Intercepting drains may be provided along the course of the irrigation canals in places where the percolation of water is detected. The percolating water is intercepted by the drains and the water is carried to other natural water course.

(2) Prevention of Percolation from Reservoirs During the construction of dam, the geological survey should be conducted on the reservoir basin to detect the zone of permeable formations through which water may percolate. These zones should be treated properly to prevent the seepage. If afterwards it is found that there is still leakage of water through some zone, then sheet piling should be done to prevent the leakage.

(3) Control of Intensity of Irrigation The intensity of irrigation may cause water logging so, it should be controlled in a planned way so that there is no possibility of water logging in a particular area.

(4) Economical Use of Water If the water is used economically, then it may control the water logging and the yield of crops may be high. So, special training is required to be given to the cultivators to realise the benefits of economical use of water. It helps them to get more crops by eliminating the possibility of water logging.

(5) Fixing of Crop Pattern Soil survey should be conducted to fix the crop pattern. The crops having high rate of evapotranspiration should be recommended for the area susceptible to water logging.

(6) Providing Drainage System Suitable drainage system should be provided in the low lying areas so that the rain water does not stand for long days. A net work of sub-surface drains are provided which are connected to the surface drains. The surface drains discharge the water to the river or any water course.

(7) Improvement of Natural Drainage Sometimes, the natural drainage may be completely silted up or obstructed by weeds, aquatic plants, etc. The affected section of the drainage should be improved by excavating and clearing the obstructions.

(8) Pumping of Ground Water A number of open wells or tube wells are constructed in the water logged area and the ground water is pumped out until the water table goes down to a safe level. The lifted ground water may be utilised for irrigation or may be discharged to the river or any water course.

(9) Construction of Sump Well Sump wells may be constructed within the water logged area and they help to collect the surface water. The water from the sump wells may be pumped to the irrigable lands or may be discharged to any river.

18.5 LAND RECLAMATION

The reclamation of land is the process of making a land culturable after it gets converted to unculturable area due to the bad effect of water logging. The following are the general methods of land reclamation,

(1) Leaching Leaching is a process for reclamation of the saline soil. In this process, the agricultural land is flooded with water to a depth of about 20–30 cm. The salt deposits on the surface are dissolved. Some portion of salt is then drained off through the sub-soil drainage system and some portion is removed by surface drainage system. This operation is repeated several times at specified intervals. This operation may be repeated next year also if the yield of crop is not satisfactory.

Generally, leaching is followed by crop rotation system as recommended by agricultural department.

(2) Addition of Chemical Agent For improving the alkaline soil a chemical like gypsum is generally added with irrigation water. The gypsum neutralises the alkaline effect of the soil and yield of the crop is increased. The application of gypsum is not necessary every year.

(3) Surface Drainage Proper surface drainage system should be provided in the agricultural land so that the water does not accumulate for a long time. The surface drains also help in draining the saline water in case of leaching operation.

(4) Sub-surface Drainage The sub-surface drainage system on the agricultural land should also be provided for draining the excess water from the root zone. It also helps in draining of saline water in case of leaching operation.

(5) Addition of Waste Products Waste products like ground nut shells, saw-dust, etc are added to the alkaline soil and these are very effective in removing the salinity of the soil. The distillary waste also is found very effective in removing the salinity of soil.

(6) Excavation of Ponds Ponds are excavated at suitable places within the water logged area. The excess run-off is collected in the ponds. The pore water also flows towards the pond and thus the saturation in the root zone of the crops is reduced. In fact, these ponds control the water logging in rainy season and in dry season the water of the ponds may be utilised for lift irrigation.

(7) Pumping of Water From Tube Wells Some tube wells (deep or shallow) are sunk within the water logged area. The water is pumped continuously from the tube wells. Initially this water is discharged to a river or pond.

When the reclamation of the land is complete, the water may be utilised for lift irrigation.

18.6 WATERSHED MANAGEMENT

The area enclosed within the watershed line is known as watershed area. The watershed area may consist of hilly area and plane area. In hilly area there is no possibility of water logging, but the sediments carried by the tributaries badly affect the downstream plane area. In the plane area the rivers, streams, etc. are silted up by the sediments and the water carrying capacity is reduced. Consequently, the surrounding areas may be submerged during the flood. Sometimes, due to lack of proper drainage system the vast area may get water logged.

The cultivation becomes impossible and it creates problems for the villagers.

It is also found that the low lying areas of some villages get submerged due to heavy rainfall and the are remains under water for a long period. During this period, the agriculture in the area is totally stoped. But in dry season, when the whole area is dried up the villagers depend on the shallow tube well or deep tube well for irrigation which works out to be expensive for them. So, the watershed management becomes very important in village areas now-a-days. The watershed management may be done in two ways.

(1) Preventive Measures The following steps should be taken:

(a) Contour bunds or terrace bunds Along the slope of the hilly catchment area contour bunds or terrace bunds should be constructed at different levels. These bunds form the water pockets which arrest the sediments and serve as detention basins for the heavy run-off during the rainy season.

(b) Small dams Small dams are constructed across the tributaries of a river and even on the river at the upstream region to form small reservoirs where sediments are arrested and the flood water is detained.

(c) Soil conservation The soil conservation methods should be adopted in the catchment area. These methods include the method of afforestation, prevention of deforestation, control of grazing, etc.

(d) Slip stabilisation The slips or land slides in the catchment area should be detected and effective methods are employed to stabilise them.

(e) Control of cultivation in catchment area Cultivation should be done in controlled way with shallow marginal bund along the boundary of specified land so that the loose soil is not washed out by rain water and carried to the downstream area.

(2) Curative Measures The following steps should be taken:

(a) Remodelling work The levelling should be done along the stream, *nalla* or *khari* and the excavation of these water ways should be done with proper depth and slope so that the water flows easily towards any river or other water course.

(b) Construction of water pockets While remodelling the water course or stream some deep excavations are made on the bed at some interval, along its course where small water pockets may develop. These water pockets may serve as small storage reservoir for irrigation in dry season.

(c) Diversion channel The source of water flowing towards the low lying area should be detected and diversion channel should be constructed to discharge water to the nearby river or any other water course.

(d) Link channel The water accumulated in low lying area may be discharged to large natural pond (i.e. Bil or river) by constructing link channel after properly surveying the area.

(e) Construction of Pond Ponds may be excavated at suitable places within the low lying area where the major portion of water may be collected. This water, may be used for irrigation in dry season.

(f) Construction of Sump Well Sump wells may be constructed at suitable places within the low lying area. These wells carry the surface water and bring down the water table. These wells may also form the underground source of water for irrigation in dry season.

REVIEW QUESTIONS

1. Fill up the blanks with proper word/words
   (i) When the soil pores get saturated by sub-soil water and air circulation within the soil pores is stopped, then it is termed as ___________
   (ii) When dissolved salts come to the surface of the soil and get deposited there, then it is termed as ___________
   (iii) The process of reclaimation of saline soil is known as ___________
   (iv) Over irrigation is responsible for ___________
   (v) The crops require some nutrients for their growth which are supplied by some ___________ by breaking the complex nitrogeneous compounds.
2. What is meant by waterlogging? State the effects of water logging.
3. Describe the methods of controlling water logging.
4. Enumerate the process of reclaimation of land effected by water logging.
5. Describe the methods of watershed management.

ANSWERS

1. (i) Water logging (ii) Salinization
   (iii) Leaching (iv) Water logging
   (v) Bacteria

19

RIVER GAUGING

19.1 INTRODUCTION

The river gauging involves the measurement of discharge of a river and the establishment of gauge post on one of its banks. The reading on the gauge post directly gives the reduced level of the water surface of the river, because the R.L. of the zero mark of the gauge is already ascertained with respect to the bench mark. For example, if the R.L. of the zero mark of the gauge post was found out as 100.000 m. If the gauge reading shows 3.5 m, then R.L. of water surface is 103.500 m; when gauge reading is 4.5 m, the R.L. of water surface is 104.500 m and so on.

The reduced level of the highest flood level can be directly obtained by simply noting the gauge reading. After noting the H.F.L. from the gauge post necessary warning should be given to the surrounding and the downstream area. Again, by noting the gauge reading the probable discharge through the river may be obtained.

19.2 NECESSITY OF RIVER GAUGING

The river gauging is undertaken for the following reasons,

1. For fixing the number of spans of road and railway bridges from the informations obtained by river gauging, so that the unexpected high flood may not cause nay damage to the structure.
2. For fixing the height of the guide bank.
3. For fixing the spill way level and the height of dam or barrage.
4. For designing the cross drainage works when a canal crosses a river.

19.3 SELECTION OF GAUGE SITE

The following points should be considered while selecting a gauge site

1. The river section should be well defined.
2. The river width should be minimum at the site.

3. The site should not be selected on the curve of the river.
4. There should be no scouring effect on the river bank.
5. There should be no obstructions at the upstream side of the site.
6. The site should remain clear to a distance of about 100 m on both upstream and downstream sides.

19.4 FIXING OF GAUGE POST

Gauge posts are wooden posts of section 10 cm × 5 cm and length 2 m. The posts are graduated in metres and fractions of metre so that minimum reading up to 0.01 m can be taken. The number of posts depend on the depth of the river. The posts are fixed on concrete foundation along the slope of the bank. The bottom post is marked with zero at the base and then the other posts are graduated accordingly.

The R.L. of the zero mark of the starting gauge post is determined by fly levelling from a bench mark. Then by noting the reading on the gauge post, the R.L. of the water level can be predicted.

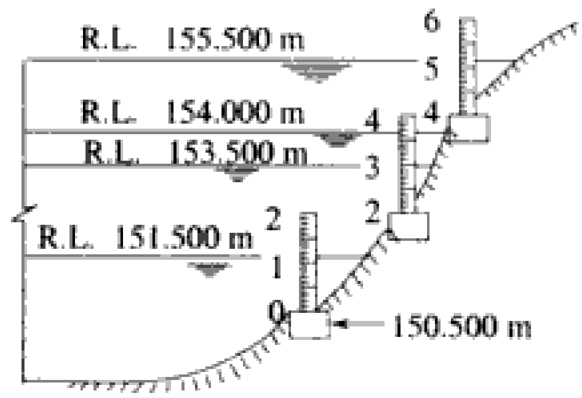


Fig. 19.1 Gauge posts

19.5 SELECTION OF DISCHARGE SITE

The following points should be considered while selecting a discharge site.

1. The site should not be on the curve of the river.
2. The site of the river should be straight for a minimum length of about 500 m or four times the width of the river.
3. The width of the river should be minimum with well defined banks.
4. The flow of the river should be confined to a single channel.
5. There should be no bifurcation of flow at the site.
6. The selected site should be marked with targets on both banks.
7. Two more sections are marked on the upstream and downstream side of discharge site at equal distance (say 100 m) to mark the 'run' of the float which will be required at the time of measurement of velocity.

19.6 MEASUREMENT OF DISCHARGE BY AREA-VELOCITY METHOD

In this method, the area of the water section of the river is calculated by measuring the depth of water by sounding rod, sounding cable, etc. and the velocity of flow is measured by floats, sub-surface floats, current metre, etc then the discharge is calculated by,

$$Q = A \times V$$

1. A theodolite is set up at 'A' and the line AB is ranged properly. Then by stadia method the width of the river is determined.
2. The width of the river is divided into equal compartments, say P_1 P_2, P_2 P_3, P_3 P_4 ..., etc.
3. The distances AP_1, AP_2, AP_3,... etc are computed.
   Then the angles α_1, α_2, α_3 ... etc are calculated as follows

$$\alpha_1 = \tan^{-1} \frac{AP_1}{AC}$$

$$\alpha_2 = \tan^{-1} \frac{AP_2}{AC}$$

   and so on.
4. A theodolite is set up at C and the angle α_1, is set out perfectly. When the line of sights from the theodolites at A and C coincide, then the point, P_1 is marked.
5. Similarly, the angles α_2, α_3, ... etc are set out successively by the theodolite at C and the points of intersection are marked as P_2, P_3 ... etc.
6. The points P_2, P_3, P_4 ..., etc are marked by floats which are tied with rope along the centreline AB and anchored at the bottom of the river so that the points may not be deflected from its true position.
7. The depths of water corresponding to the points P_2, P_3, P_4 are measured by sounding cable method or echo sounder method.
8. Suppose, the common distance between the floats is b and d_1, d_2, d_3 ..., etc. are the depths of water. Then the area is calculated as usual by considering end compartments as triangles and other compartments as trapezium (Fig. 19.6).

