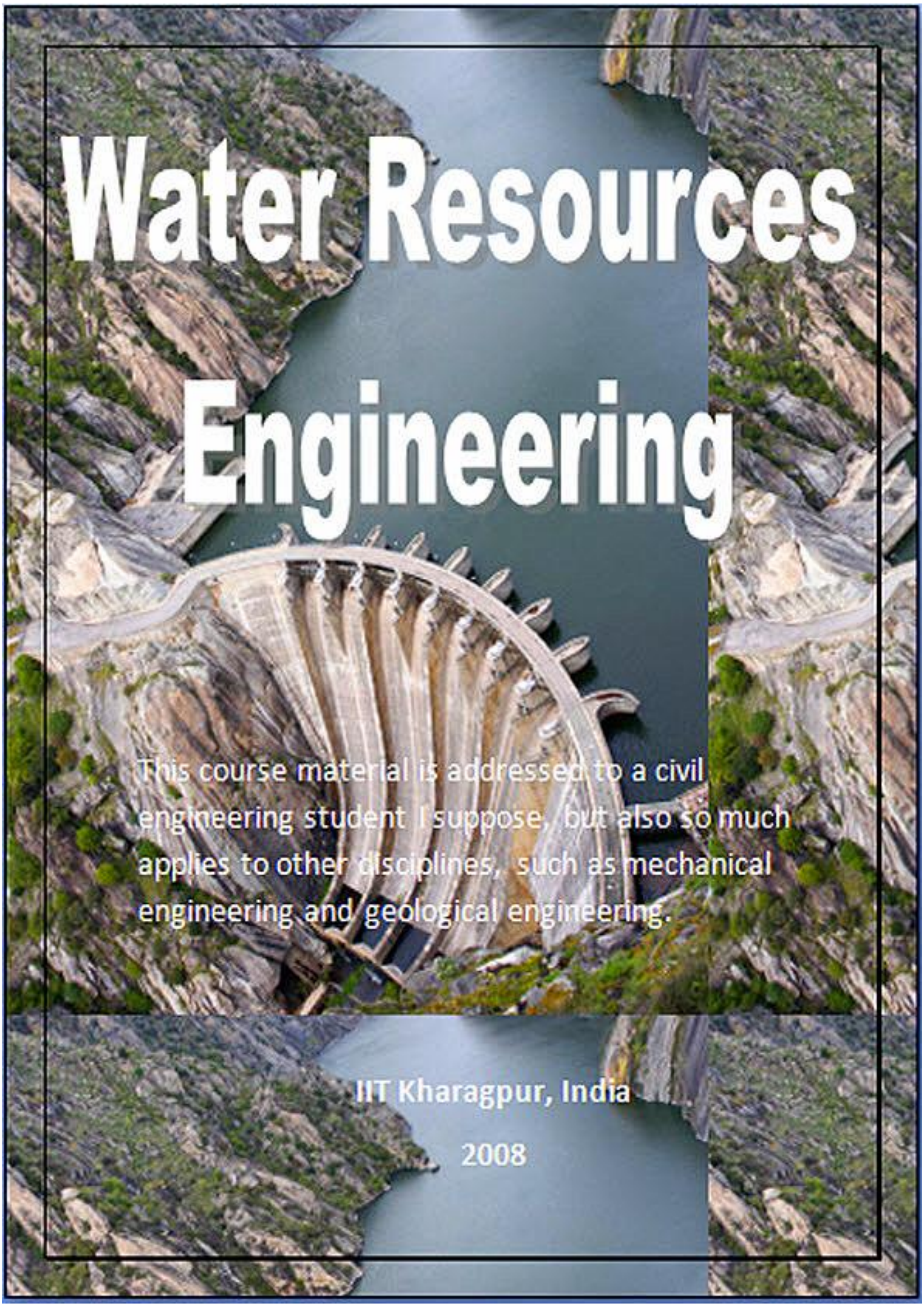

Water Resources Engineering

This course material is addressed to a civil engineering student I suppose, but also so much applies to other disciplines, such as mechanical engineering and geological engineering.

IIT Kharagpur, India

2008

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Module
1

Principles of Water Resources Engineering

Lesson
1
Surface and Ground Water Resources

Instructional Objectives

After completion of this lesson, the student shall know about

1. Hydrologic cycle and its components
2. Distribution of earth's water resources
3. Distribution of fresh water on earth
4. Rainfall distribution in India
5. Major river basins of India
6. Land and water resources of India; water development potential
7. Need for development of water resources

1.1.0 Introduction

Water in our planet is available in the atmosphere, the oceans, on land and within the soil and fractured rock of the earth's crust Water molecules from one location to another are driven by the solar energy. Moisture circulates from the earth into the atmosphere through evaporation and then back into the earth as precipitation. In going through this process, called the Hydrologic Cycle (Figure 1), water is conserved – that is, it is neither created nor destroyed.

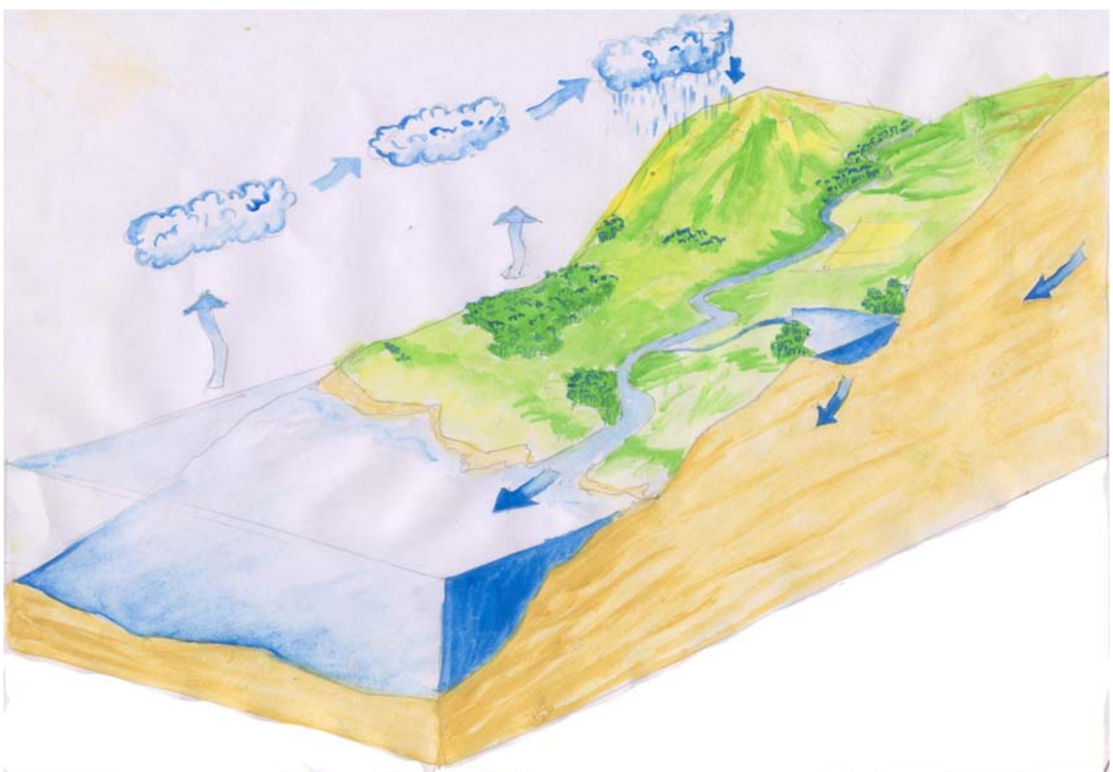


Figure 1. Hydrologic cycle

It would perhaps be interesting to note that the knowledge of the hydrologic cycle was known at least by about 1000 BC by the people of the Indian Subcontinent. This is reflected by the fact that one verse of Chhandogya Upanishad (the Philosophical reflections of the Vedas) points to the following:

“The rivers… all discharge their waters into the sea. They lead from sea to sea, the clouds raise them to the sky as vapour and release them in the form of rain…”

The earth’s total water content in the hydrologic cycle is not equally distributed (Figure 2).

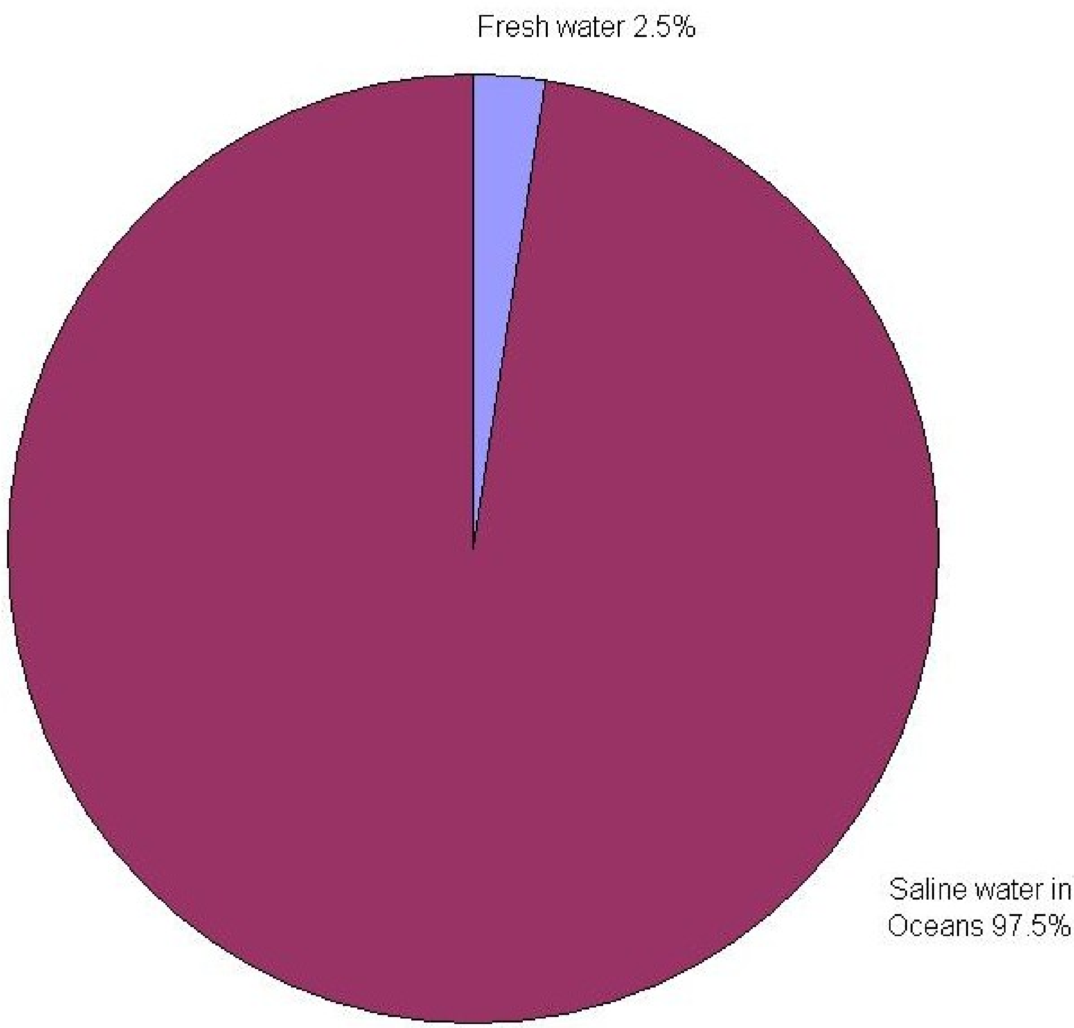


Figure 2. Total global water content

The oceans are the largest reservoirs of water, but since it is saline it is not readily usable for requirements of human survival. The freshwater content is just a fraction of the total water available (Figure 3).