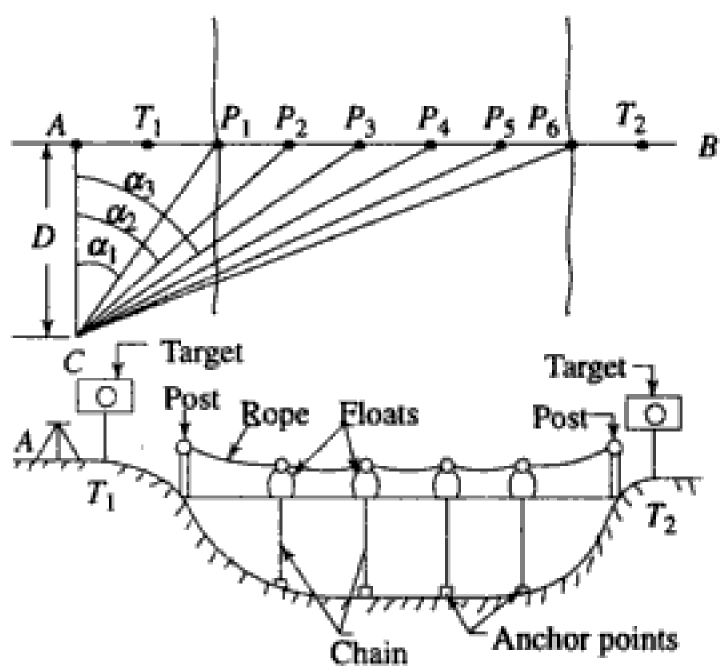


Fig. 19.6 Cross-section of large river

19.9 MEASUREMENT OF VELOCITY OF FLOW

Before the measurement of the velocity of flow, two cross-sections are marked on the upstream and downstream side at equal distance (say 50 m) from the discharge site. The distance between these two cross-sections is known as 'run'. The run is required for the measurement of velocity of flow by float method (Fig. 19.7).

The velocity of flow can be measured by the following methods:

(1) The Surface Float Method The surface floats are made of cork which can easily float on water. These are generally in the form of a 10 cm cube. The floats are painted red or white. Sometimes a small flag is fixed on the top (Fig. 19.8). The velocity is measured in the following way,

(a) The float is released slightly ahead of the upstream cross-section (Fig. 19.7). When the float just crosses the u/s cross-section, the stop watch is started. The observer should move along with the movement of the float and when it just crosses the D/S cross-section, the stop watch is stopped. The time taken by the float to cover the 'run' (known distance) is noted. From this the velocity of flow is calculated.

(b) Similarly, the velocities of all the compartments (as shown in Fig. 19.5) are measured. The average of these velocities is taken as the surface velocity. The actual mean velocity of flow is obtained by multiplying the surface velocity by a coefficient which varies from 0.95 to 0.98.

(c) Sometimes, to get more accurate result, the observations are taken twice or thrice for each compartment. The mean of all the observations is taken as the surface velocity.

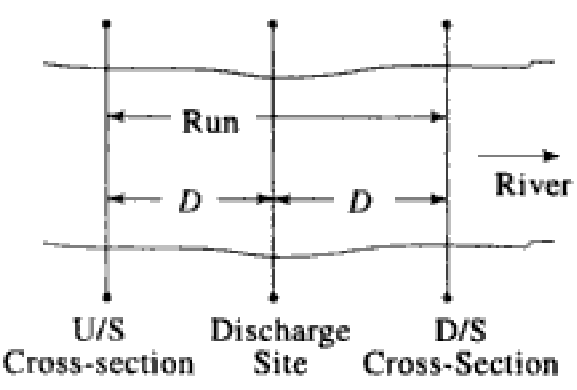


Fig. 19.7 Demarcation of discharge site

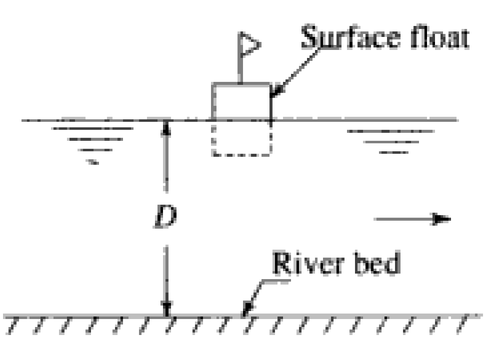


Fig. 19.8 Surface float

(2) The Sub-Surface Float Method The sub-surface float is a hollow metal cylinder which is attached by a cord to the surface float. The sub surface float remains submerged in water to a height 0.2 D m above the bottom, where D is the depth of water in the river. Here, the movement of the surface float is guided by the movement of the sub-surface float (Fig. 19.9) the procedure of measurement is as follows:

(a) The floats are released slightly ahead of the upstream cross-section (Fig. 19.7). When the floats just cross the u/s cross-section, the stop watch is started.

Let the rise in water level be h, then by applying Bernoullis theorem between the points A and B.

$$H + \frac{v^2}{2g} = H + h$$

or
$$V = \sqrt{2gh}$$

So, by measuring the rise of water column the velocity of flow can be calculated. Here, the pitot tube is held along the centreline of the discharge site and at the mid point of each compartment. In this case also the measured velocity gives the required mean velocity.

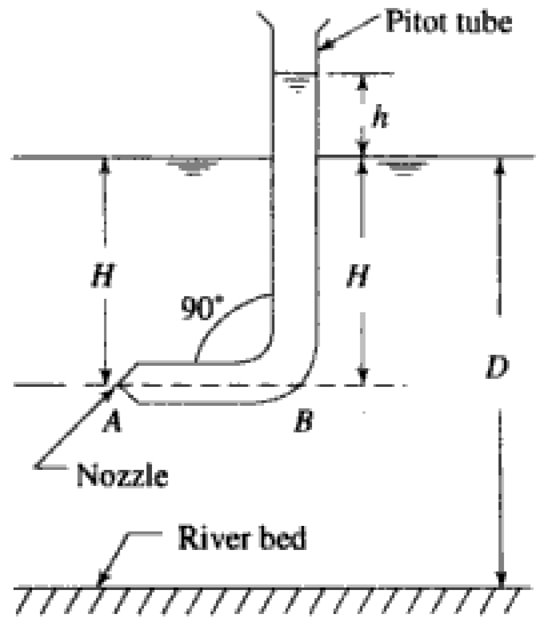


Fig. 19.11 Pitot tube

(5) The Current Meter Method The price current meter is commonly used for measuring the velocity of flow of the river. It consists of the following components (Fig. 19.12).

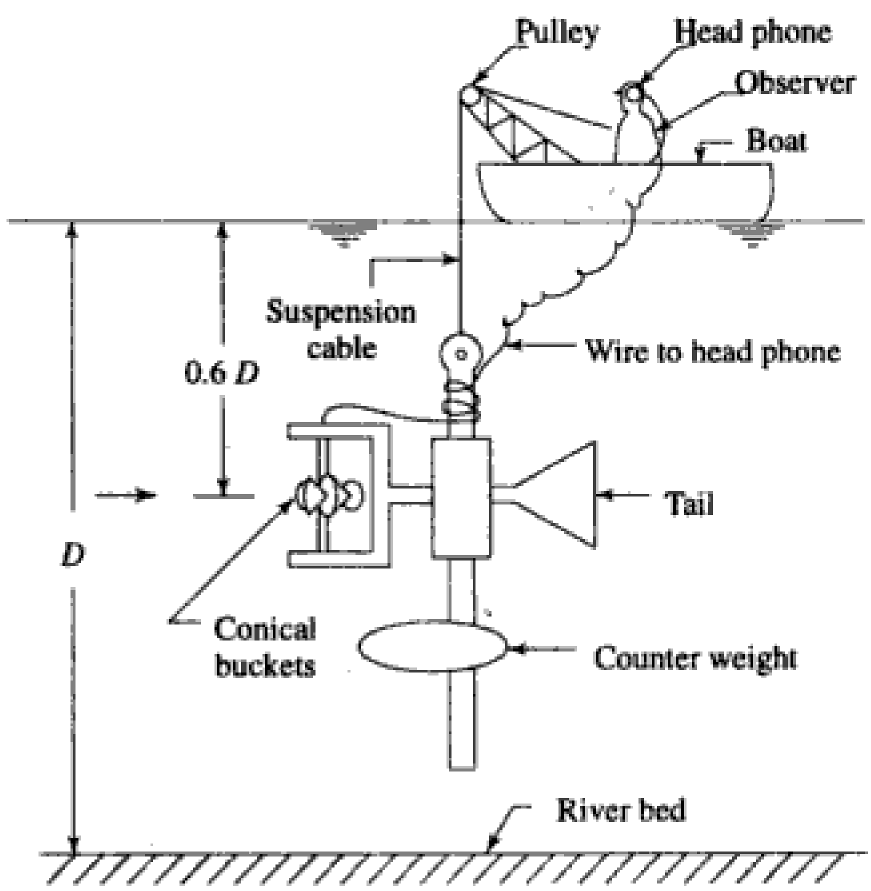


Fig. 19.12 Price current meter

5. Describe the method of measuring the velocity of flow in a river by current meter.

ANSWERS

1. (i) reduced level
   (ii) curve
   (iii) sounding rod
   (iv) echo sounder
   (v) surface float
   (vi) pendants
   (vii) 0.6 D

20

RIVER TRAINING WORKS

20.1 INTRODUCTION

River is the most important source for irrigation water supply, hydroelectric power generation, navigation, etc. The river may be regarded as a best friend of human life. But, if river is not properly managed it may cause extreme misery to human life by changing its course, by devastating vast area, by destroying bridges and hydraulic structures, etc.

That is why, the behaviour of the rivers should be studied thoroughly and necessary protection works (i.e. training works) should be done so that the river may not cause any destruction sufferings to human life.

A river has several stages starting from its origin to the point of out fall in the sea or ocean. The nature of the river is different at different stage. So, the characteristics of these stages should be known to arrest the destroying tendency of the river.

20.2 STAGES OF RIVER FLOW

The following are the stages of river flow,

(1) Rocky Stage The portion of the river within the steep hilly region is termed as rocky stage. In this stage, the bed slope of the river is steep and the velocity of flow is very high. The cross-section of the river is formed by scouring or eroding the rock formation. Hence, the cross-section is narrow and deep. The course of the river is guided by the deep gorge valley.

(2) Boulder Stage When the river reaches the base of the hilly area, then it is known as boulder stage. At this stage, the bed slope and the velocity of flow is reduced to some extent. The cross-section is formed by boulders and some permeable soil the sub-soil water flows in this region. Here, the width of the river is greater than that of the rocky stage.

(3) Trough Stage or Alluvial Stage When the river flows through the plain terrain, then it is known as trough stage or alluvial stage. In this stage, the cross-section of the river is formed by the alluvial soil. Both velocity of flow and the bed slope is low. The width of the river is large and some times the banks are not well defined. The course of the river is very zig-zag. The water of the river contains much silt which is very useful in increasing the crop yield on the agricultural land.

(4) Delta Stage As the river approaches the ocean, the formation of delta, i.e. shoal starts due to the very low velocity of flow. This region is known as delta stage. Here, the velocity of flow is unable to carry the sediment loads which are deposited at the junction point forming delta. The river course is bifurcated into several channels by a number of deltas.

20.3 NECESSITY OF RIVER TRAINING WORKS

The following are the reasons for river training works,

(a) In trough stage, the tendency of river is to change its course frequently. So, proper protection work should be adopted at the place from where the change of course is suspected.

(b) In rainy season, the flood water may submerge the vast cultivated and inhabited area by overflowing the banks of the river. So, proper protection works should be provided so that flood water do not overtop the banks.

(c) The scouring or erosion effect of the river water may damage structures like bridge, culvert, barrage, weir, etc. So, protection works should be provided to eliminate the scouring effect.

(d) In the curve of a river, one bank goes on eroding continuously and the villages or towns in that bank may get washed out. So, to protect the villages, towns or valuable agricultural land appropriate measures should be taken.

20.4 MEANDERING OF RIVER

In trough stage, the river course takes the shape of a serpentile curve due to the formation of shoals in both the banks in a zig-zag manner which is known as meandering of river. The following are the causes of meandering:

1. When a river carries heavy sediment, then it tends to form shoals on both banks irregularly by depositing the sediments on the bed and thus meander may be formed.
2. Again, the river banks may be affected by erosion unequally which tends to deviate the axis of the river in a zig-zag way and thus meander may be formed (Fig. 20.1).

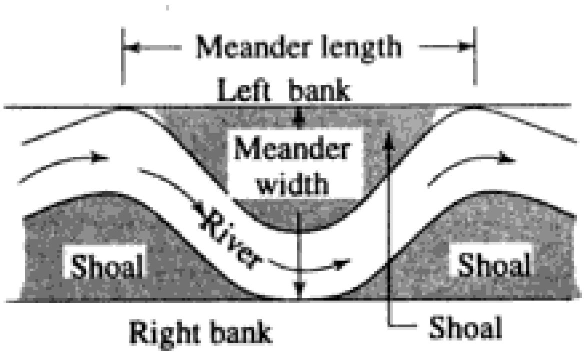


Fig. 20.1 Meander formation.

20.5 COMPONENTS OF RIVER TRAINING WORKS

The following are the different components of the river training works (Fig. 20.2).

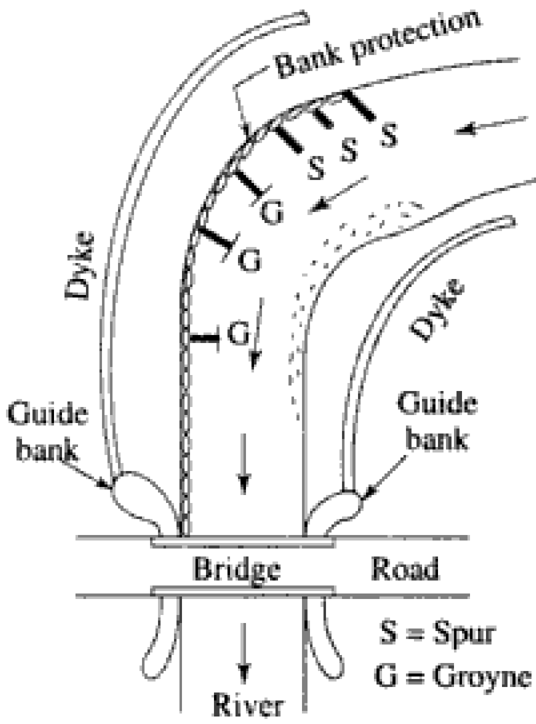


Fig. 20.2 River training works

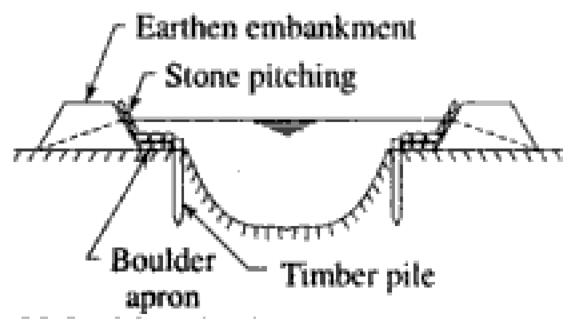


Fig. 20.3 Marginal embankment or dyke

20.6 MARGINAL EMBANKMENT (DYKE OR LEVEE)

The marginal embankment or dyke is an earthen embankment of trapezoidal section constructed approximately parallel to the bank of the river to confine the flood water within a section between the embankments. The embankment may be constructed on one or both the banks according to the conditions. A margin of about 2 m is provided between the toe of the embankment and the top edge of river bank (Fig. 20.3). Sometimes, due to some difficulties the embankment may run away from the river bank. In such a case the river side area remains unprotected in floods during rainy season. The height of the embankment depends on the H.F.L. of the river. The top width varies from 2 m to 3 m the side slope of the embankment varies from 2:1 to 3:1 according to the characteristics of the available soil. Timber piles (generally sal ballah) are driven about 1 m away from the toe of the embankment at an interval of 1 m, centre to centre. This is to stabilise the soil and to resists the scouring effect of the river water. The piles are kept projected about 50 cm above the ground level. The space between the pile line and the toe of the embankment is pròperly packed with boulders, i.e. boulder apron. The river side slope is protected by stone pitching with cement grouting. The country side slope is protected by turfing. The country side slope should be such that a cover of 50 cm on the hydraulic gradient line may be available. The marginal embankment serves the following purposes,

1. It protects the towns, villages, agricultural land, etc from flood water.

2. It provides roadway on the top which serves the purpose of communication.
3. In case of barrage, the marginal embankment retains the storage water within a specified section.

20.7 GUIDE BANK (See Chapter 10, sec. 10.13)

20.8 SPURS

These are temporary structures permeable in nature provided on the curve of a river to protect the river bank from erosion. These are projected from the river bank towards the bed making angles 60° to 75° with the bank of the river. The length of the spurs depend on the width of the river and the sharpness of the curve. The function of the spurs is to break the velocity of flow and to form a water pocket on the upstream side where the sediments get deposited. Thus the reclamation of land on the river bank can be achieved. The spurs may be of the following types,

(a) Bamboo Spur In this type of spur, a box like compartment is prepared by driving bamboo piles at 15 cm centre to centre. The piles are secured by bamboo bracings. The hollow space is filled up with sand bags. It is permeable in nature and water can seep through its body. This type of spur is suitable for small rivers. This is purely temporary and requires repair work every year. The length of bamboo piles depends on bed condition (Fig. 20.4).

(b) Timber Spur In this type, a box like compartment is prepared by driving timber piles at 15 cm to 30 cm centre to centre. The piles are secured by wooden bracings. The hollow space is filled up by boulders. This spur is permeable but stable. This is recommended for big rivers with high velocity of flow (Fig. 20.5). The length of the timber piles depend on bed condition.

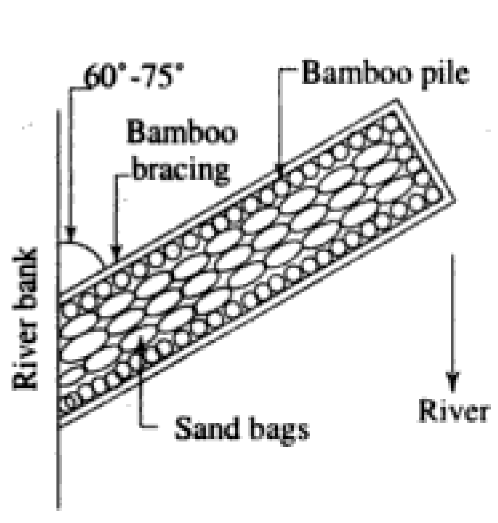


Fig. 20.4 Bamboo spur

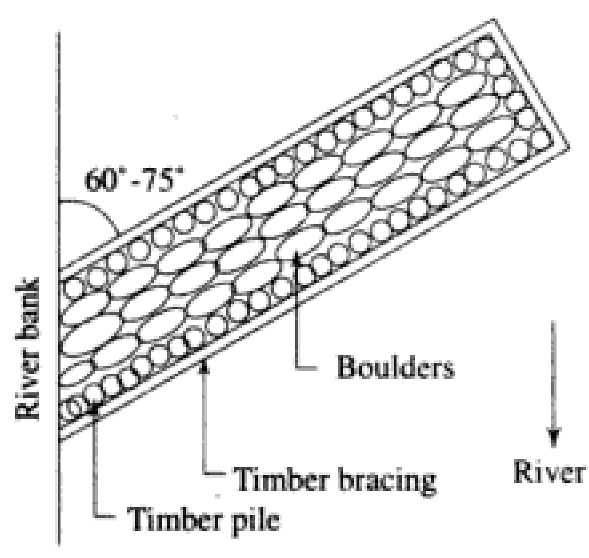


Fig. 20.5 Timber spur

(c) Boulder Spur In this type of spur, boulders are enclosed in G.I. wire net in circular shape. The boulders should be heavy, varying from 30 kg to 50 kg and the wire net should be made of 4 mm diameter G.I. wires. It is laid from the river bank towards the bed making an angle of 60°–75° with the bank. This type of spur is recommended for the rivers where velocity of flow is very high (Fig. 20.6).

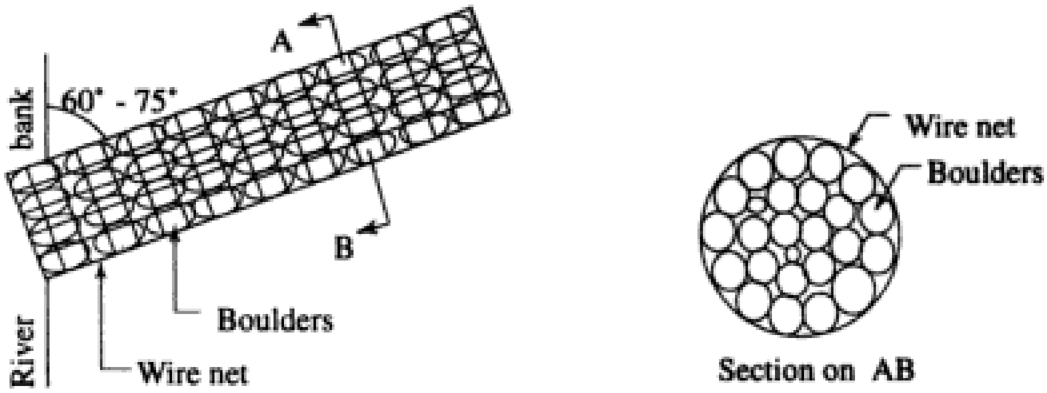


Fig. 20.6 Boulder spur

20.9 GROYNES

The function of groynes is similar to that of spur. But these are impervious permanent structures constructed on the curve of a river to protect the river bank from erosion. They extend from the bank towards the bed by making an angle of 60° to 75° with the bank. The angle may be towards the upstream or downstream. Sometimes, it is made perpendicular to the river bank. These are constructed with rubble masonry in trapezoidal section and the surface is finished with stone pitching or concrete blocks. The stone pitching or the concrete blocks are set with rich cement mortar. The length of the groyne depends on the width and nature of the river. The top width varies from 3 m to 4 m. The side slope may be $1\frac{1}{2}$: 1 or 2 : 1. The groynes are provided in series throughout the affected length of the river bank. The spacing between the adjacent groyne is generally kept as $2L$, where L is the length of the groyne. These are recommended for the river where the permanent solution of erosion control is extremely necessary (Fig. 20.7).

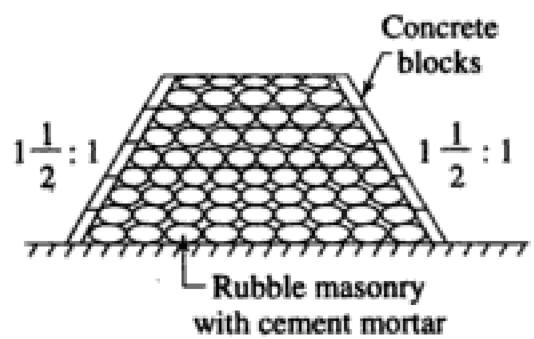


Fig. 20.7 Section of groyne

The groynes may be designated as follows.

(a) Attracting Groyne The groyne which is constructed obliquely to the bank by making an angle of 60° to 75° towards the downstream, is known as attracting groyne. here the flow of water is attracted towards the bank, and the velocity of flow is reduced to such a extent that it cannot cause any erosion to the bank. However, a bank protection of stone pitching is provided for safety (Fig. 20.8).

(b) Repelling Groyne A groyne which is aligned towards upstream at an angle of 60° to 75° with the river bank is known as repelling groyne. A still water pocket is formed on the upstream where silting takes place. Here, the bank protection is not necessary, because the flow of water does not touch the bank and there is no effect of erosion on the bank (Fig. 20.9). But still boulder pitching should be provided for safety.