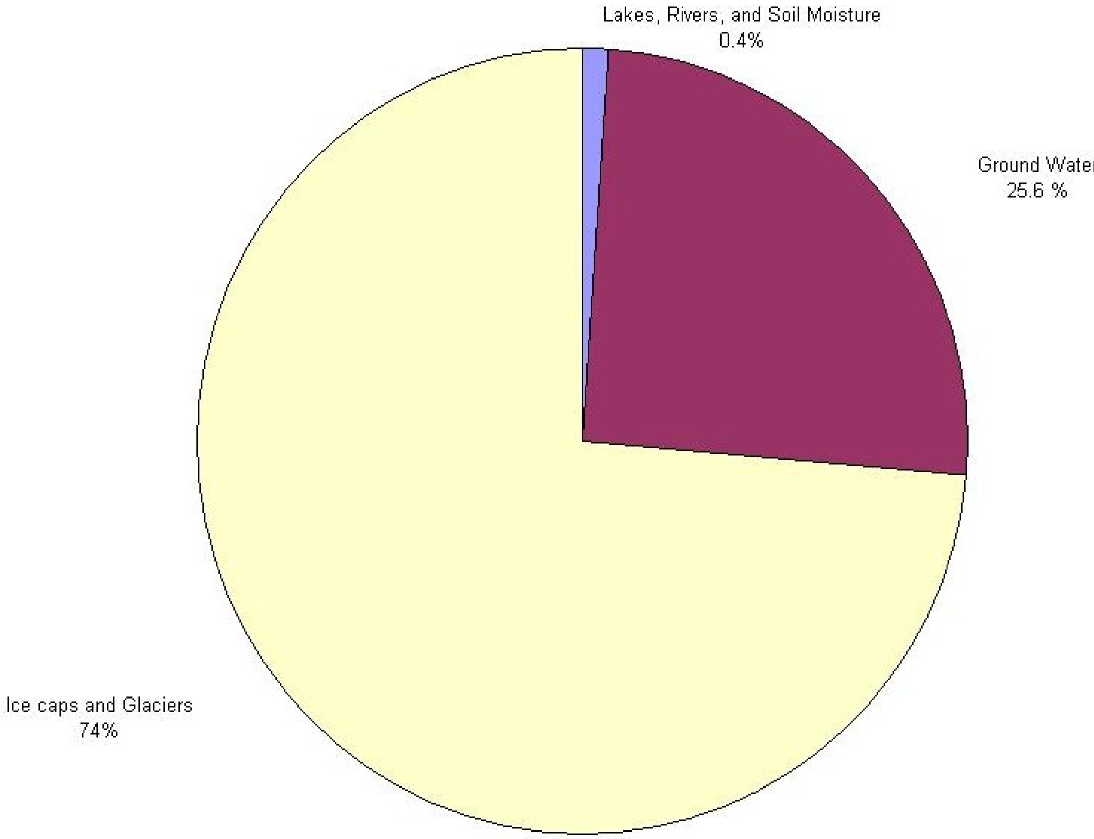


Figure 3. Global fresh water distribution

Again, the fresh water distribution is highly uneven, with most of the water locked in frozen polar ice caps.

The hydrologic cycle consists of four key components

1. Precipitation
2. Runoff
3. Storage
4. Evapotranspiration

These are described in the next sections.

1.1.1 Precipitation

Precipitation occurs when atmospheric moisture becomes too great to remain suspended in clouds. It denotes all forms of water that reach the earth from the atmosphere, the usual forms being rainfall, snowfall, hail, frost and dew. Once it reaches the earth's surface, precipitation can become surface water runoff, surface water storage, glacial ice, water for plants, groundwater, or may evaporate and return immediately to the atmosphere. Ocean evaporation is the greatest source (about 90%) of precipitation.

Rainfall is the predominant form of precipitation and its distribution over the world and within a country. The former is shown in Figure 4, which is taken from the site http://cics.umd.edu/~yin/GPCP/main.html of the Global Precipitation Climatology Project (GPCP) is an element of the Global Energy and Water Cycle Experiment (GEWEX) of the World Climate Research program (WCRP).

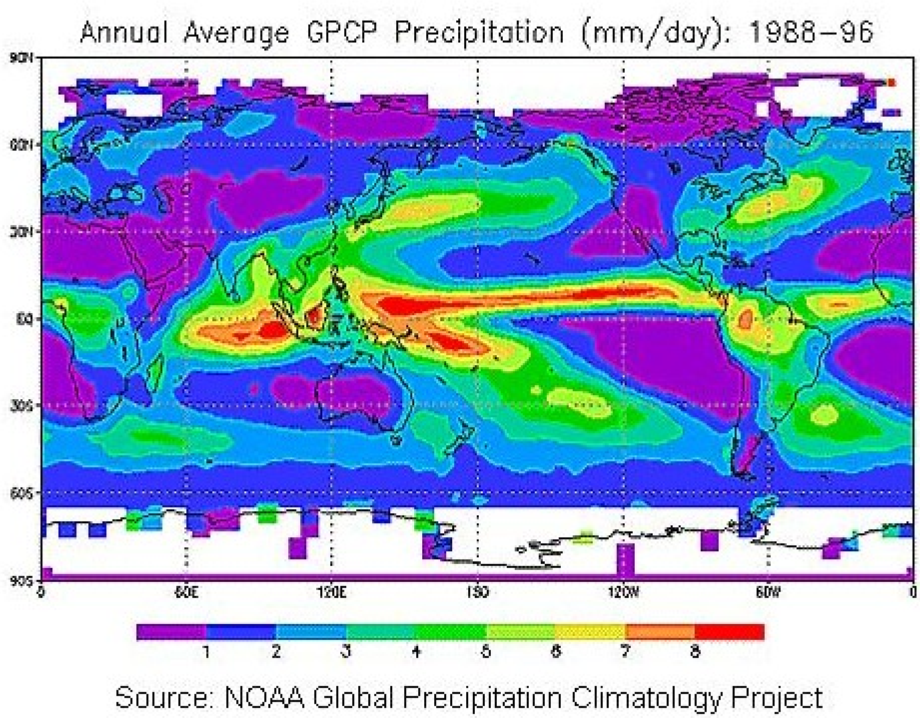


Figure 4. A typical distribution of global precipitation (Courtesy: Global Precipitation Climatology Project)

The distribution of precipitation for our country as recorded by the India Meteorological Department (IMD) is presented in the web-site of IMD http://www.imd.ernet.in/section/climate/. One typical distribution is shown in Figure 5 and it may be observed that rainfall is substantially non-uniform, both in space and over time.

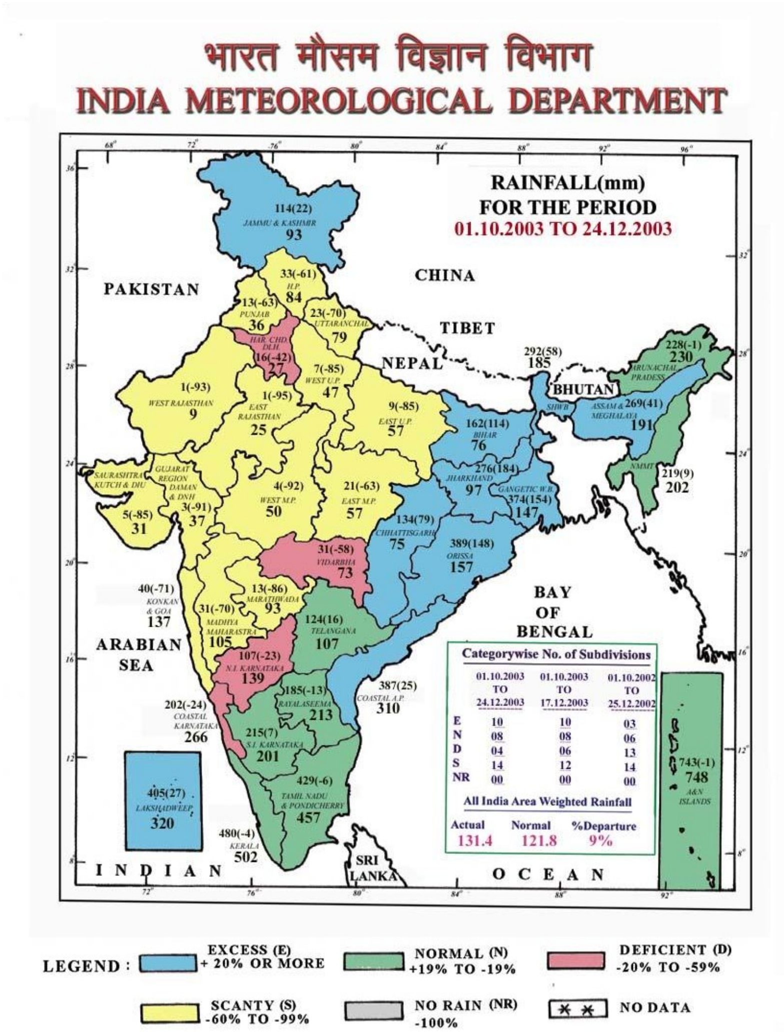


Figure 5. A typical distribution of rainfall within India for a particular week (Courtsey: India Meteorological Department)

India has a typical monsoon climate. At this time, the surface winds undergo a complete reversal from January to July, and cause two types of monsoon. In winter dry and cold air from land in the northern latitudes flows southwest (northeast monsoon), while in summer warm and humid air originates over the ocean and flows in the opposite direction (southwest monsoon), accounting for some 70 to 95 percent of the annual rainfall. The average annual rainfall is estimated as 1170 mm over the country, but varies significantly from place to place. In the northwest desert of Rajasthan, the average annual rainfall is lower than 150 mm/year. In the broad belt extending from Madhya Pradesh up to Tamil Nadu, through Maharastra, parts of Andhra Pradesh and Karnataka, the average annual rainfall is generally lower than 500 mm/year. At the other extreme, more than 10000 mm of rainfall occurs in some portion of the Khasi Hills in the northeast of the country in a short period of four months. In other parts of the northeast (Assam, Arunachal Pradesh, Mizoram, etc.,) west coast and in sub-Himalayan West Bengal the average annual rainfall is about 2500 mm.

Except in the northwest of India, inter annual variability of rainfall in relatively low. The main areas affected by severe droughts are Rajasthan, Gujarat (Kutch and Saurashtra).

The year can be divided into four seasons:

- The winter or northeast monsoon season from January to February.
- The hot season from March to May.
- The summer or south west monsoon from June to September.
- The post – monsoon season from October to December.

The monsoon winds advance over the country either from the Arabian Sea or from the Bay of Bengal. In India, the south-west monsoon is the principal rainy season, which contributes over 75% of the annual rainfall received over a major portion of the country. The normal dates of onset (Figure 6) and withdrawal (Figure 7) of monsoon rains provide a rough estimate of the duration of monsoon rains at any region.

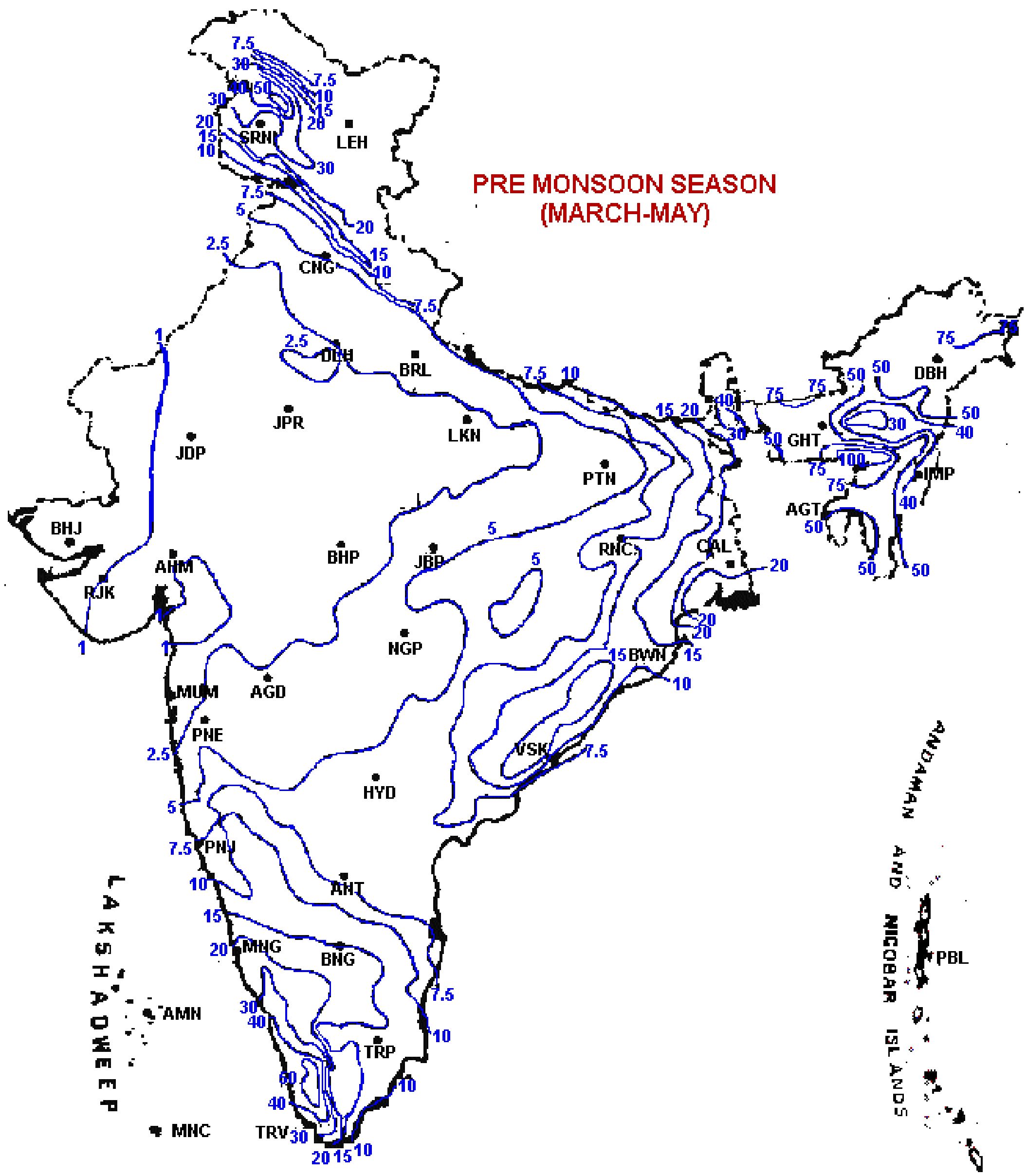


Figure 6. Normal onset dates for Monsoon (Courtsey: India Meteorological Department)

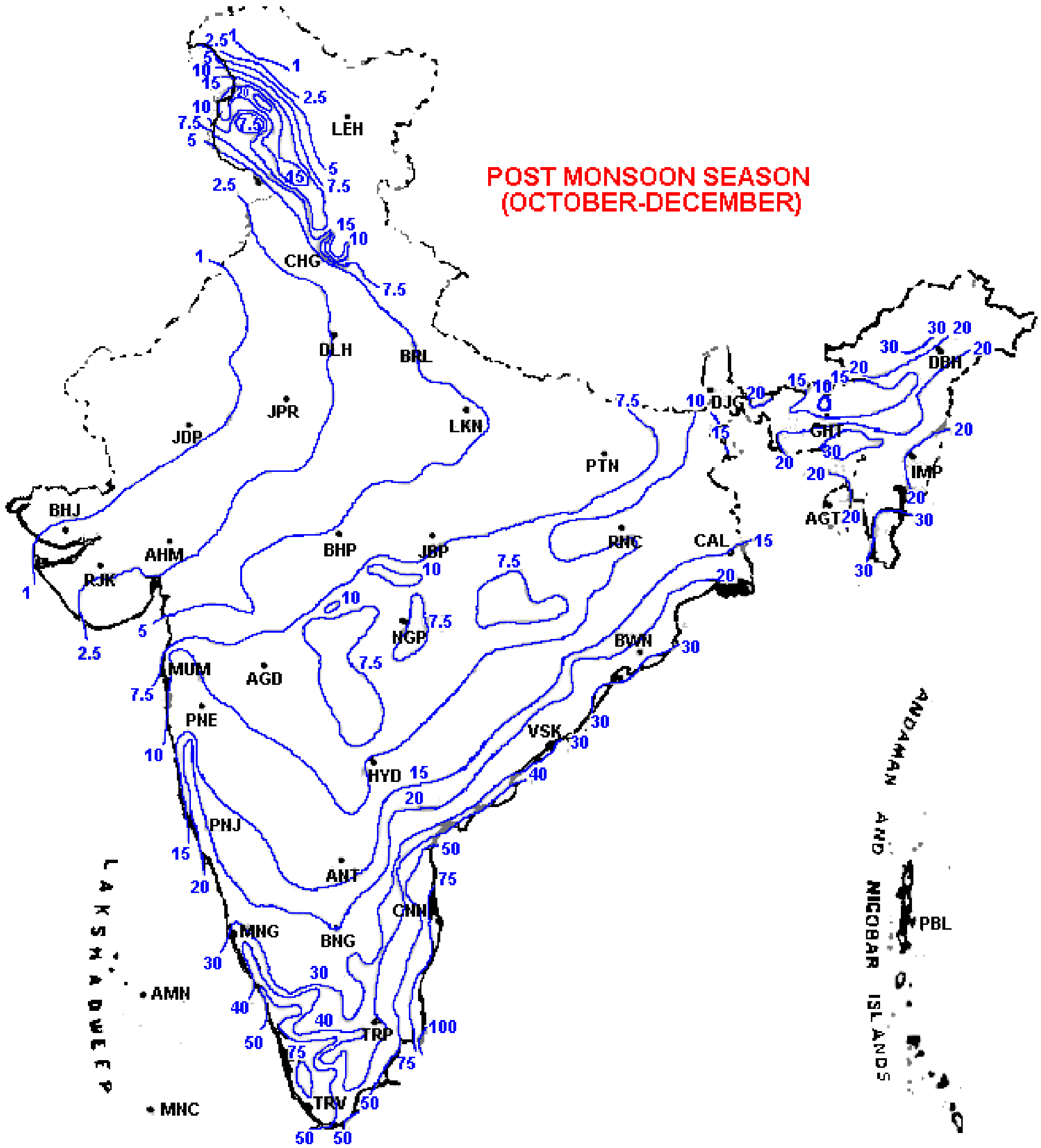


Figure 7. Normal withdrawal dates for Monsoon (Courtsey: India Meteorological Department)

1.1.2 Runoff

Runoff is the water that flows across the land surface after a storm event. As rain falls over land, part of that gets infiltrated the surface as overland flow. As the flow bears down, it notches out rills and gullies which combine to form channels. These combine further to form streams and rivers.

The geographical area which contributes to the flow of a river is called a river or a watershed. The following are the major river basins of our country, and the corresponding figures, as obtained from the web-site of the Ministry of Water Resources, Government of India (http://www.wrmin.nic.in) is mentioned alongside each.

1. Indus (Figure 8)
2. Ganges (Figure 9)
3. Brahmaputra (Figure 10)
4. Krishna (Figure 11)
5. Godavari (Figure 12)
6. Mahanandi (Figure 13)
7. Sabarmati (Figure 14)
8. Tapi (Figure 15)
9. Brahmani-Baitarani (Figure 16)
10. Narmada (Figure 17)
11. Pennar (Figure 18)
12. Mahi (Figure 19)

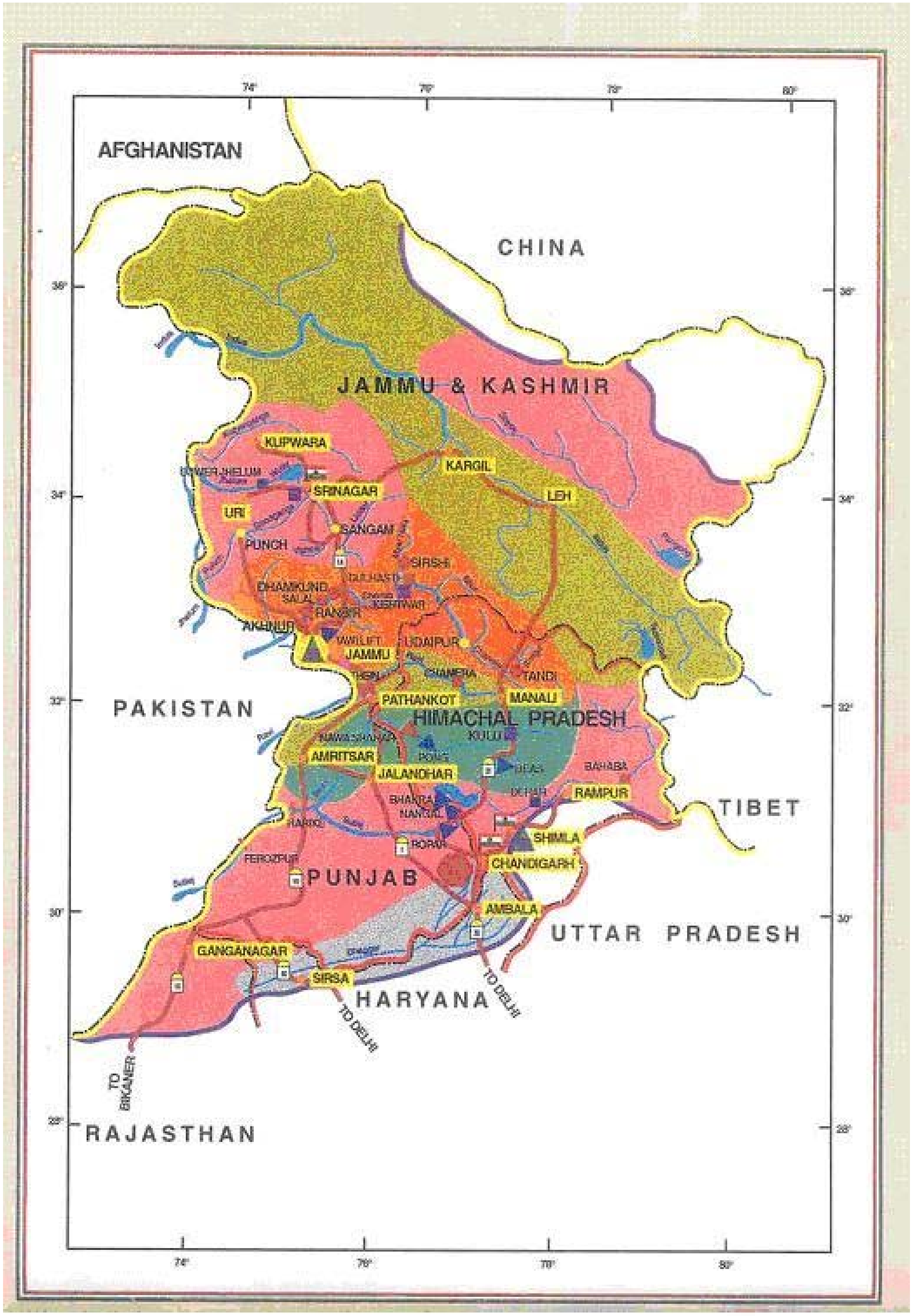


FIGURE 8. INDUS BASIN (PORTION WITHIN INDIA)