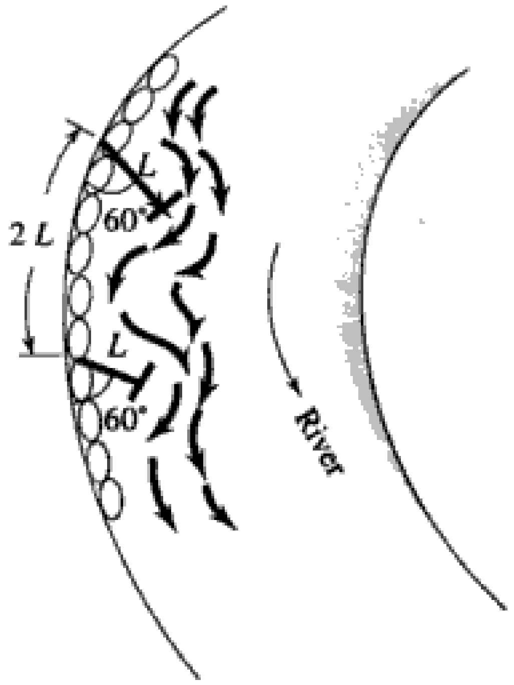


Fig. 20.8 Attracting groyne

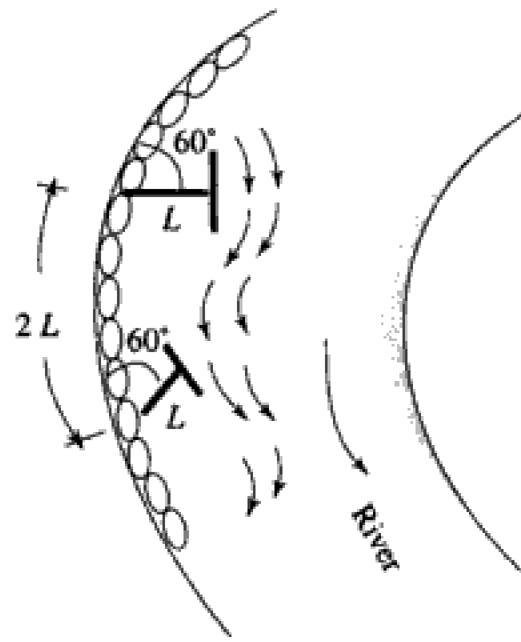


Fig. 20.9 Repelling groyne

(c) Deflecting Groyne The groyne which is constructed perpendicular to the river bank is known as deflecting groyne. Here the flow of water is deflected from the bank by the perpendicular obstruction i.e. groyne. The flow of water follows a undulating path just outside the head of the groyne. An eddy current is formed on the upstream side of the groyne. This eddy current will not affect the river bank. But the bank protection is provided for safety (Fig. 20.10).

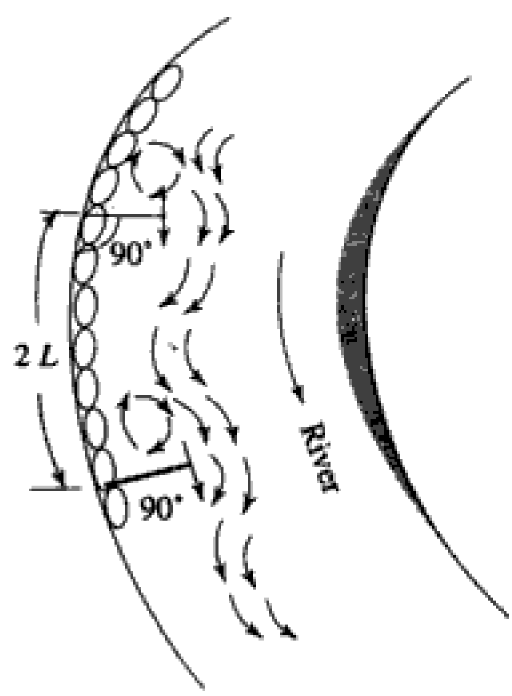


Fig. 20.10 Deflecting groyne

Modification of Groyne

The groynes are modified as follows:

(a) Denehy's Groyne or T-Headed Groyne After long investigation in different sites, Denehy developed a groyne in the shape of a *T*. The length of the head is kept as $\frac{L}{2}$, where L is the length of groyne. A still water pocket is formed on the upstream side where silting takes place. It is constructed with rubble masonry in trapezoidal section. The upstream face is finished with concrete blocks with cement mortar (Fig. 20.11).

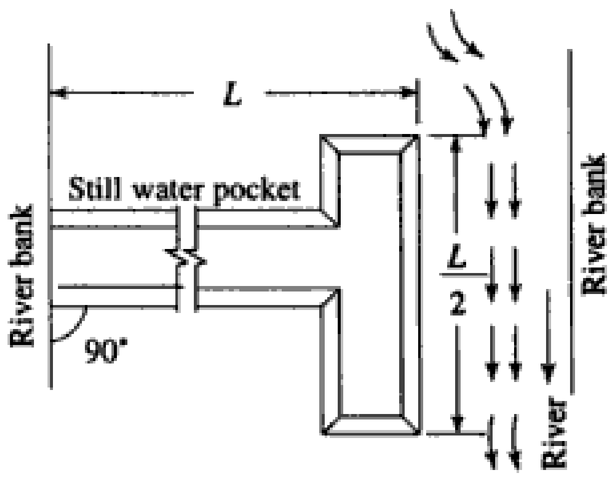


Fig. 20.11 Denehy's groyne

(b) Hockey Head Groyne Another development is hockey head groyne. Here, the head of the groyne is curved towards the downstream in the shape of a hockey stick. It behaves like an attracting groyne.

But it allows the water to flow smoothly over the head of the groyne. It is also constructed with rubble masonry in trapezoidal section. Here, the bank protection by stone pitching is necessary (Fig. 20.13).

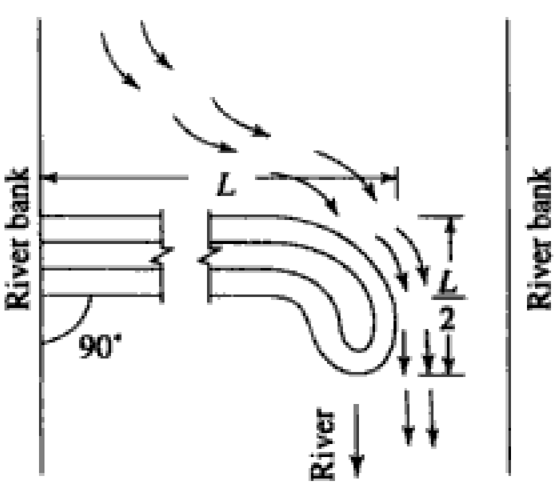


Fig. 20.12 Hockey head groyne

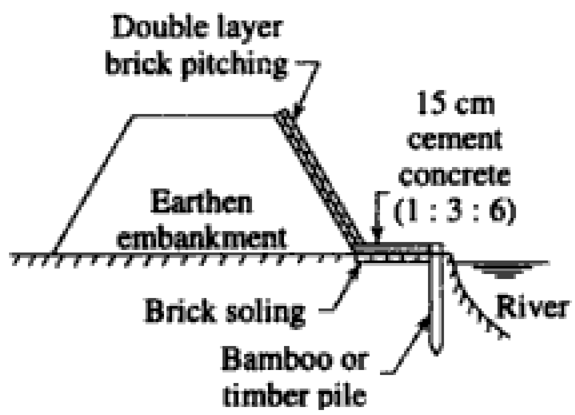


Fig. 20.13 Brick pitching

20.10 COMPARISON BETWEEN SPUR AND GROYNE

Spur	Groyne
(1) It is a temporary structure	(1) It is a permanent structure,
(2) It is permeable	(2) It is impermeable.
(3) It is constructed with bamboo pile, timber pile, sand bag, boulders, etc.	(3) It is constructed with rubble masonry with cement mortar.
(4) It requires repair works.	(4) It does not require any repair work.
(5) It is recommended for small rivers.	(5) It is recommended for large rivers.
(6) It is useful for low or medium velocity of flow.	(6) It is useful for high velocity of flow.

20.11 BANK PROTECTION

The following are the different types of bank protection,

(1) Brick Pitching In this type of bank protection bamboo or timber piles of length 3 m are driven at 15 cm centre to centre along a line about 1 m away from the toe of the embankment. Cement concrete (1:3:6) of thickness 15 cm is laid over a brick flat soling on the space between the toe and the pile line. The sloping side is protected by double layer brick pitching with cement mortar (1:6) (Fig. 20.13). This figure is not shown. See sheet 42 Fig. 20.13.

(2) Stone Riprap In this type of bank protection, timber piles of length 3 m are driven at 1 m centre to centre along the line about 1 m away from the toe of the embankment. The piles are projected about 45 cm above the ground surface. Then sausage work (boulders enclosed in wire net) is done along the space between the toe and the pile line, the sloping side is provided with stone riprap which is finished with cement mortar (Fig. 20.14).

(3) Boulder Pitching In this type of bank protection, timber piles of length 4 m to 5 m are driven at 1 m centre to centre along the line about 1 m away from the toe of the embankment. The piles are projected about 50 cm above the ground surface. Then two layers of boulder apron is provided within the space between the toe and the pile line. The sloping side is lined with boulder pitching. The surface is finished with cement grouting (Fig. 20.15).

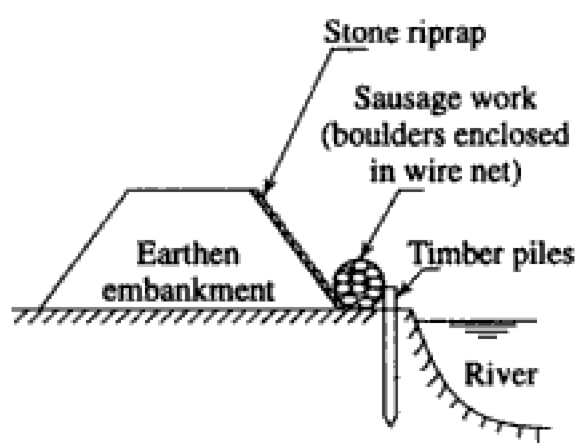


Fig. 20.14 Stone riprap

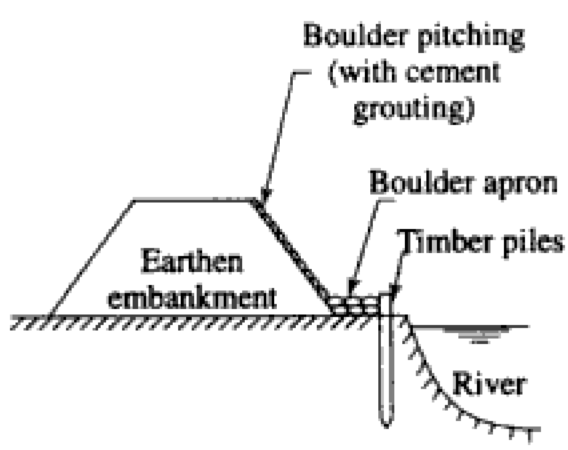


Fig. 20.15 Boulder pitching

(4) Concrete Slab Lining In this type of bank protection, a toe wall is constructed along the bank of the river. Concrete slabs are set with cement mortar within the space between the toe of the embankment and toe wall. The sloping side is lined with concrete slabs. The joints are finished with cement mortar. Concrete slabs may be of various dimensions according to the site condition. Generally, concrete slabs of size 50 cm × 50 cm × 10 cm are used (Fig. 21.16).

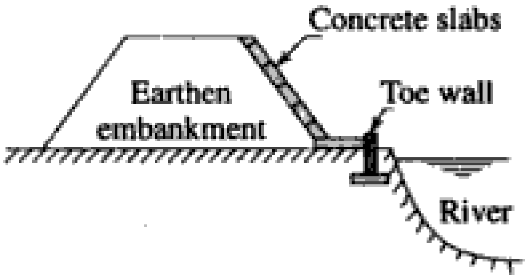


Fig. 20.16 Concrete slab lining

20.12 Reclamation of Land In the course of a river, it has been observed that the river destroys one bank by erosion and shoals are formed on the other bank. Thus, many villages towns, valuable agricultural lands, etc are found to be destroyed when the river tries to change its course. Suppose, a river was originally flowing along the bank $A\ B\ C\ D$. But it goes on extending it's one bank every year and ultimately damages many villages, agricultural land and takes the bank line along $A\ B_1\ C_1\ D$. Here, we shall study, how the bank erosion may be controlled and the land can be reclaimed.

The following methods are suggested to reclaim the land.

(a) The entire affected length $A\ B\ C\ D$ is divided into several compartments (10 m × 1.5 m) by timber piles leaving a gap of 1 m between the adjacent compartments.

(b) Timber piles are driven at 50 cm centre to centre on both sides and ends of the compartments. Thus, a hollow box is formed which is filled up with boulders (30 kg to 50 kg).

(c) During flood, the sediment laden water enters the affected area through the gaps.

(d) The still water pocket which is formed at the affected area causes the sediments to settle down every year.

(e) After several years it will be found that the whole affected area has been completely silted up (Fig. 20.17).

Thus the river will come back to its original course ABCD again.

REVIEW QUESTIONS

1. Fill up the blanks with appropriate word/words.

   (i) When river flows through the steep hilly region, then that stage is known as __________

   (ii) When a river flows through the plain terrain, then that stape is known as __________

   (iii) Due to the unequal erosion of the river banks, the __________ section is developed in the river course.