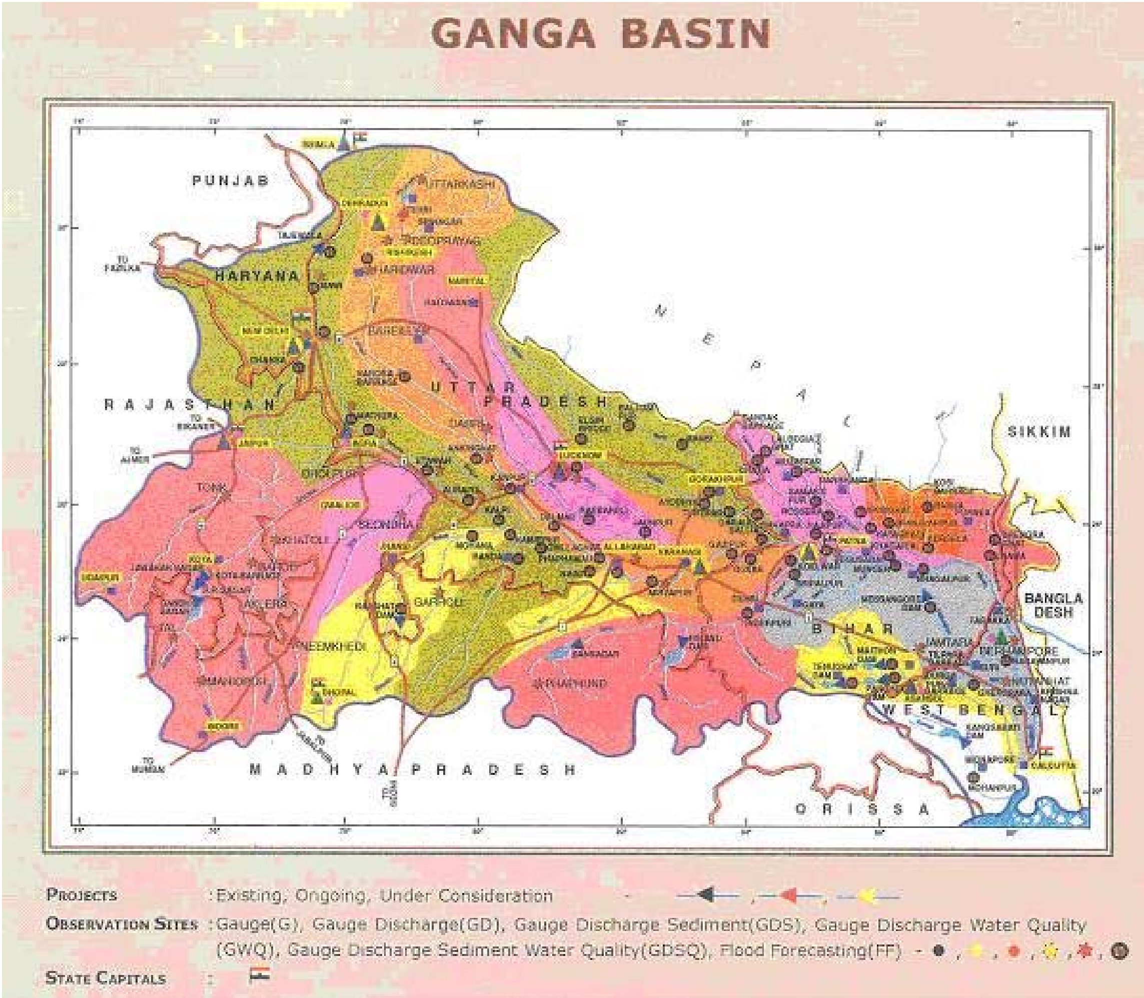


FIGURE 9. GANGA BASIN

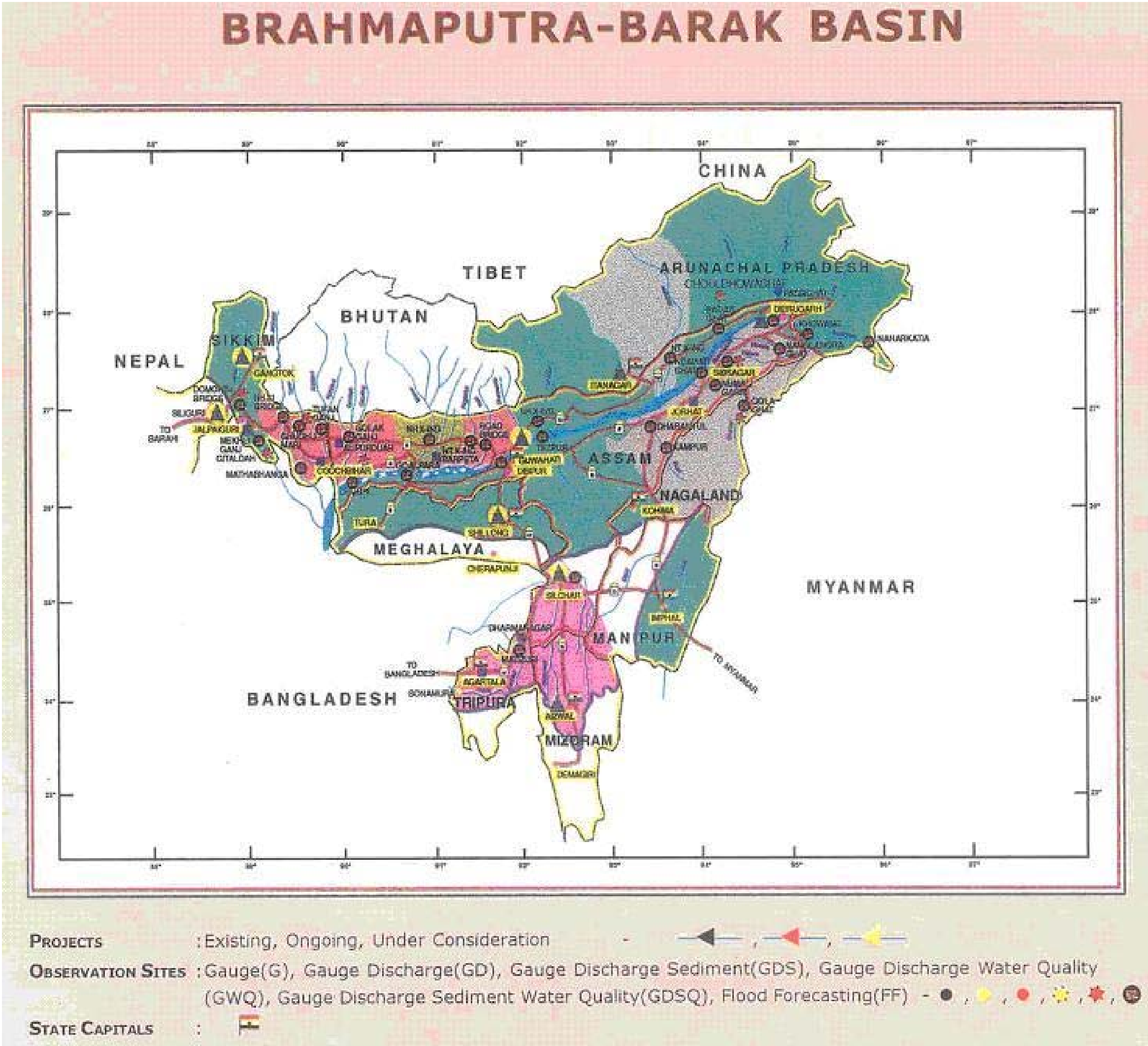


FIGURE 10. BRAHMAPUTRA AND BARAK BASINS (WITHIN INDIA)

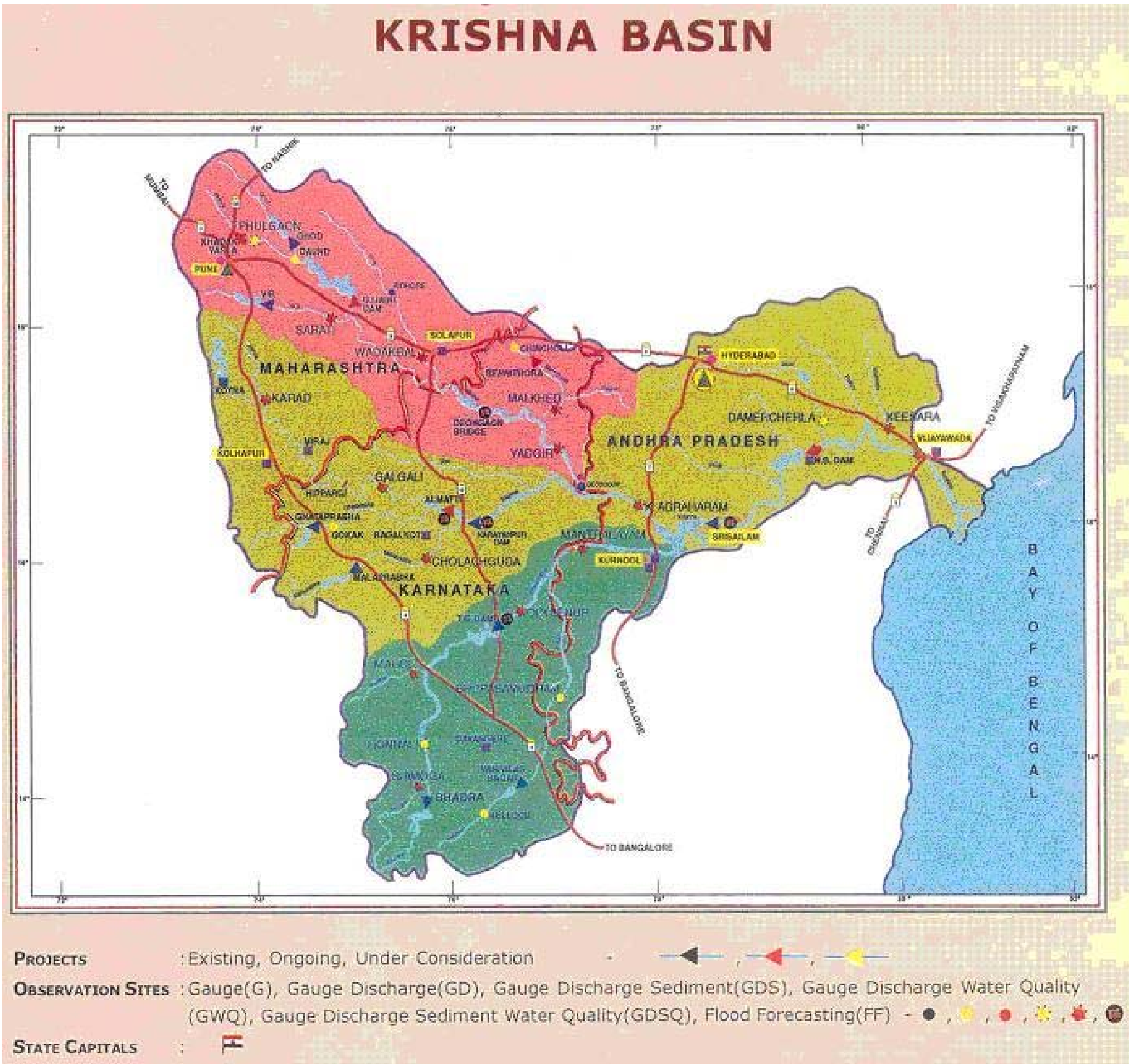


FIGURE 11. KRISHNA BASIN

FIGURE 12 GODAVARI BASIN

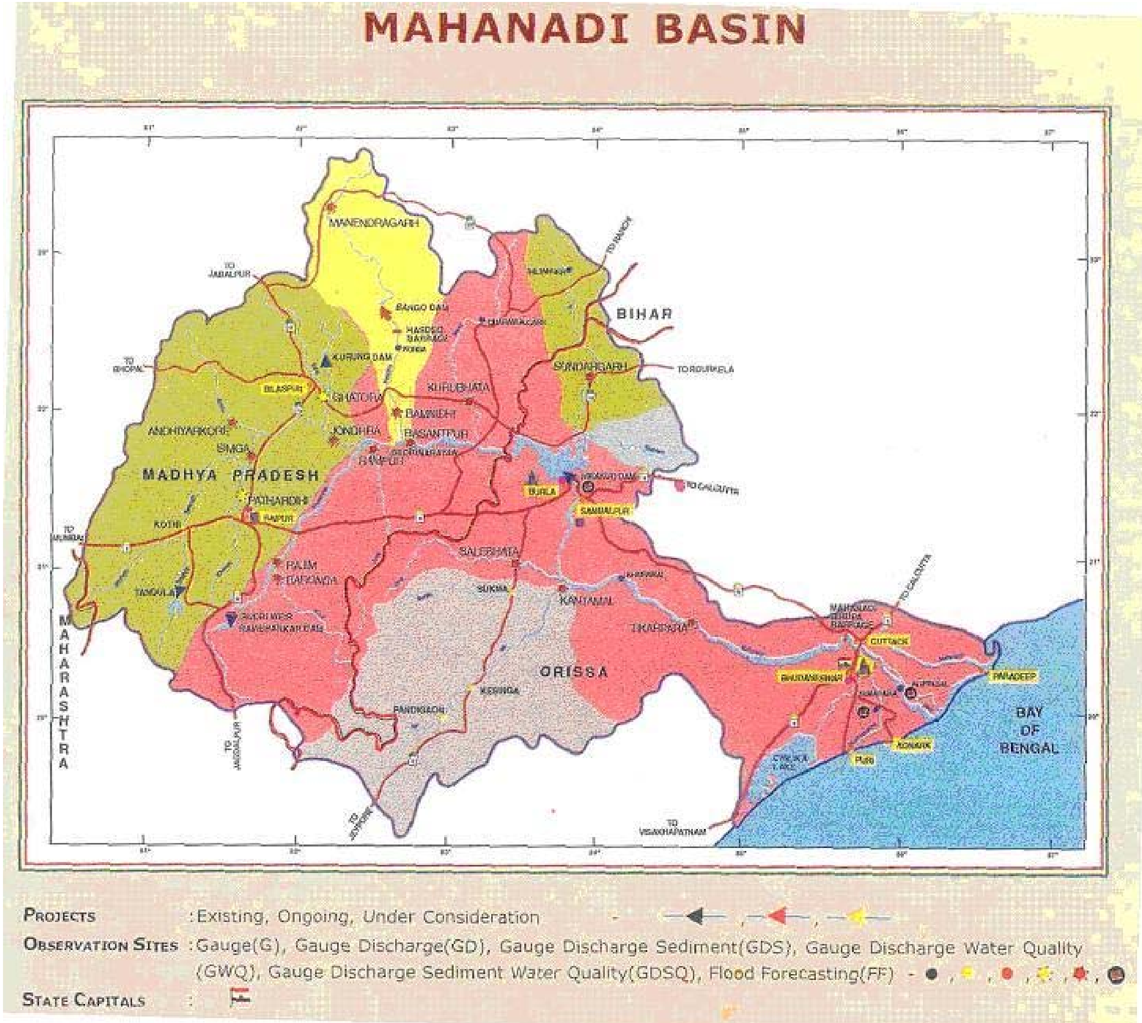


FIGURE 13. MAHANADI BASIN

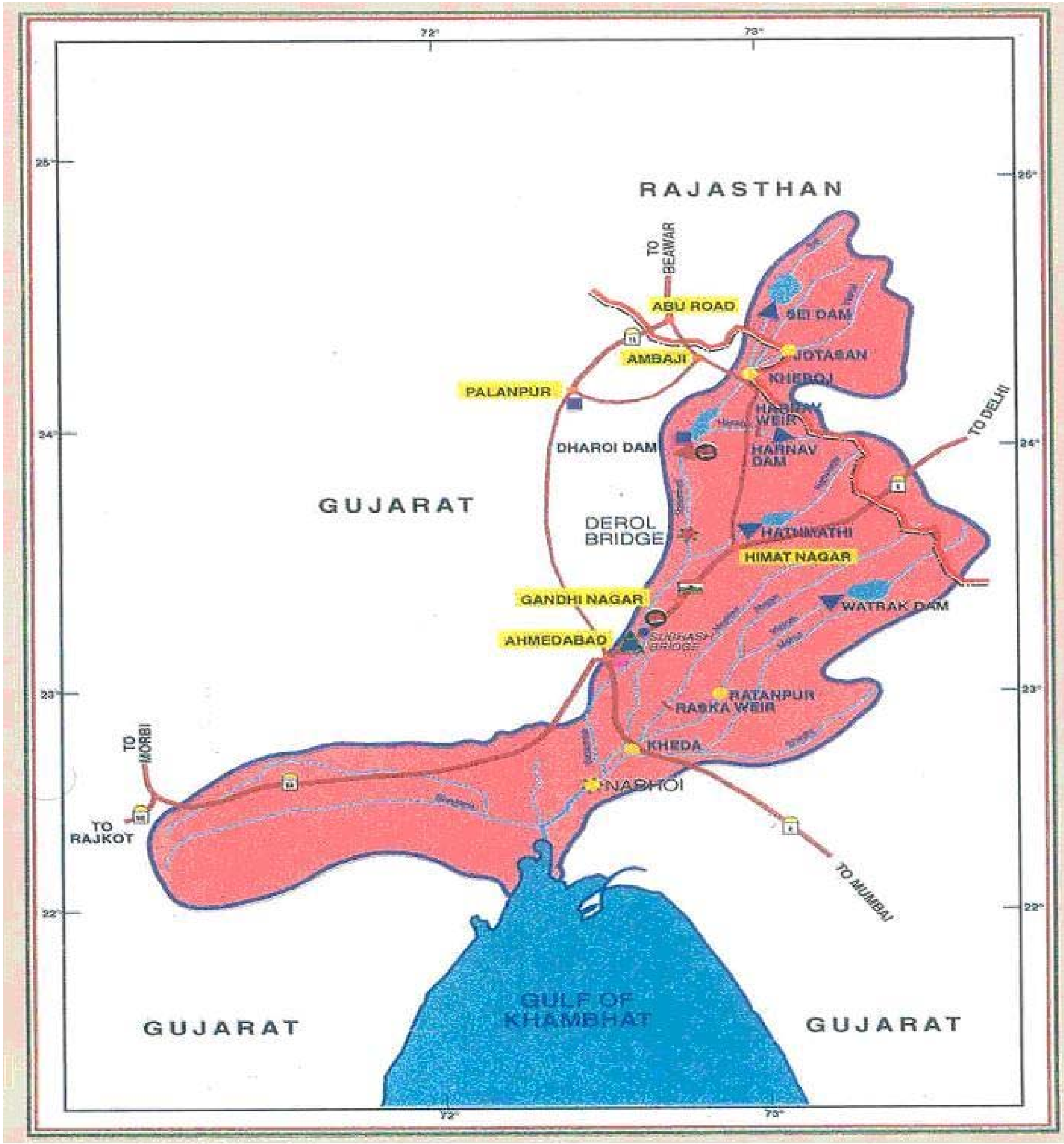


FIGURE 14 SABARMATI BASIN

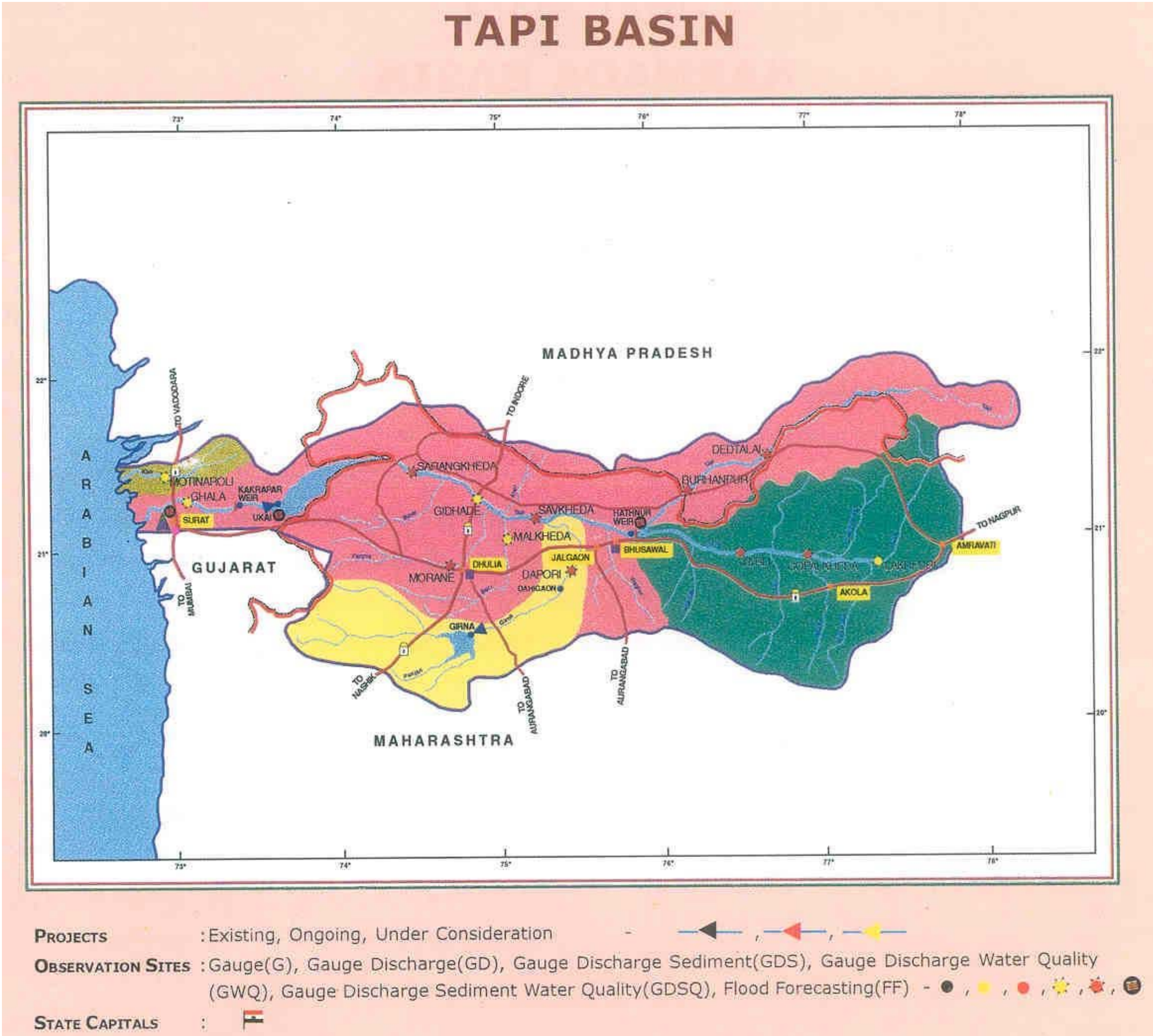


FIGURE 15. TAPI RIVER BASIN

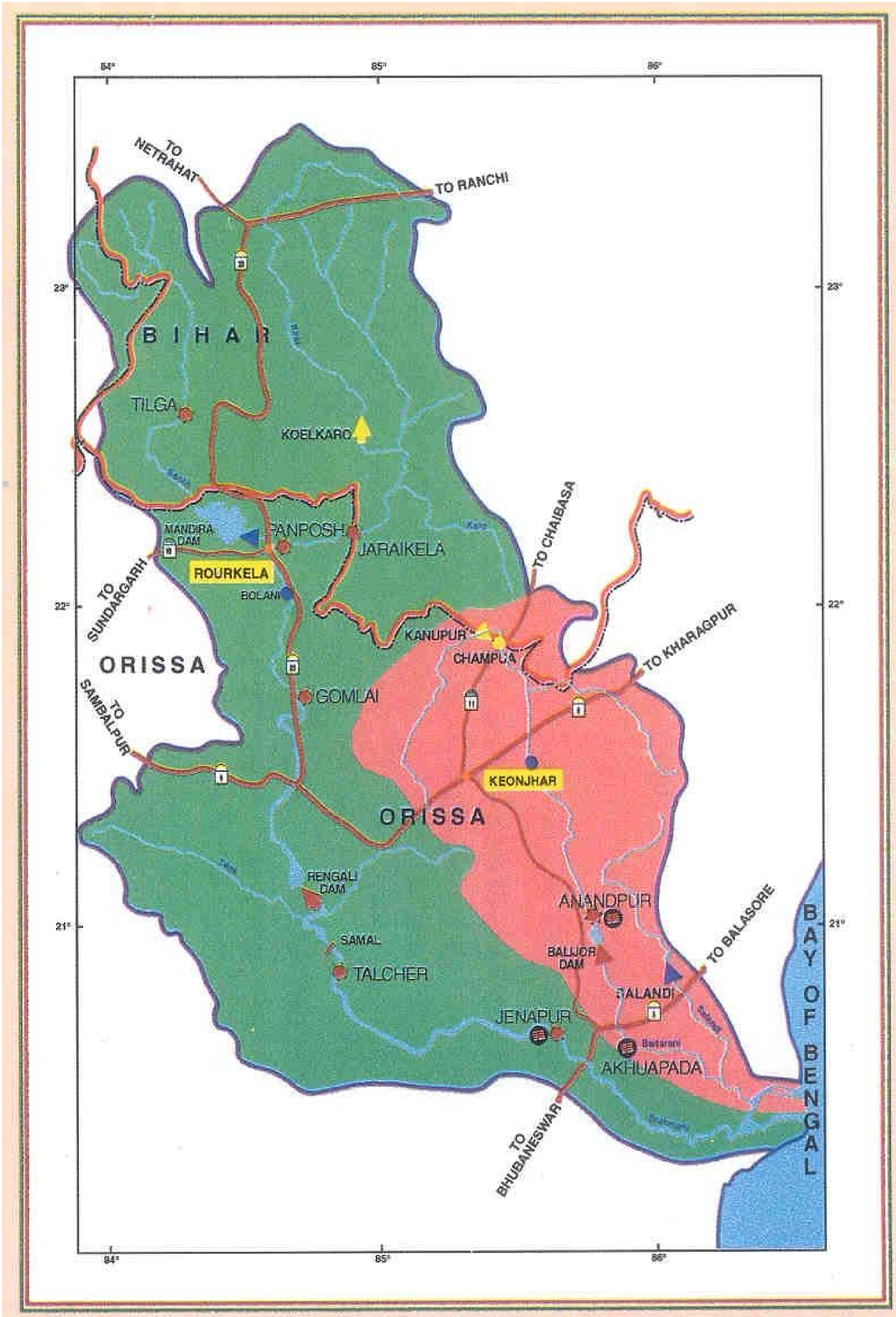


FIGURE 16. BRAHMANU AND BAITARANI RIVER BASINS

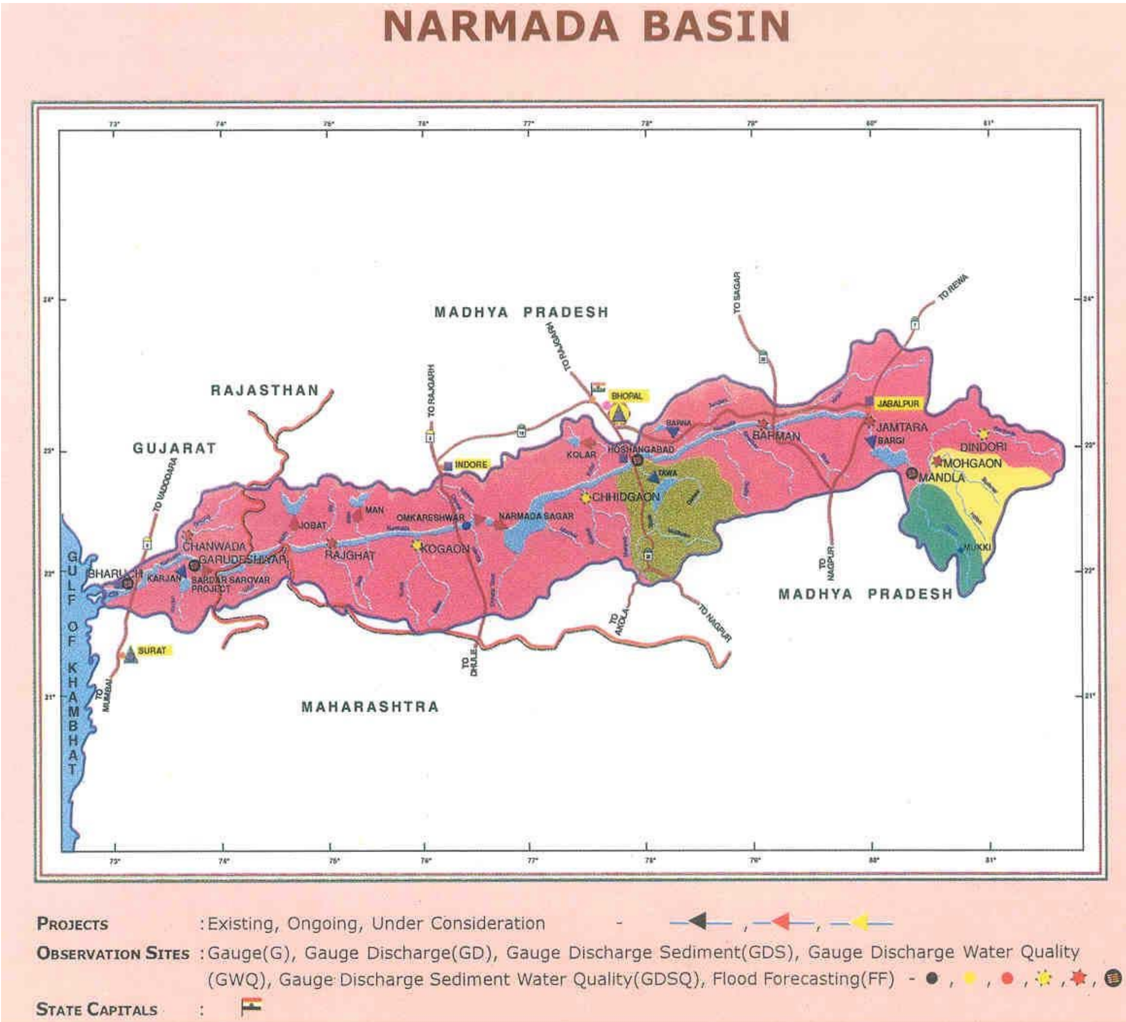


FIGURE 17. NARMADA RIVER BASIN

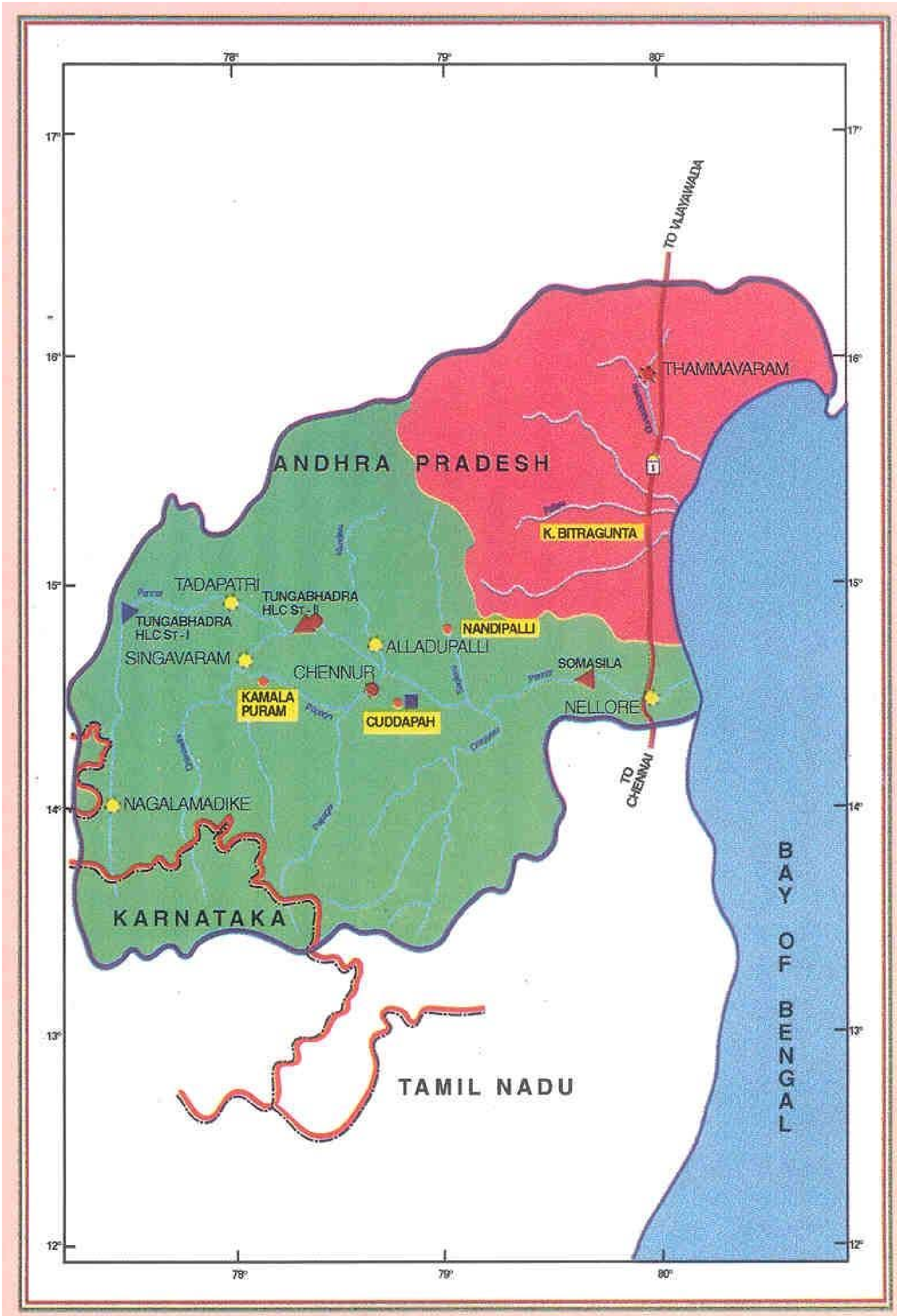


FIGURE 18. PENNAR RIVER BASIN

FIGURE 19. MAHI RIVER BASIN

Some statistical information about the surface water resources of India, grouped by major river basin units, have been summarised as under. The inflow has been collected from the inistry of Water Resources, Government of India web-site.

	River basin unit	Location	Draining into	Catchment area km^2	Average annual runoff (km^3)	Utilizable surface water (km^3)
1	Ganges-	Northeast	Bangladesh			
	Brahmaputra-					
	Meghna			861 452	525.02*	250.0
	-Ganges			(1)	537.24*	24.0
	-			193	48.36	-
	Brahmaputra(2)			413(1)		
	-Barak(3)			41	113.53	24.3
2		Southwest	Arabian	723(1)		
	West flowing	coast	sea			
	river from Tadri			56 177	110.54	76.3
3	to Kanyakumari				87.41	11.9
4		Central	Bay of			
	Godavari	Central-	Bengal	312 812	78.12	58.0
5	West flowing	West	Arabian	55 940	73.31*	46.0
6	rivers from Tapi	coast	sea		66.88*	50.0
7	to Tadri	Central		258 948	45.64	34.5
8	Krishan	Northwest	Bay of	321	31.00*	-
9	Indus	Central-	Bengal	289(1)		
	Mahanadi	east	Pakistan	141 589	28.48	18.3
10	Namada(5)	Central-	Bay of	98 796	22.52	13.1
11	Minor rivers of	west	Bengal	36		
	the northeast	Extreme	Arabian	302(1)		
	Brahmani-	northeast	sea		21.36	19.0
12	Baitarani	Northeast	Myanmar	51 822	16.46	16.7
13	East flowing	Central-	and	86 643		
	rivers between	east coast	Bangladesh			
	Mahanadi &		Bay of		15.10	15.0
14	Pennar		Bengal	81 155		
		South	Bay of	100 139	14.88	14.5
15	Cauvery(4)	Southeast	Bengal		12.37	6.8
16	East flowing	coast			11.02	3.1
17	rivers between			321 851	6.32	6.9
18	Kanyakumari		Bay of		3.81	1.9
19	and Pennar	Northwest	Bengal	65 145	negligible	-
20		coast	Bay of	29 196		
	West flowing	Central-	Bengal	34 842		
	rivers of Kutsh	west		55 213		
	and Saurashtra	Northeast		21 674		

	Tapi Subernarekha Mahi Pennar Sabarmati Rajasthan and inland basin	Northwest Southeast Northwest northwest	Arabian sea Arabian sea Bay of Bengal Arabian sea Bay of Bengal Arabian sea -	-		
			Total	3 227 121	1 869.35	690.3

* Earlier estimates reproduced from Central Water Commission (1988).

Notes:

(1) Areas given are those in India territory.
(2) The potential indicated for the Brahmaputra is the average annual flow at Jogighopa situated 85 km upstream of the India-Bangladesh border. The area drained by the tributaries such as the Champamati, Gaurang, Sankosh, Torsa, Jaldhaka and Tista joining the Brahmaputra downstream of Jogighopa is not accounted for in this assessment.
(3) The potential for the Barak and others was determined on the basis of the average annual flow at Badarpurghat (catchment area: 25 070 km2) given in a Brahmaputra Board report on the Barak sub-basin.
(4) The assessment for Cauvery was made by the Cauvery Fact Finding Committee in 1972 based on 38 years' flow data at Lower Anicut on Coleroon. An area of nearly 8 000 km2 in the delta is not accounted for in this assessment.
(5) The potential of the Narmada basin was determined on the basis of catchment area proportion from the potential assessed at Garudeshwar (catchment area: 89 345 km2) as given in the report on Narmada Water Disputes Tribunal Decision (1978).

1.1.3 Storage

Portion of the precipitation falling on land surface which does not flow out as runoff gets stored as either as surface water bodies like ***Lakes***, ***Reservoirs*** and ***Wetlands*** or as sub-surface water body, usually called ***Ground water***.

Ground water storage is the water infiltrating through the soil cover of a land surface and traveling further to reach the huge body of water underground. As mentioned earlier, the amount of ground water storage is much greater than that of lakes and rivers. However, it is not possible to extract the entire groundwater by practicable means. It is interesting to note that the groundwater also is in a state of continuous movement – flowing from regions of higher potential to lower. The rate of movement, however, is exceptionally small compared to the surface water movement.

The following definitions may be useful:

Lakes**:** Large, naturally occurring inland body of water

Reservoirs**:** Artificial or natural inland body of water used to store water to meet various demands.

Wet Lands**:** Natural or artificial areas of shallow water or saturated soils that contain or could support water–loving plants.

1.1.4 Evapotranspiration

Evapotranspiration is actually the combination of two terms – evaporation and transpiration. The first of these, that is, evaporation is the process of liquid converting into vapour, through wind action and solar radiation and returning to the atmosphere. Evaporation is the cause of loss of water from open bodies of water, such as lakes, rivers, the oceans and the land surface. It is interesting to note that ocean evaporation provides approximately 90 percent of the earth's precipitation. However, living near an ocean does not necessarily imply more rainfall as can be noted from the great difference in the amount of rain received between the east and west coasts of India.