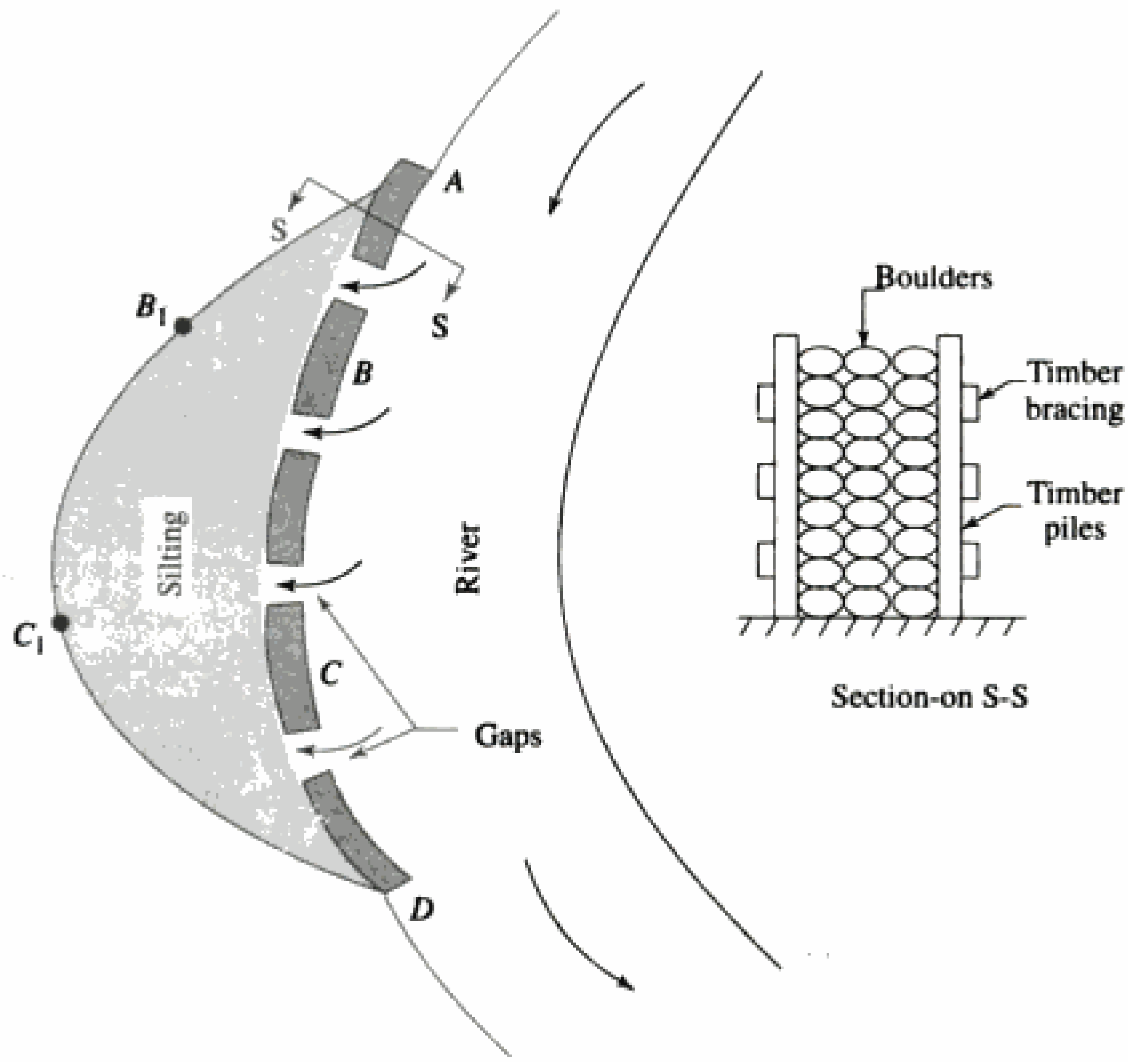


Fig. 20.17 Reclamation of land

(iv) The permeable temporary structure constructed on the river bank to protect it from erosion is known as ___________

(v) The impermeable permanent structure constructed on the river bank to protect it from erosion is known as ___________

(vi) The groyne which is constructed obliquely to the bank at an angle 60° towards the downstream is known as ___________ groyne.

2. What is meant by meandering of a river? How is it formed?
3. Distinguish between the following
   (a) Spur and groyne
   (b) Attracting groyne and repelling groyne.
   (c) Turfing and riprap.
4. Describe a guide bank with a neat sketch.
5. What are the different types of bank protection? Describe with a neat sketch.

21

FLOOD CONTROL WORKS

21.1 INTRODUCTION

In rainy season, when heavy rainfall occurs in the catchment area, the discharge of the river is increased and sometimes it exceeds the normal carrying capacity of the river. Then the surplus water overtops the bank of the river and submerges the surrounding areas consisting of villages, agricultural lands, etc. This phenomenon is known as flood.

The nature is beyond the control of human being. Heavy rainfall, tornedo, cyclone, etc cannot be controlled and the natural calamities keep occurring. The problem of flood damage is universal. Every country has to face the calamity due to flood. River Nile of Egypt, Hoang Ho of China and Padma are rivers which spell misery on their respective countries.

The causes of flood should be ascertained by different investigations and precaustionery measures should be taken to reduce the destructive effect of the flood. The civil engineer should investigate the sources of flood. He should know the methods of flood forecasting, evacuation system, flood mitigation works, etc. Now we shall study the different aspects of the flood in the following sections.

21.2 CAUSES OF FLOOD

The following are the causes of flood:

(1) Intensity of Rainfall in Catchment Area The intensity of rainfall in the catchment area is the main cause of the flood. If the rainfall is normal and the storm duration is short, then the surface run-off will flow down smoothly through the tributaries and rivers will not create any trouble to the downstream side. But if the rainfall is very heavy and the storm duration is longer, then the surface run-off will be increased unexpectedly and it may exceed the normal carrying capacity of the river and hence overtopping of the river bank may occur and the surrounding area may get submerged.

(2) Topography of the Catchment The catchment area with steep slope increases the run-off and increases the sediment inflow due to the high velocity of flow. While the catchment area with flatter slope reduces the run-off and reduces the sediment inflow due to the low velocity of flow. So, the topography of the catchment area directly affects the discharge of the river.

(3) Sedimentation of Rivers If the tributaries of a river carry heavy sediment load the river bed goes on silting up gradually every year. Thus, the carrying capacity of the river goes on reducing every year. Ultimately the cross-section of the river will be shallow and it will not be able to carry the high flood discharge. The sedimentation of the river also is responsible for the flood.

(4) Obstruction in the River Flow In hilly catchment area, sometimes it may happen that the debrises from the land slides may form an obstruction in the river valley like a dam, and thus a reservoir may be formed on the upstream side. Due to heavy rainfall, when the water pressure reaches a maximum value, then suddenly that obstruction may be removed and a high column of water may rush downstream destroying roads and railway bridges on its way and wipe out towns, villages, etc.

Note The river obstruction was the main cause of the destructive flood on 4th Oct, 1968 in Jalpaiguri district in North Bengal. In that year, a vast land slide obstructed the flow of Tista river just on the upstream side of Tista Bazar bridge (near Kalimpong). Due to the nonstop rainfall for about a week, that obstruction was suddenly removed and a water column of about 15 m rushed downstream, destroying Tista Bazar bridge, road and railway bridges near Jalpaiguri town and smashed Jalpaiguri town on the night of 4th Oct, 1968.

(5) Contraction of River Section While constructing road or railway bridges across a river, the approach works are done on both banks which reduce the cross-section of the river. Again, the waterways provided by constructing piers may not be sufficient for the outlet of the high flood discharge. In that case, the water rises on the upstream side due to insufficient passage and thus the upstream area may get submerged.

(6) Inadequate Cross Drainage Works In cross drainage works like aqueduct the river passes below the canal. Here, the structure which is constructed for the smooth running of the river water may be inadequate for the high flood discharge. Thus the water level may rise on the upstream side and may submerge the surrounding area.

(7) Negligence in Discharge Observation For the construction of bridges, cross drainage works, etc the hydrological survey is very essential. It includes the observation of discharge of a river at different sites. The discharge observation should be done sincerely and regularly at the specified time. The discharge data and silt analysis data are sent to the design office for the necessary design work of the structures. If the discharge records are made arbitrarily and false statements are sent to the office then this might lead to a serious consequence since the design is based on false statements. So, the observers should be loyal to their duties and there should be no negligence in discharge observation.

21.3 EFFECTS OF FLOOD

Ill effects The following are the ill effects of flood:

(a) Damage of Property When the villages or towns are submerged under considerable depth of water (1.5 m to 2 m) with high velocity of flow, then many houses may collapse, furniture and other valuable things may get damaged. If the flood water remains stagnant for several days, it accelerates the damage of buildings and other structures.

(b) Loss of Life If the flood water suddenly submerges the inhabitant areas under high depth and with high velocity the loss of life (both human and cattle) is more. The loss of life is maximum, if the flood water suddenly enters the inhabited areas at night.

(c) Water Logging The flood water may cause waterlogging in agricultural land making the soil alkaline in nature and reducing the yield of crop. Again, if the water remains stagnant for months, the cultivation of the land gets totally hampered.

(d) Loss of Crops If the flood water enters are agricultural land where the crops are nearly matured, they get totally spoiled. This loss of crops has financial implications for the cultivators.

(e) Disruption of Communication Due to the flood the culverts or bridges on the road and railways may be damaged. In some places the roads or railways may be washed out and communication may be disrupted. This may pose problems to the people.

(f) Rise of Price of Food Grains When the road and railway communication is disrupted due to damage by flood, the movement of the food grains and essential commodities is hampered. This leads to the rise of price of food grains and other essential materials.

(g) Loos of Work During flood all types of works such as building works, road works, agricultural works, etc remain suspended. So, the labourers who depend entirely on such works become unemployed during the period of flood and their life becomes miserable as they live a hand to mouth.

(h) Epidemic During flood, the water gets contaminated and the whole environment becomes polluted. Due to the pollution of water, the fishes carry germs of some diseases like cholera, dysentry, etc there is every chance of outbreak of epidemic of these diseases.

Good Effect The only good effect is that the agricultural land becomes enriched with silt which has got a good manure value and hence the yield of the crop becomes high.

21.4 BENEFITS OF FLOOD CONTROL (ECONOMICS OF FLOOD CONTROL)

The economics of flood control involves the study of benefits against the cost incurred in flood control works. The benefits of flood control are expressed in two ways,

1. Intangible Losses and Benefits The intangible losses are those which cannot be estimated in money values. The following are the intangible losses,

(a) Loss of human life.
(b) Loss of health due to diseases caused by flood.
(c) Loss caused by social distress.
(d) Loss due to hindrance in development works of towns or cities.

All the above losses will be converted to benefits, if the flood control works are done successfully.

2. Tangible Losses and Benefits The tangible losses are those which can be estimated in terms of some money value. The following are tangible losses,

(a) Damage of personal properties like building, furniture, etc.
(b) Loss of crops.
(c) Loss due to disruption of trade, business, etc.
(d) Loss due to disruption of road and railway communications.
(e) Additional expenditure for the safety against flood.
(f) Additional expenditure for medical care.

All the above losses will be converted to benefits, if the flood control works are done satisfactorily.

21.5 FLOOD FORECASTING

Network of Forecasting System The following network should be established (Fig. 21.1).

(1) Installation of Raingauge Stations Sufficient number of raingauge stations should be established at suitable places within the catchment area so as to record the rainfall covering the whole area.

(2) Installation of Monitoring Station A monitoring station should be established on the river valley or at a suitable place where the river enters the plane terrain. This monitoring station will collect the data from all the raingauge stations.

(3) Installation of Gauge Stations Gauge and discharge observation stations should be established at suitable places on the course of the river. The stations should be adjacent to the villages or towns.

station should alert the people through appropriate signal by siren, and inform the district authority, immediately.

(d) When the danger is over, the operator should inform the people by clear signal.

(e) According to the information received from the gauge stations the district authority should arrange everything for evacuation and other allied works.