Transpiration is the process by which water molecules leaves the body of a living plant and escapes to the atmosphere. The water is drawn up by the plant root system and part of that is lost through the tissues of plant leaf (through the stomata). In areas of abundant rainfall, transpiration is fairly constant with variations occurring primarily in the length of each plants growing season. However, transpiration in dry areas varies greatly with the root depth.

Evapotranspiration, therefore, includes all evaporation from water and land surfaces, as well as transpiration from plants.

1.1.5 Water resources potential

1.1.5.1 Surface water potential:

The average annual surface water flows in India has been estimated as 1869 cubic km. This is the ***utilizable surface water potential*** in India. But the amount of water that can be actually put to beneficial use is much less due to severe limitations posed by Physiography, topography, inter-state issues and the present state of technology to harness water resources economically. The recent estimates made by the ***Central Water Commission***, indicate that the water resources is utilizable through construction of structures is about 690 cubic km (about 36% of the total). One reason for this vast difference is that not only does the whole rainfall occur in about four months a year but the spatial and temporal distribution of rainfall is too uneven due to which the annual average has very little significance for all practical purposes.

Monsoon rain is the main source of fresh water with 76% of the rainfall occurring between June and September under the influence of the southwest monsoon. The average annual precipitation in volumetric terms is 4000 cubic km. The average annual surface flow out of this is 1869 cubic km, the rest being lost in infiltration and evaporation.

1.1.5.2 Ground water potential:

The potential of dynamic or *rechargeable* ground water resources of our country has been estimated by the ***Central Ground Water Board*** to be about 432 cubic km.

Ground water recharge is principally governed by the intensity of rainfall as also the soil and aquifer conditions. This is a dynamic resource and is replenished every year from natural precipitation, seepage from surface water bodies and conveyance systems return flow from irrigation water, etc.

The highlighted terms are defined or explained as under:

Utilizable surface water potential: This is the amount of water that can be purpose fully used, without any wastage to the sea, if water storage and conveyance structures like dams, barrages, canals, etc. are suitably built at requisite sites.

Central Water Commission: Central Water Commission is an attached office of Ministry of Water Resources with Head Quarters at New Delhi. It is a premier technical organization in the country in the field of water resources since 1945.

The commission is charged with the general responsibility of initiating, coordinating and furthering, in consultation with the State Governments concerned, schemes for control, conservation and utilization of water resources throughout the country, for purpose of flood control, irrigation, navigation, drinking water supply and water power development.

Central Ground Water Board: It is responsible for carrying out nation-wide surveys and assessment of groundwater resources and guiding the states appropriately in scientific and technical matters relating to groundwater. The Central Ground Water Board has generated valuable scientific and technical data through regional hydro geological surveys, groundwater exploration, resource and water quality monitoring and research and development. It assists the States in developing broad policy guidelines for development and management of groundwater resources including their conservation, augmentation and protection from pollution, regulation of extraction and conjunctive use of surface water and ground water resources. The Central Ground Water Board organizes Mass Awareness Programmes to create awareness on various aspects of groundwater investigation, exploration, development and management.

Ground water recharge: Some of the water that precipitates, flows on ground surface or seeps through soil first, then flows laterally and some continues to percolate deeper into the soil. This body of water will eventually reach a saturated zone and replenish or recharge groundwater supply. In other words, the recuperation of groundwater is called the groundwater recharge which is done to increase the groundwater table elevation. This can be done by many artificial techniques, say, by constructing a detention dam called a water spreading dam or a dike, to store the flood waters and allow for subsequent seepage of water into the soil, so as to increase the groundwater table. It can also be done by the method of rainwater harvesting in small scale, even at individual houses. The all India figure for groundwater recharge volume is 418.5 cubic km and the per capita annual volume of groundwater recharge is 412.9 cubic m per person.