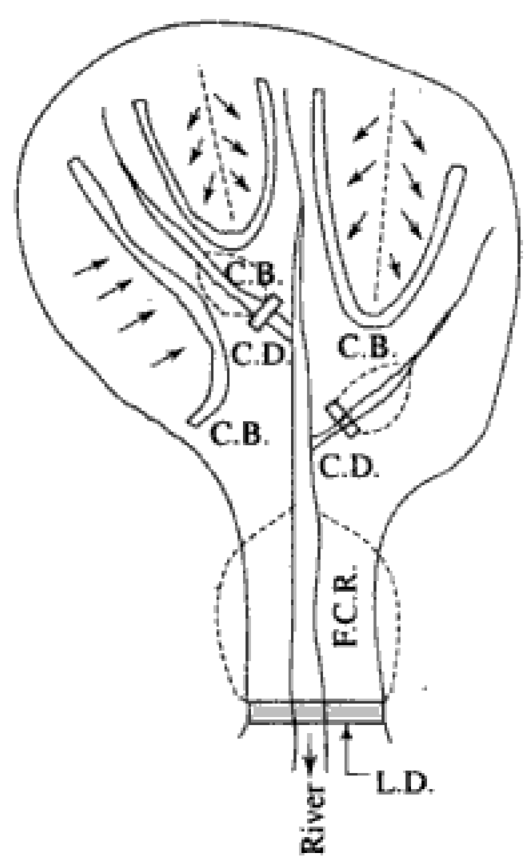
21.6 METHODS OF FLOOD CONTROL OR FLOOD MITIGATION WORKS

The following are the general methods of flood control works:

(1) Construction of Check Dam The check dams are constructed across the tributaries of river at a suitable places near the confluence point. These are low dams like weirs where the surplus water flows over the crest, some openings are provided slightly above the base of the dam through which the flood water flows out completely. But, the number of openings are such that the water takes much time to discharge off completely. The sediments are arrested just at the base of the dam. The height and section of the dam depends on the site condition. The dam may be constructed with stone masonary or concrete. If required such types of low dams may be constructed at different points of the tributaries (Fig. 21.2).

(2) Construction of Contour Bunds or Terrace Bunds In hilly catchment area, the contour bunds are constructed in rows at different elevations transverse to the slope. These are low height embankments constructed with earth work (with stone pitching) or stone masonry. The sediments and surface run-off are arrested by these bunds (Fig. 21.2).

(3) Construction of Flood Control Reservoirs Flood control reservoirs are formed by constructing low dams at suitable places on the course of the river. Generally, the reservoirs are formed on the upstream of the area to be protected or on the head reach of the river. The function of these reservoirs is to store a portion of the flood water temporarily to minimise the peak flood



C.D. = Check dam
C.B. = Contour bund
L.D. = Low dam
F.C.R. = Flood control reservoir

Fig. 21.2 Flood control works

(8) Construction of Diversion Channel From the upstream side of the flood affected area a diversion channel is excavated to connect the river with a large lake or bil (marshy land). Again, the lake is connected to other rivers or any water course by a link channel. Thus the flood water is diverted to the lake to reduce the water pressure at the flood affected area. The water of the lake again flows to the other river which reduces the pressure in the lake (Fig. 21.3).

(9) Construction of Cut-Off In case of sharp bends in the course of a river, the velocity of flow and the rate of discharge is reduced. During heavy rainfall when large flood discharge approaches the sharp bend of the river, it overflows it banks and submerges the surrounding area. So, cut-off or chord channel may be constructed so that the water can flow with high velocity along a straight path (Fig. 21.4).

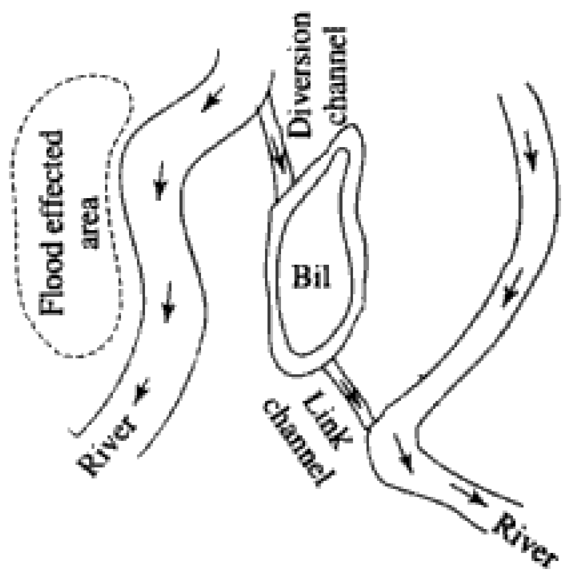


Fig. 21.3 Diversion and link channel

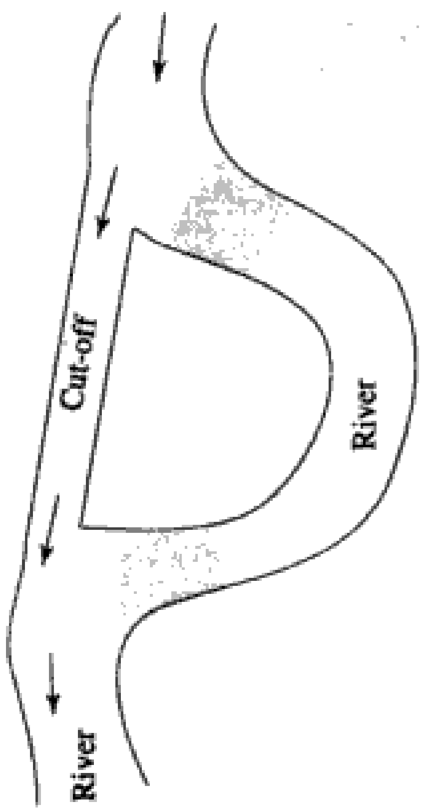


Fig. 21.4 Cut-off

21.7 EVACUATION AND FLOOD PROOFING

Emergency evacuation from the flood threatened area is one of the most effective way of reducing the flood damage. Flood protection works for some places are not economical. So, it is better to implement emergency evacuation. The following are the specific cases for evacuation,

1. The residents and livestocks in slum areas by the river bank or in the river bottom lands should be promptly evacuated after receiving the flood warning thereby preventing loss of life.
2. The equipments for the bridge construction should be promptly removed from the river side lands.

3. River side godowns should be emptied immediately to prevent loss. Isolated valuable property may be flood-proofed by providing following steps:
   (i) Ring levee or flood wall may be constructed around the industrial plant, storage yard, office building, etc to protect them from flood.
   (ii) Coffer dam may be constructed around the isolated valuable property.

21.8 LAND MANAGEMENT

It is established that the vegetal cover on the catchment area reduces the surface run off by increasing the rate of infiltration and by the process of transpiration. So, deforestation in the catchment area should be strictly prohibited and afforestation should be encouraged.

But it is seen that the vegetal cover is effective for minor storm. It is not effective during the flood producing storm.

REVIEW QUESTIONS

1. Fill up the blanks with appropriate word/words.
   (i) After flood there is a chance of outbreak of epidemic of disease like ___________.
   (ii) The only good effect of flood is that the agricultural land is enriched with ___________.
   (iii) The loss of life during flood is considered as ___________ loss.
   (iv) The loss of crops during flood is considered as ___________ loss.
   (v) Emergency ___________ from the flood threatened area is the most effective way of reduction of flood damage.
2. Distinguish between tangible loss and intangible loss.
3. How can the flood be controlled?
4. Enumerate the systems of flood forecasting.
5. Enumerate the ill-effects of flood.

ANSWERS

1. (i) Cholera (ii) Silt
   (iii) Intangible (iv) Tangible
   (v) Evacuation

APPENDIX A: OBJECTIVE-TYPE QUESTIONS WITH ANSWERS

1. The water goes to atmosphere by evaporation and transpiration and comes back again in the form of precipitation, this process is known as,
   (a) Water cycle (b) Hydrologic cycle
   (c) Atmospheric cycle.
2. The water, after precipitation, flows over the ground surface to join ponds, lakes or rivers is known as,
   (a) Run-off (b) Surface flow
   (c) Stream flow.
3. Simon's rain gauge is
   (a) Automatic type (b) Non-recording type
   (c) Recording type.
4. The average rainfall intensity above which the volume of rainfall is equal to volume of run-off is known as,
   (a) Intensity index (b) ϕ-index
   (c) Rainfall index.
5. The average rate of infiltration during the time when rainfall intensity exceeds infiltration capacity is known as,
   (a) W-index (b) Infiltration index
   (c) Percolation index.
6. Dicken's formula for estimating flood discharge is given by the expression,
   (a) $Q = C \cdot A^{2/3}$ (b) $Q = C \cdot A^{3/4}$
   (c) $Q = C \cdot A^{2/5}$.
7. Ryve's formula for estimating flood discharge is given by the expression,
   (a) $Q = C \cdot A^{2/3}$ (b) $Q = C \cdot A^{3/2}$
   (c) $Q = C \cdot A^{5/2}$.
8. A graph in which discharge is plotted on Y-axis and corresponding time on X-axis is known as,
   (a) Hydrograph (b) Pentagraph
   (c) Seismograph.

9. The hydrograph resulting from 1 cm of effective rainfall for unit duration is known as,
   (a) S-hydrograph (b) Unit hydrograph
   (c) Differential hydrograph.
10. The entire area enclosed between an imaginary boundary line to include an irrigation project is known as,
    (a) Irrigation project area (b) Gross command area
    (c) Culturable command area.
11. The total time that elapses between the sowing of the crop and its harvesting is known as,
    (a) Crop period (b) Base period
    (c) Total period.
12. The total time between the first watering and last watering to the crop is known as,
    (a) Harvesting period (b) Crop period
    (c) Base period.
13. The first watering before sowing any crop is known as,
    (a) Paleo (b) Paleva
    (c) Kor watering.
14. The first watering after the crop has grown to few centimetre is known as,
    (a) Primary watering (b) Kor watering
    (c) Initial watering.
15. The total depth of water required by a crop for its maturity is known as
    (a) Delta (b) Duty
    (c) Base.
16. The relation between Base, Delta and Duty is given by,
    (a) $\Delta = \dfrac{8.64 \times D}{B}$ (b) $\Delta = \dfrac{8.64 \times B}{D}$
    (c) $D = \dfrac{8.64 \times B}{\Delta}$.
17. Basin method is suitable for the irrigation of
    (a) Orchard (b) Plain land
    (c) Hilly area.
18. The method of irrigation for the crops like potato, sugarcane, ground nut, etc. is,
    (a) Contour method (b) Furrow method
    (c) Border strip method.
19. In hilly area the suitable method of irrigation is
    (a) Flooding method. (b) Basin method.
    (c) Contour forming method.
20. When an irrigation canal is taken over the natural drainage, the cross-drainage work is known as,
    (a) Aqueduct (b) Super passage
    (c) Level crossing.

21. When a natural drainage is taken over the irrigation canal, the cross drainage work is known as,
    (a) Level crossing (b) Inlet and outlet system
    (c) Super passage.
22. When the natural slope of the ground is greater than the bed slope of channel, the structure to be provided is,
    (a) Fall (b) Weir
    (c) Barrage.
23. The strip of land left between the top edge of cutting and toe of banking is known as,
    (a) Free board (b) Berm
    (c) Dowla.
24. The vertical distance between the F.S.L. and the top of embankment is known as,
    (a) Free board (b) Counter berm
    (c) Lead.
25. For the necessary cover over the hydraulic gradient line sometimes it is required to provide,
    (a) Barrowpit (b) Counter berm
    (c) Service road.
26. The ratio of wetted cross-sectional area and wetted perimeter is known as,
    (a) Hydraulic mean radius (b) Hydraulic factor
    (c) Regime factor.
27. When the character of bed and bank materials of a channel is same as that of the transported material, the channel is called,
    (a) Idial channel (b) Regime channel
    (c) Normal channel.
28. When the velocity of flowing water causes no silting or scouring in a channel, it is known as,
    (a) Critical velocity (b) Average velocity
    (c) Optimum velocity.
29. A channel is paid to be in scouring when,
    (a) $C.V.R. < 1$ (b) $C.V.R. = 1$
    (c) $C.V.R. > 1$.
30. A channel is said to be in silting when,
    (a) $C.V.R. < 1$ (b) $C.V.R. = 1$
    (c) $C.V.R. > 1$.
31. Mean velocity of flow is given by Chezy's formula,
    (a) $V = C.R. \sqrt{S}$ (b) $V = S. \sqrt{CR}$
    (c) $V = C. \sqrt{RS}$.
32. Mean velocity of flow given by Manning's formula is,
    (a) $V = \frac{1}{N} \cdot R^{2/3} \cdot S^{1/2}$

(b) $V = \frac{1}{N} \cdot R^{1/2} \cdot S^{2/3}$

(c) $V = \frac{1}{N} \cdot R^{1/2} \cdot S^{1/2}$

33. Critical velocity of flow given by Kennedy's silt theory is,
    (a) $V_0 = 0.546. \text{ m. } D^{0.64}$
    (b) $V_0 = 0.645. \text{ m. } D^{0.46}$
    (c) $V_0 = 0.456. \text{ m. } D^{0.06}$
34. When it is required to raise the water level on the upstream side, the structure constructed is,
    (a) Dam (b) Barrage
    (c) Weir.
35. When it is required to form a storage reservoir, the structure constructed is,
    (a) Weir (b) Barrage
    (c) Dam.
36. When it is required to control the water level on the upstream side, the structure constructed is,
    (a) Barrage (b) Weir
    (c) Dam.
37. The earthen embankment constructed along both the banks of a river to control flood is known as,
    (a) Dyke (b) Levee
    (c) Guide bank.
38. The soil moisture useful for the growth of plants is,
    (a) Capillary water (b) Gravity water
    (c) Hygroscopic water.
39. The field capacity of soil depends upon,
    (a) Capillary tension in soil (b) Porosity of soil
    (c) Both the above.
40. The depth of root zone of rice is,
    (a) 60 cm (b) 70 cm
    (c) 80 cm (d) 90 cm.
41. For crops is undulating sandy field, the best method of irrigation is
    (a) Sprinkler method (b) Furrow method
    (c) Check dam method.
42. Irrigation canals are generally aligned along,
    (a) Ridge line (b) Contour line
    (c) Valley line.
43. The canal which carry water in rainy season only is known as
    (a) Perennial canal (b) Inundation canal
    (c) Permanent canal.
44. If 'V' and 'R' are regime mean velocity and hydraulic mean depth respectively, then Lacey's silt factor is given by,

(e) Forecasting flood
(f) All the above.