1.1.6 Land and water resources of India

The two main sources of water in India are rainfall and the snowmelt of glaciers in the Himalayas. Although reliable data on snow cover in India are not available, it is estimated that some 5000 glaciers cover about 43000 km2 in the Himalayas with a total volume of locked water estimated at 3870 km3. considering that about 10000 km2 of the Himalayan glacier is located within India, the total water yield from snowmelt contributing to the river runoff in India may be of the order of 200 km3/year. Although snow and glaciers are poor producers of fresh water, they are good distributors as they yield at the time of need, in the hot season.

The total surface flow, including regenerating flow from ground water and the flow from neighbouring countries is estimated at 1869 km^3/year, of which only 690 km^3 are considered as utilizable in view of the constraints of the present technology for water storage and inter – state issues. A significant part (647.2 km^3/year) of these estimated water resources comes from neighbouring countries; 210.2 km3/year from Nepal, 347 km3/year from China and 90 km3/year from Bhutan. An important part of the surface water resources leaves the country before it reaches the sea: 20 km^3/year to Myanmar, 181.37 km^3/year to Pakistan and 1105.6 km3/year to Bangladesh ("Irrigation in Aisa in Figures", Food and Agricultural Organisation of the United Nations, Rome, 1999; http://www.fao.org/ag/agL/public.stm). For further information, one may also check the web-site "Earth Trends" http://elearthtrends.wri.org.

The land and water resources of India may be summarized as follows.

Geographical area	329 million hectare
Natural runoff (Surface water and ground water)	1869 cubic km/year
Estimated utilizable surface water potential	690 cubic km/year
Ground water resources	432 cubic km/year
Available ground water resource for irrigation	361 cubic km/year
Net utilizable ground water resource for irrigation	325 cubic km/year

1.1.7 International indicators for comparing water resources potential

Some of the definitions used to quantify and compare water resource potential internationally are as follows:

1. ***Internal Renewable Water Resources (IRWR)***: Internal Renewable Water Resources are the surface water produced internally, i.e., within a country. It is that part of the water resources generated from endogenous precipitation. It is the sum of the surface runoff and groundwater recharge occurring inside the countries' borders. Care is taken strictly to avoid double counting of their common part. The IRWR figures are the only water resources figures that can be added up for regional assessment and they are being used for this purpose.

2. ***Surface water produced internally***: Total surface water produced internally includes the average annual flow of rivers generated from endogenous precipitation (precipitation occurring within a country's borders). It is the amount of water produced within the boundary of a country, due to precipitation. Natural incoming flow originating from outside a country's borders is not included in the total.

3. ***Groundwater recharge***: The recuperation of groundwater is called the groundwater recharge. This is requisite to increase the groundwater table elevation. This can be done by many artificial techniques, say, by constructing a detention dam called a water spreading dam or a dike, to store the flood waters and allow for subsequent seepage of water into the soil, so as to increase the groundwater table. It can also be done by the method of rainwater harvesting in small scale, even at individual houses. The groundwater recharge volume is 418.5 cubic km and the per capita annual volume of groundwater recharge is 412.9 cubic m per person.

4. ***Overlap***: It is the amount of water quantity, coinciding between the surface water produced internally and the ground water produced internally within a country, in the calculation of the Total Internal Renewable Water Resources of the country. Hence, Overlap = Total IRWR- (Surface water produced internally + ground water produced internally). The overlap for Indian water resources is 380 cubic km.

5. ***Total internal Renewable Water Resources***: The Total Internal Renewable Water Resources are the sum of IRWR and incoming flow originating outside the countries' borders. The total renewable water resources of India are 1260.5 cubic km.

6. ***Per capita Internal Renewable Water Resources***: The Per capita annual average of Internal Renewable Water Resources is the amount of average IRWR, per capita, per annum. For India, the Per capita Internal Renewable Water Resources are 1243.6 cubic m.

7. ***Net renewable water resources***: The total natural renewable water resources of India are estimated at 1907.8 cubic km per annum, whereas the total actual renewable water resources of India are 1896.7 cubic km.

8. ***Per capita natural water resources***: The present per capita availability of natural water, per annum is 1820 cubic m, which is likely to fall to 1341 cubic m, by 2025.

9. ***Annual water withdrawal***: The total amount of water withdrawn from the water resources of the country is termed the annual water withdrawal. In India, it amounts 500000 to million cubic m.

10. ***Per capita annual water withdrawal***: It is the amount of water withdrawn from the water resources of the country, for various purposes. The per capita annual total water withdrawal in India is 592 cubic m per person.

The above definitions have been provided by courtesy of the following web-site: http://earthtrends.wri.org/text/theme2vars.htm.

1.1.8 Development of water resources

Due to its multiple benefits and the problems created by its excesses, shortages and quality deterioration, water as a resource requires special attention. Requirement of technological/engineering intervention for development of water resources to meet the varied requirements of man or the human demand for water, which are also unevenly distributed, is hence essential.

The development of water resources, though a necessity, is now pertinent to be made sustainable. The concept of sustainable development implies that development meets the needs of the present life, without compromising on the ability of the future generation to meet their own needs. This is all the more important for a resource like water. Sustainable development would ensure minimum adverse impacts on the quality of air, water and terrestrial environment. The long term impacts of global climatic change on various components of hydrologic cycle are also important.

India has sizeable resources of water and a large cultivable land but also a large and growing population to feed. Erratic distribution of rainfall in time and space leads to conditions of floods and droughts which may sometimes occur in the same region in the same year. India has about 16% of the world population as compared to only 4% of the average annual runoff in the rivers.

With the present population of more than 1000 million, the per capita water availability comes to about 1170 m3 per person per year. Here, the average does not reflect the large disparities from region to region in different parts of the country. Against this background, the problems relating to water resources development and management have been receiving critical attention of the Government of India. The country has prepared and adopted a comprehensive ***National Water Policy*** in the year 1987, revised in 2002 with a view to have a systematic and scientific development of it water resources. This has been dealt with in Lesson 1.3, “Policies for water resources development”.

Some of the salient features of the ***National Water Policy*** (2002) are as follows:

- Since the distribution of water is spatially uneven, for water scarce areas, local technologies like rain water harvesting in the domestic or community level has to be implemented.

- Technology for/Artificial recharge of water has also to be bettered.
- Desalination methods may be considered for water supply to coastal towns.

1.1.9 Present water utilization in India

Irrigation constitutes the main use of water and is thus focal issue in water resources development. As of now, ***irrigation*** use is 84 percent of total water use. This is much higher than the world's average, which is about 65 percent. For advanced nations, the figure is much lower. For example, the irrigation use of water in USA is around 33 percent. In India, therefore, the remaining 16 percent of the total water use accounts for Rural domestic and livestock use, Municipal domestic and public use, Thermal-electric power plants and other industrial uses.
The term ***irrigation*** is defined as the artificial method of applying water to crops. Irrigation increases crop yield and the amount of land that can be productively farmed, stabilizes productivity, facilitates a greater diversity of crops, increases farm income and employment, helps alleviate poverty and contributes to regional development.

1.1.10 Need for future development of water resources

The population of India has been estimated to stabilize by about 2050 A.D. By that time, the present population of about 1000 million has been projected to be about 1800 million (considering the low, medium and high estimates of 1349 million 1640 million and 1980 million respectively). The present food grain availability of around 525 grams per capita per day is also presumed to rise to about 650 grams, considering better socio-economic lifestyle (which is much less than the present figures of 980 grams and 2850 grams per capita per day for China and U.S.A., respectively). Thus, the annual food grain requirement for India is estimated to be about 430 MT. Since the present food grain production is just sufficient for the present population, it is imperative that additional area needs to be brought under cultivation. This has been estimated to be 130 Mha for food crop alone and 160 Mha for all crops to meet the demands of the country by 2050 A.D.

Along with the inevitable need to raise food production, substantial thrust should be directed towards water requirement for domestic use. The national agenda for governance aims to ensure provision of potable water supply to every individual in about five years time. The National Water Policy (2002) has accorded topmost water allocation priority to drinking water. Hence, a lot of technological intervention has to be made in order to implement the decision. But this does not mean that unlimited funds would be allocated for the drinking water sector. Only 20% of urban demand is meant for ***consumptive use*** . A major concern will therefore be the treatment of urban domestic effluents.

Major industrial thrust to steer the economy is only a matter of time. By 2050, India expects to be a major industrial power in the world. Industry needs water fresh or recycled. Processing industries depend on abundance of water. It is estimated that 64 cubic km of water will be needed by 2050 A.D. to sustain the industries. Thermal power generation needs water including a small part that is consumptive. Taking into account the electric power scenario in 2050 A.D., energy related requirement (evaporation and consumptive use) is estimated at 150 cubic km.

Note:
Consumptive use: Consumptive use is the amount of water lost in evapo-transpiration from vegetation and its surrounding land to the atmosphere, inclusive of the water used by the plants for building their tissues and to carry on with their metabolic processes. Evapo-transpiration is the total water lost to the atmosphere from the vegetative cover on the land, along with the water lost from the surrounding water body or land mass.

1.1.11 Sustainable water utilisation

The quality of water is being increasingly threatened by pollutant load, which is on the rise as a consequence of rising population, urbanization, industrialization, increased use of agricultural chemicals, etc. Both the surface and ground water have gradually increased in contamination level. Technological intervention in the form of providing sewerage system for all urban conglomerates, low cost sanitation system for all rural households, water treatment plants for all industries emanating polluted water, etc. has to be made. Contamination of ground water due to over-exploitation has also emerged as a serious problem. It is difficult to restore ground water quality once the aquifer is contaminated. Ground water contamination occurs due to human interference and also natural factors . To promote human health, there is urgent need to prevent contamination of ground water and also promote and develop cost-effective techniques for purifying contaminated ground water for use in rural areas like solar stills.

In summary, the development of water resources potential should be such that in doing so there should not be any degradation in the quality or quantity of the resources available at present. Thus the development should be sustainable for future.

References to web-sites:

1. http://cics.umd.edu/~yin/GPCP/main.html
2. http://www.imd.ernet.in/section/climate/
3. http://www.wrmin.nic.in/

Bibliography:

1. Linsley, R K and Franzini, J B (1979) “Water Resources Engineering”, Third Edition, McGraw Hill, Inc.
2. Mays, L (2001) “Water Resources Engineering”, First Edition, John Wiley and Sons.

Module 1

Principles of Water Resources Engineering

Lesson
2
Concepts for Planning Water Resources Development

Instructional Objectives

On completion of this lesson, the student shall be able to know:

1. Principle of planning for water resource projects
2. Planning for prioritizing water resource projects
3. Concept of basin – wise project development
4. Demand of water within a basin
5. Structural construction for water projects
6. Concept of inter – basin water transfer project
7. Tasks for planning a water resources project

1.2.0 Introduction

Utilisation of available water of a region for use of a community has perhaps been practiced from the dawn of civilization. In India, since civilization flourished early, evidences of water utilization has also been found from ancient times. For example at Dholavira in Gujarat water harvesting and drainage systems have come to light which might had been constructed somewhere between 300-1500 BC that is at the time of the Indus valley civilization. In fact, the Harappa and Mohenjodaro excavations have also shown scientific developments of water utilization and disposal systems. They even developed an efficient system of irrigation using several large canals. It has also been discovered that the Harappan civilization made good use of groundwater by digging a large number of wells. Of other places around the world, the earliest dams to retain water in large quantities were constructed in Jawa (Jordan) at about 3000 BC and in Wadi Garawi (Egypt) at about 2660 BC. The Roman engineers had built log water conveyance systems, many of which can still be seen today, *Qanats* or underground canals that tap an alluvial fan on mountain slopes and carry it over large distances, were one of the most ingenious of ancient hydro-technical inventions, which originated in Armenia around 1000BC and were found in India since 300 BC.

Although many such developments had taken place in the field of water resources in earlier days they were mostly for satisfying drinking water and irrigation requirements. Modern day projects require a scientific planning strategy due to:

1. Gradual decrease of per capita available water on this planet and especially in our country.
2. Water being used for many purposes and the demands vary in time and space.

3. Water availability in a region – like county or state or watershed is not equally distributed.
4. The supply of water may be from rain, surface water bodies and ground water.

This lesson discusses the options available for planning, development and management of water resources of a region systematically.

1.2.1 Water resources project planning

The goals of water resources project planning may be by the use of constructed facilities, or structural measures, or by management and legal techniques that do not require constructed facilities. The latter are called non-structural measures and may include rules to limit or control water and land use which complement or substitute for constructed facilities. A project may consist of one or more structural or non-structural resources. Water resources planning techniques are used to determine what measures should be employed to meet water needs and to take advantage of opportunities for water resources development, and also to preserve and enhance natural water resources and related land resources.