57. The surface run-off is the quantity of water,
(a) Intercepted by buildings and vegetative cover
(b) Required to fill the depressions
(c) That reaches the stream or river.

58. Hydrograph is a graphical representation of,
(a) Surface run-off (b) Ground water flow
(c) Rainfall (d) Discharge flowing in the river.

59. Infiltration capacity of soil depends upon,
(a) Number of voids in the soil
(b) Shape and size of soil particles
(c) Compaction of soil particles
(d) All the above.

60. A soil strata may consist of,
(a) Soil zone (b) Capillary zone
(c) Ground water zone (d) All the above.

61. The main factor which affects the infiltration capacity is
(a) Soil moisture (b) Surface detention
(c) Thickness of saturated layer (d) All the above.

62. The infiltration capacity during rain is reduced by,
(a) Surface detention (b) Soil moisture
(c) Both (a) & (b).

63. Rain simulators are used for the determination of,
(a) Evaporation (b) Precipitation
(c) Infiltration capacity.

64. Precipitation includes,
(a) Rain (b) Snow
(c) Hail (d) All the above

65. Pressure exerted by fully saturated air is known as,
(a) Partial pressure (b) Vapour pressure
(c) Saturation pressure.

66. Humidity refers to,
(a) Temperature of the air (b) Pressure of the air
(c) Moisture content of the air.

67. Relative humidity is the ratio of actual vapour pressure to saturation vapour pressure at,
(a) The same temperature
(b) The same pressure
(c) The same volume.

68. Absolute humidity in air,
(a) Decreases at higher altitude (b) Increases at higher altitude
(c) Remains constant.

69. Precipitation caused due to upward movement of warmer air is called,
    (a) Cyclonic precipitation (b) Convective precipitation
    (c) Orographic precipitation.
70. Precipitation caused due to striking of air masses with a topographical features is called,
    (a) Orographic precipitation (b) Convective precipitation
    (c) Cyclonic precipitation.
71. The standard height of a standard rain gauge is,
    (a) 10 cm (b) 20 cm (c) 30 cm (d) 50 cm.
72. In India rainfall is generally recorded at,
    (a) 8 A.M. (b) 12 Noon (c) 4 P.M. (d) 8 P.M.
73. A recording type rain gauge.
    (a) Produces a mass curve of rainfall
    (b) Records the cumulative rainfall
    (c) all the above.
74. Simon's rain gauge is,
    (a) Tipping bucket rain gauge (b) Weighing type rain gauge
    (c) Float recording type rain gauge (d) Non-recording rain gauge.
75. The rainfall cycle period in India is taken as,
    (a) 15 years (b) 25 years (c) 30 years (d) 35 years.
76. Isohyets are the imaginary lines joining the points of equal,
    (a) Pressure (b) Height (c) Rainfall (d) Humidity.
77. For the determination of average annual precipitation, the best method is,
    (a) Arithmetical method (b) Thiessen's polygon method.
    (c) Isohyetal method.
78. The rainfall at any place is described by its,
    (a) Intensity (b) Duration (c) Frequency (d) All the above.
79. Sharp crested weirs are generally used for,
    (a) Large flows (b) Small flows
    (c) River carrying floating debris.
80. The best method for measuring the velocity of flow is,
    (a) Pitot tube method (b) Current meter method
    (c) Surface float method (d) Velocity rod method.
81. The run-off is affected by,
    (a) Size of basin (b) Shape of basin
    (c) Elevation of watershed (d) All the above.
82. Run-off is measured in,
    (a) m^3/hr (b) m^3/min
    (c) m^3/sec.
83. A unit hydrograph is a hydrograph of a rainstorm of a specified duration resulting from a rainfall of,
    (a) 10 mm (b) 20 mm
    (c) 30 mm.

84. The quantity of water retained by the sub soil against the pull of gravity is known as,
    (a) Yield (b) Porosity
    (c) Specific yield (d) Specific retention.
85. If the gain size of soil increases,
    (a) Surface area decreases (b) Specific retention decreases
    (c) Specific yield increases (d) All the above.
86. Dupuits formula is based on,
    (a) One observation well (b) Two observation well
    (c) Three observation well (d) No observation well.
87. The radius of circle of influence is,
    (a) The radius of main well
    (b) The distance from the well to the point of zero draw down
    (c) The distance between the centre of well and the point of zero draw down.
88. Shrouding is provided in,
    (a) Cavity type tube well (b) Slotted type tube well
    (c) Strainer type tube well.
89. The efficiency of a pump may be taken as,
    (a) 0.55 (b) 0.60 (c) 0.65 (d) 0.70.
90. Consumptive use of water for crops includes,
    (a) Interception (b) Transpiration
    (c) Evaporation (d) All the above.
91. Evaporation losses depend on,
    (a) Area of water surface (b) Humidity and wind velocity
    (c) Atmospheric temperature (d) All the above.
92. Dicken's formula for high flood discharge is useful for the catchment in,
    (a) Southern India (b) Northern India
    (c) Eastern India (d) Western India.
93. For the estimate of high flood discharge in fan shaped catchment, the formula used is,
    (a) Inglis formula (b) Ryve's formula
    (c) Dicken's formula.
94. For predicting flood of a given frequency the best method is,
    (a) Unit hydrograph method (b) Gumbel's method
    (c) California method.
95. The earthen embankments running parallel to the river are known as,
    (a) Dykes (b) Flood walls
    (c) Levees (d) Guide wall.
96. The zone below the water table is known as,
    (a) Zone of saturation (b) Zone of aeration
    (c) Zone of hydrostatic pressure.
97. The zone above the water table is known as,
    (a) Zone of vacuum (b) Zone of aeration
    (c) Zone of capillary fringe.

98. The critical depth in a channel can be produced by,
    (a) Decreasing the width of the channel
    (b) Increasing the width of the channel
    (c) Increasing the side slope.
99. If R is the hydraulic mean radius and D is the depth of water, the section of the canal, will be most economical when,
    (a) $R = D/2$ (b) $D = R/2$
    (c) $D = R$.
100. Sugarcane is designated as,
    (a) Eight month crop (b) Kharif crop
    (c) Rabi crop (d) Perennial crop.

ANSWERS

1. (b)	2. (a)	3. (b)	4. (b)	5. (a)
6. (b)	7. (a)	8. (a)	9. (b)	10. (b)
11. (a)	12. (c)	13. (a) & (b)	14. (b)	15. (a)
16. (b)	17. (a)	18. (b)	19. (c)	20. (a)
21. (c)	22. (a)	23. (b)	24. (a)	25. (b)
26. (a)	27. (b)	28. (a)	29. (c)	30. (a)
31. (c)	32. (a)	33. (a)	34. (c)	35. (c)
36. (a)	37 (a) & (b)	38. (a).	39. (c)	40. (d)
41. (a)	42. (c)	43. (b)	44. (b).	45. (d)
46. (b)	47. (a)	48. (b)	49. (b)	50. (b)
51. (d)	52. (d)	53. (c)	54. (a)	55. (d)
56. (d)	57. (d)	58. (d)	59. (d)	60. (d)
61. (d)	62. (c)	63. (c)	64. (d)	65. (b) & (c)
66. (c)	67. (a)	68. (a)	69. (b)	70. (a)
71. (c)	72. (a)	73. (c)	74. (d)	75. (d)
76. (c)	77. (c)	78. (d)	79. (a)	80. (b)
81. (d)	82. (c)	83. (a)	84. (d)	85. (d)
86. (d)	87. (c)	88. (b)	89. (c)	90. (d)
91. (d)	92. (b)	93. (a)	94. (a)	95. (a) & (c)
96. (a)	97. (b)	98. (a)	99. (a)	100. (d)

APPENDIX B: MODEL QUESTIONS (WITH HINTS)

1. What is irrigation? What are the necessity of irrigation? (See Sec. 1.1 and 1.2)
2. Enumerate the ill-effects and benefits of irrigation. (See Sec. 1.3 and 1.4)
3. What are the advantages and disadvantages of lift irrigation? (See Sec. 1.5)
4. What is the difference between inundation system and perennial system of irrigation? (See Sec. 1.5)
5. Enumerate, with sketch, the direct irrigation system and storage irrigation system (See Sec. 1.5)
6. State clearly, with sketch, the difference between weir, barrage and dam. (See Sec. 1.6)
7. Write short notes on the following,
   (a) Furrow method
   (b) Contour farming method
   (c) Flooding method
   (d) Sub-surface method
   (e) Sprinkler method. (See Sec. 1.7)
8. Write short notes on the following,
   (a) Bhakra-Nangal project
   (b) Damodar valley project
   (c) Ukai project
   (d) Mahanadi delta project. (See Sec. 1.10)
9. State the factors that affect the water requirement of crops. (See Sec. 2.2)
10. Define the following,
    (a) Cross command area
    (b) Culturable command area
    (c) Intensity of irrigation
    (d) Crop ratio
    (e) Crop season.
    (f) Crop period and base period.
    (g) Capacity factor
    (h) Cumec-day (See Sec. 2.3)
11. Define base, delta and duty, Establish a relation between them. (See Sec 2.4 and 2.7).
12. What are the factors that affect duty? Describe the methods of improving duty. (See Sec. 2.5 and 2.6)

13. What are the different types of agricultural soil? (See Sec 2.9)
14. Write notes on gravitational water, capillary water and hygrosopic water. (See Sec 2.10)
15. Describe the methods of determination of consumptive use of water. (See. Sec. 2.12)
16. Write short notes on frequency of irrigation and irrigation efficiencies. (See Sec. 2.13 and 2.15)
17. State the standard of irrigation water. (See Sec. 2.14)
18. What is meant by hydrology? State the importance of hydrology. (See Sec. 3.1 and 3.2)
19. Explain the hydrologic cycle with sketch. (See Sec. 3.4)
20. Explain the following,
    (a) Hydrograph (b) Hyetograph
    (c) Unit hydrography (See Sec. 3.5, 3.6, and 3.7)
21. State the assumption in unit hydrograph.
    What are the limitations of unit hydrograph?
    State the advantages of unit hydrograph. (See Sec. 3.7)
22. What are base flow and negative base flow? (See Sec. 3.8)
23. Write short notes on
    (a) Cyclonic precipitation. (b) Convective precipitation.
    (c) Orographic precipitation (See Sec. 3.9)
24. What are the different types of rain gauges. Describe, with sketch, the non recording type raingauge. (See Sec. 3.9)
25. Describe Simon's raingauge with sketch. (See Sec. 3.9)
26. What are the points to be considered while selecting a raingauge station? (See Sec. 3.9)
27. Describe the methods of computing average depth of precipitation (See Sec. 3.10)
28. Explain the different causes of water losses. (See Sec. 3.11)
29. What are the factors that affect the evaporation from free water surface. (See Sec. 3.11)
30. What are infiltration and infiltration capacity? State the factors that affect the infiltration capacity. (See Sec. 3.11)
31. What are infiltration indices? (See Sec 3.12)
32. What is run-off? What are the factors that affect the run-off? (See Sec. 3.3 and 3.13)
33. State the different methods of estimation of run-off. (See Sec. 3.14)
34. State the different methods of estimation of peak flow (flood discharge) (See Sec. 3.16)
35. What is meant by flood routing? Explain the methods of flood routing (See Sec. 3.17)
36. Describe, with sketch, the following methods of lift irrigation,
    (a) Doon (b) Persian wheel
    (c) Denkli (d) Mote. (See Chapter 4)

37. Define the following,
    (a) Porosity
    (b) Aquifers
    (c) Specific yield
    (d) Specific capacity
    (e) Efficiency of well
    (f) Cone of depression
    (g) Draw down
    (h) Circle of influence
    (See Sec. 4.4)
38. What is meant by shallow well and deep well? (See Sec. 4.4.)
39. Write short notes on,
    (a) Strainer type tube-well.
    (b) Cavity type tube-well.
    (c) Slotted type tube-well. (See Sec. 4.5)
40. Describe the procedure of construction of lined open well by masonry work. (See Sec. 4.6)
41. Describe the procedure of determining the yield of open well. (See Sec. 4.7)
42. Describe the method of sinking shallow tube-well by waterjet method. (See Sec. 4.9)
43. Describe the method of sinking deep tube-well by percussion method. (See Sec. 4.9)
44. Establish an expression for the yield of tube-well in unconfined aquifer. (See Sec. 4.10)
45. Establish an expression for the yield of tube-well in confined aquifer. (See Sec. 4.11)
46. What is meant by shrouding? How is shrouding of well done? (See Sec. 4.14)
47. What is meant by development of well? How is development of well done? (See Sec. 4.15)
48. What are the points to be remembered for the maintenance of tube-well? (See Sec. 4.16)
49. What are the causes of failure of a tube-well? State the precautions to be taken to prevent the failure. (See Sec. 4.17)
50. Describe a centrifugal pump with a sketch. (See Sec. 4.19)
51. Draw a section of deep tube well. (See Sec. 4.25)
52. Define the following
    (a) Irrigation canal
    (b) Navigation canal
    (c) Power canal
    (d) Feeder canal
    (e) Inundation canal
    (f) Perennial canal
    (g) Water shed canal
    (h) Contour canal
    (i) Side slope canal (See Sec. 5.2)
53. What are the points that should be considered while selecting a barrage and a dam site? (See Sec. 5.3)
54. What are the points that should be considered while selecting the alignment of a perennial canal? (See Sec. 5.4)
55. What are the points that should be considered while selecting the alignment of an inundation canal? (See Sec. 5.5)

56. Explain the advantages and disadvantages of inundation irrigation system. (See Sec.5.7)
57. What are the fundamental differences between the inundation irrigation system and perennial irrigation system? (See Sec. 5.8)
58. Describe, with sketch, the system of bandhara irrigation. State the advantages and disadvantages of this system. (See Sec. 5.9 and 5.10)
59. Draw a canal section and show in it, the positions of the following components.
    (a) Berm
    (b) Counter berm
    (c) Dowla
    (d) Spoil bank
    (e) Borrowpit (See Sec. 6.1)
60. Define the following
    (a) Hydraulic gradient
    (b) Berm
    (c) Counter berm
    (d) Dowla
    (e) Spoil bank
    (f) Borrowpit
    (g) Free board (See. Sec. 6.3-6.11)
61. What is balancing depth of cutting and banking? Explain, how can it be determined. (See sec. 6.13)
62. Draw a canal section in full cutting. (See Sec. 6.14)
63. Draw a canal section in full banking (See Sec. 6.15)
64. Draw a canal section in partial cutting and partial banking. (See Sec. 6.16)
65. What are the objects of canal lining? (See Sec. 7.1)
66. What are the different types of canal lining? Explain them with sketch where necessary. (See Sec. 7.2-7.12)
67. What are the advantages and disadvantages of canal lining? (See Sec. 7.13)
68. What are the points to be considered while selecting the type of lining? (See Sec. 7.14)
69. What arguments can you give justification of canal lining? (See Sec. 7.15)
70. What is meant by storage head works and diversion head works? State the objects of diversion head works. (See Sec. 10.1-10.2)
71. State the points to be remembered while selecting the site for storage head works and diversion head works. (See Sec. 10.3-10.4)
72. Give a neat sketch of diversion head works showing the different component parts. (See Sec. 10.5)
73. What are the component parts of a diversion head work? Mention the function of each component. (See Sec. 10.5-10.13)
74. Describe the following with a neat sketch,
    (a) Weir
    (b) Barrage
    (c) Divide wall
    (d) Fish ladder
    (e) Canal head regulator
    (f) Silt excluder
    (g) Marginal embankment
    (h) Guide bank (See Sec. 10.6-10.13)

75. What are stream lines, equipotential lines and exit gradient? (See Sec. 11.2-11.3)
76. What are the causes of failure of weir or barrage on permeable foundation? (See Sec. 11.4)
77. What are the precautions to be taken against failure? (See Sec. 11.5)
78. Explain Bligh's creep theory and state the limitations of this theory. (See Sec. 11.6)
79. What are the different types of weirs? Describe each type with a sketch. (See Sec. 11.9)
80. What are the component parts of the weir? State the functions of each component. (See Sec. 11.10)
81. What are the component parts of the barrage? State the functions of each component. (See Sec. 11.11)
82. Describe, with sketch the method of construction of well foundation. (See sec. 11.11)
83. Describe, with sketch, the method of construction of pneumatic caisson foundation. (See Sec. 11.11)
84. State the necessity of spill way and the location of spill way. (See Sec. 12.1-12.2)
85. Describe the following with sketch,
    (a) Drop spill way
    (b) Ogee spill way
    (c) Siphon spill way
    (d) Chute spill way
    (e) Shaft spill way
    (f) Side channel spill way.
    (See Sec. 12.5-12.10)
86. What is cross drainage works? Why are cross drainage works necessary? (See Sec. 13.1-13.2)
87. What are the different types of cross drainage works? What are the considerations of selecting the type of cross-drainage works? (See Sec. 13.3-13.4)
88. Describe the following with sketch,
    (a) Aqueduct
    (b) Siphon aqueduct
    (c) Super passage
    (d) Level crossing
    (See Sec. 13.5-13.9)
89. What is meant by canal fall? What is the necessity of canal fall? (See Sec. 14.1-14.2)
90. Describe the following with sketch,
    (a) Ogee falls
    (b) Rapid falls
    (c) Stepped falls
    (d) Sarda falls
    (e) Montague falls
    (f) Inglish fall
    (See Sec 14.4-14.9)
91. State the requirements of navigable water way. (See Sec. 15.2)
92. What are the different methods of river navigation? Describe each method with a sketch. (See Sec. 15.4)

116. (a) State the methods of irrigation water management.
(b) State the water laws applicable during the period of water scarcity. (See Sec. 17.15-17.16)
117. (a) What are the causes of waterlogging?
(b) State the effects of water logging. (See Sec. 18.2-18.3)
118. Enumerate the process of controlling water logging. (See Sec. 18.4)
119. Describe the methods of land reclamation. (See Sec. 18.5)
120. What is the necessity of watershed management? How is watershed management done? (See Sec. 18.6)
121. What is river gauging? What is the necessity of river gauging? (See Sec. 19.1-19.2)
122. Why and how are gauge posts fixed? (See Sec. 19.1-19.4)
123. What are the points to be considered while selecting a discharge site? (See Sec. 19.5)
124. How can depth of water in a river be measured? (See Sec. 19.7)
125. How can the area of the water section of a river be determined? (See Sec. 19.8)
126. How can the velocity of flow of a river be measured? (See Sec. 19.9)
127. (a) What are the different stages of a river?
(b) What are the different types of river training works?
(c) Draw a sketch showing the position of each training work. (See Sec. 20.2-20.5)
128. Write short notes on the following,
(a) Spurs (b) Groynes
(c) Guide bank (d) Bank protection
(e) Marginal embankment. (See Sec. 20.7-20.9)
129. (a) What is flood?
(b) What are the causes of flood?
(c) What are the effects of flood? (See Sec. 21.1-21.3)
130. Describe the different forms of flood control works? (See Sec. 21.6)
131. State the systems to be adopted for flood forecasting. (See Sec. 21.5)
132. Enumerate the benefits of flood control works. (See Sec. 21.4)
133. Determine the head discharge of a canal having the following data,
1. G.C.A. = 7500 ha.
2. C.C.A. = 80%
3. Intensity of irrigation for kharif = 40%
4. Intensity of irrigation for rabi = 60%
5. Duty for kharif = 1400 ha/cumec.
6. Duty for rabi = 2100 ha/cumec.
7. Consider losses and overlap allowance = 30%

(*Ans.* 22.3 cumec)
134. Find the reservoir capacity of irrigate a command area of 50,000 hectares having the following data,

Crop.	Base period in days	Duty (in ha/cumec)	Intensity of irrigation (in percentage)
Rice	140	900	15
Wheat	120	2000	20
Cotton	180	1600	10
Sugar cane	360	2500	20

Consider all losses (canal and reservoir) as 13%.

(*Ans.* 36770 ha-m)

135. A tube well of diameter 30 cm. penetrates fully an unconfined aquifer. Determine the discharge of the well having the following data
   (a) Draw down = 3 m.
   (b) Effective length of strainer = 10.5 m.
   (c) Radius of circle of influence = 300 m.
   (d) Coefficient of permeability = 0.05 m/sec.

(*Ans.* = 14.83 lits/sec)

136. A tube well penetrates fully a confined aquifer. Calculate the yield of the well with the following data,
   (a) Diameter of well = 30 cm.
   (b) Length of strainer = 25 m.
   (c) Draw down = 4 m.
   (d) Radius of circle of influence = 200 m.
   (e) Coefficient of permeability = 60 m/day.

(*Ans.* = 50.37 lits/sec)

137. Determine the diameter of a tube well from the following data,
   (a) Yield of well = 100 lits/sec.
   (b) Radius of circle of influence = 200 m.
   (c) Coefficient of permeability = 6.944×10^{4} m/sec.
   (d) Draw down = 6 m.
   (e) Thickness of confined aquifer = 30 m. (*Ans.* 16 cm)

138. Determine the radius of a tube well from the following data,
   (a) Discharge required = 0.04 m^3/sec.
   (b) Thickness of confined aquifer = 30 m.
   (c) Coefficient of permeability = 4.398×10^{-4} m/sec.
   (d) Draw down = 4 m.
   (e) Radius of circle of influence may be calculated by Sichardt formula, $R = 3000\, S\sqrt{K}$. (*Ans.* 6.5 cm)

139. Design a most economical trapezoidal section of a irrigation channel having the following data,
   (a) Required discharge = 15 cumec.
   (b) Permissible velocity = 1 m/sec.
   (c) Side slope = 2 : 1.
   (d) Value of $C = 60$.

(*Ans.* $B = 1.16$ m, $D = 2.46$ m, $S = 1$ in 4435)

140. Design a irrigation canal to carry the discharge 50 cumec with permissible velocity 1 m/sec. assuming the other data as, (i) $B/D = 6$, (ii) Side slope, = 2:1, (iii) N = 0.225.

(*Ans.* $B = 15$ m, $D = 2.5$ m, $S = 1$ in 4740)

141. Design a irrigation channel with the following data,
   (a) Full supply discharge = 14 cumec.
   (b) Bed slope = 0.0002
   (c) $N = 0.0225$
   (d) $m = 1$
   (e) Side slope = 1/2 : 1
   (f) Assume full supply depth = 1.8 m. (*Ans.* $B = 8.88$ m)

142. Design an irrigation channel with the following data,
   (a) Full supply discharge = 15 cumec.
   (b) Longitudinal slope = 1 in 5000
   (c) C.V.R. = 1
   (d) N = 0.0225
   (e) Full supply depth = 1.75 m
   (f) Side slope = 1/2 : 1

   Comment whether the designed channel will be non-scouring and non-silting or not. (*Ans.* $B = 10$ m, non-scouring and non-silting).

143. An irrigation canal is to be designed to carry 35 cumec with the following data,
   (a) Coefficient of roughness (N) = 0.025.
   (b) Longitudinal slope = 0.0002
   (c) Side slope = 1 : 1.
   (d) Full supply depth = 2 m.

   Verify the design by computing mean velocity by Chezy's formula.

(*Ans.* $B = 18.35$ m)

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