The scientific and technological development has been conspicuously evident during the twentieth century in major fields of engineering. But since water resources have been practiced for many centuries, the development in this field may not have been as spectacular as, say, for computer sciences. However, with the rapid development of substantial computational power resulting reduced computation cost, the planning strategies have seen new directions in the last century which utilises the best of the computer resources. Further, economic considerations used to be the guiding constraint for planning a water resources project. But during the last couple of decades of the twentieth century there has been a growing awareness for environmental sustainability. And now, environmental constrains find a significant place in the water resources project (or for that matter any developmental project) planning besides the usual economic and social constraints.

1.2.2 Priorities for water resources planning

Water resource projects are constructed to develop or manage the available water resources for different purposes. According to the National Water Policy (2002), the water allocation priorities for planning and operation of water resource systems should broadly be as follows:

1. Domestic consumption
   This includes water requirements primarily for drinking, cooking, bathing, washing of clothes and utensils and flushing of toilets.

2. Irrigation
   Water required for growing crops in a systematic and scientific manner in areas even with deficit rainfall.

3. Hydropower
   This is the generation of electricity by harnessing the power of flowing water.

4. Ecology / environment restoration
   Water required for maintaining the environmental health of a region.

5. Industries
   The industries require water for various purposes and that by thermal power stations is quite high.

6. Navigation
   Navigation possibility in rivers may be enhanced by increasing the flow, thereby increasing the depth of water required to allow larger vessels to pass.

7. Other uses
   Like entertainment of scenic natural view.

This course on Water Resources Engineering broadly discusses the facilities to be constructed / augmented to meet the demand for the above uses. Many a times, one project may serve more than one purpose of the above mentioned uses.

1.2.3 Basin – wise water resource project development

The total land area that contributes water to a river is called a Watershed, also called differently as the Catchment, River basin, Drainage Basin, or simply a Basin. The image of a basin is shown in Figure 1.

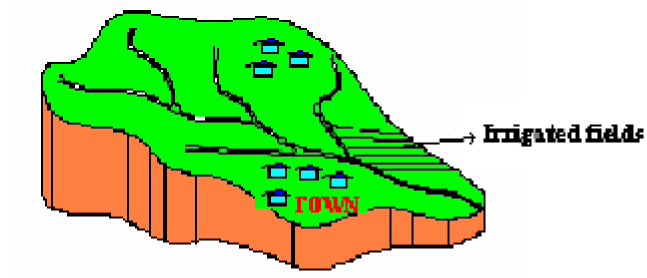


Figure 1. A typical Water Shed

A watershed may also be defined as a geographic area that drains to a common point, which makes it an attractive planning unit for technical efforts to conserve soil and maximize the utilization of surface and subsurface water for crop production. Thus, it is generally considered that water resources development and management schemes should be planned for a hydrological unit such as a Drainage Basin as a whole or for a Sub-Basin, multi-sectorially, taking into account surface and ground water for sustainable use incorporating quantity and quality aspects as well as environmental considerations.

Let us look into the concept of watershed or basin-wise project development in some detail. The objective is to meet the demands of water within the Basin with the available water therein, which could be surface water, in the form of rivers, lakes, etc. or as groundwater. The source for all these water bodies is the rain occurring over the Watershed or perhaps the snowmelt of the glacier within it, and that varies both ***temporally*** and ***spatially***.

Further due to the land surface variations the rain falling over land surface tries to follow the steepest gradient as overland flow and meets the rivers or drains into lakes and ponds. The time for the overland flows to reach the rivers may be fast or slow depending on the obstructions and detentions it meet on the way. Part of the water from either overland flow or from the rivers and lakes penetrates into the ground and recharge the ground water. Ground water is thus available almost throughout the watershed, in the underground aquifers. The variation of the water table is also fairly even, with some rise during rainfall and a gradual fall at other times. The water in the rivers is mostly available during the rains. When the rain stops, part of the ground water comes out to recharge the rivers and that results in the dry season flows in rivers.

Note:

Temporal: That which varies with time
Spatial: That which varies with time

1.2.4 Tools for water resources planning and management

The policy makers responsible for making comprehensive decisions of water resources planning for particular units of land, preferably a basin, are faced with various parameters, some of which are discussed in the following sections.

1.2.4.1 The supply of water

Water available in the unit

This may be divided into three sources

- Rain falling within the region. This may be utilized directly before it reaches the ground, for example, the roof – top rain water harvesting schemes in water scarce areas.

- Surface water bodies. These static (lakes and ponds) and flowing (streams and rivers), water bodies may be utilized for satisfying the demand of the unit, for example by constructing dams across rivers.

- Ground water reservoirs. The water stored in soil and pores of fractured bed rock may be extracted to meet the demand, for example wells or tube – wells.

Water transferred in and out of the unit

If the planning is for a watershed or basin, then generally the water available within the basin is to be used unless there is inter basin water transfer. If however, the unit is a political entity, like a nation or a state, then definitely there shall be inflow or outflow of water especially that of flowing surface water. Riparian rights have to be honored and extraction of more water by the upland unit may result in severe tension.

Note: Riparian rights mean the right of the downstream beneficiaries of a river to the river water.

Regeneration of water within the unit

Brackish water may be converted with appropriate technology to supply sweet water for drinking and has been tried in many extreme water scarce areas. Waste water of households may be recycled, again with appropriate technology, to supply water suitable for purposes like irrigation.

1.2.4.2 The demand of water

Domestic water requirement for urban population

This is usually done through an organized municipal water distribution network. This water is generally required for drinking, cooking, bathing and sanitary purposes etc, for the urban areas. According to National Water Policy (2002), domestic water supplies for urban areas under various conditions are given below. The units mentioned “lpcd” stands for Liters per Capita per Day”.

1. 40 lpcd where only spot sources are available
2. 70 lpcd where piped water supply is available but no sewerage system
3. 125 lpcd where piped water supply and sewerage system are both available. 150 lpcd may be allowed for metro cities.

Domestic and livestock water requirement for rural population

This may be done through individual effort of the users by tapping a local available source or through co-operative efforts, like Panchayats or Block Development Authorities. The accepted norms for rural water supply according to National Water Policy (2002) under various conditions are given below.

- 40 lpcd or one hand pump for 250 persons within a walking distance of 1.6 km or elevation difference of 100 m in hills.
- 30 lpcd additional for cattle in Desert Development Programme (DDP) areas.

Irrigation water requirement of cropped fields

Irrigation may be done through individual effort of the farmers or through group cooperation between farmers, like Farmers’ Cooperatives. The demands have to be estimated based on the cropping pattern, which may vary over the land unit due to various factors like; farmer’s choice, soil type, climate, etc. Actually, the term “Irrigation Water Demand” denotes the total quantity and the way in which a crop requires water, from the time it is sown to the time it is harvested.

Industrial water needs

This depends on the type of industry, its magnitude and the quantity of water required per unit of production.

1.2.5 Structural tools for water resource development

This section discusses the common structural options available to the Water Resources Engineer to development the water potential of the region to its best possible extent.

Dams

These are detention structures for storing water of streams and rivers. The water stored in the reservoir created behind the dam may be used gradually, depending on demand.

Barrages

These are diversion structures which help to divert a portion of the stream and river for meeting demands for irrigation or hydropower. They also help to increase the level of the water slightly which may be advantageous from the point of view of increasing navigability or to provide a pond from where water may be drawn to meet domestic or industrial water demand.

Canals/Tunnels

These are conveyance structures for transporting water over long distances for irrigation or hydropower.

These structural options are used to utilise surface water to its maximum possible extent. Other structures for utilising ground water include rainwater detentions tanks, wells and tube wells.

Another option that is important for any water resource project is Watershed Management practices. Through these measures, the water falling within the catchment area is not allowed to move quickly to drain into the rivers and streams. This helps the rain water to saturate the soil and increase the ground water reserve. Moreover, these measures reduce the amount of erosion taking place on the hill slopes and thus helps in increasing the effective lives of reservoirs which otherwise would have been silted up quickly due to the deposition of the eroded materials.

1.2.6 Management tools for water resource planning

The following management strategies are important for water resources planning:

- Water related allocation/re-allocation agreements between planning units sharing common water resource.
- Subsidies on water use
- Planning of releases from reservoirs over time
- Planning of withdrawal of ground water with time.
- Planning of cropping patterns of agricultural fields to optimize the water availability from rain and irrigation (using surface and/or ground water sources) as a function of time
- Creating public awareness to reduce wastage of water, especially filtered drinking water and to inculcate the habit of recycling waste water for purposes like gardening.

- Research in water management: Well established technological inputs are in verge in water resources engineering which were mostly evolved over the last century. Since, then not much of innovations have been put forward. However, it is equally known that quite a few of these technologies run below optimum desired efficiency. Research in this field is essential for optimizing such structure to make most of water resource utilization.

An example for this is the seepage loss in canals and loss of water during application of water in irrigating the fields. As an indication, it may be pointed out that in India, of the water that is diverted through irrigation canals up to the crop growing fields, only about half is actually utilized for plant growth. This example is also glaring since agriculture sector takes most of the water for its assumption from the developed project on water resources.

A good thrust in research is needed to increase the water application efficiently which, in turn, will help optimizing the system.

1.2.7 Inter-basin water transfer

It is possible that the water availability in a basin (Watershed) is not sufficient to meet the maximum demands within the basin. This would require Inter-basin water transfer, which is described below:

The National water policy adopted by the Government of India emphasizes the need for inter-basin transfer of water in view of several water surplus and deficit areas within the country. As early as 1980, the Minister of Water Resources had prepared a National perspective plan for Water resources development. The National Perspective comprises two main components:
a) Himalayan Rivers Development, and
b) Peninsular Rivers Development

Himalayan rivers development

Himalayan rivers development envisages construction of storage reservoirs on the principal tributaries of the Ganga and the Brahamaputra in India, Nepal and Bhutan, along with interlinking canal systems to transfer surplus flows of the eastern tributaries of the Ganga to the west, apart from linking of the main Brahmaputra and its tributaries with the Ganga and Ganga with Mahanadi.

Peninsular rivers development

This component is divided into four major parts:

1. Interlinking of Mahanadi-Godavari-Krishna-Cauvery rivers and building storages at potential sites in these basins.
2. Interlinking of west flowing rivers, north of Mumbai and south of Tapi.
3. Interlinking of Ken-Chambal rivers.
4. Diversion of other west flowing rivers.

The possible quantity of water that may be transferred by ***donor basin*** may be equal to the average water availability of basin minus maximum possible water requirement within basin (considering future scenarios).

Note: A ***Donor basin*** is the basin, which is supplying the water to the downstream basin.

The minimum expected quantity of water for ***recipient basin*** may be equal to the minimum possible water requirement within basin (considering future scenarios) minus average water availability of basin.

Note: A ***Recipient basin*** is the basin, which is receiving the water from the Donor basin.

National Water Development Agency (NWDA) of the Government of India has been entrusted with the task of formalizing the inter-linking proposal in India. So far, the agency has identified some thirty possible links within India for inter-basin transfer based on extensive study of water availability and demand data.

Note:

The National Water Development Agency (NWDA) was set up in July, 1982 as an Autonomous Society under the Societies Registration Act, 1960, to carry out the water balance and other studies on a scientific and realistic basis for optimum utilization of Water Resources of the Peninsular rivers system for preparation of feasibility reports and thus to give concrete shape to Peninsular Rivers Development Component of National Perspective. In 1990, NWDA was also entrusted with the task of Himalayan Rivers Development Component of National Perspective.

Possible components of an inter-basin transfer project include the following:

- Storage Dam in Donor basin to store flood runoff
- Conveyance structure, like canal, to transfer water from donor to recipient basin
- Possible pumping equipments to raise water across watershed-divide

Possible implications of inter-basin transfer: Since a large scale water transfer would be required, it is necessary to check whether there shall be any of the following:

- River bed level rise or fall due to possible silt deposition or removal.
- Ground water rise or fall due to possible excess or deficit water seepage.
- Ecological imbalance due to possible disturbance of flora and fauna habitat.
- Desertification due to prevention of natural flooding (i.e. by diversion of flood water)
- Transfer of dissolved salts, suspended sediments, nutrients, trace elements etc. from one basin to another.

1.2.8 Tasks for planning a water resources project

The important tasks for preparing a planning report of a water resources project would include the following:

- Analysis of basic data like maps, remote sensing images, geological data, hydrologic data, and requirement of water use data, etc.
- Selection of alternative sites based on economic aspects generally, but keeping in mind environmental degradation aspects.
- Studies for dam, reservoir, diversion structure, conveyance structure, etc.
    - Selection of capacity.
    - Selection of type of dam and spillway.
    - Layout of structures.
    - Analysis of foundation of structures.
    - Development of construction plan.
    - Cost estimates of structures, foundation strengthening measures, etc.
- Studies for local protective works – levees, riverbank revetment, etc.
- Formulation of optimal combination of structural and non-structural components (for projects with flood control component).
- Economic and financial analyses, taking into account environmental degradation, if any, as a cost.
- Environmental and sociological impact assessment.

Of the tasks mentioned above, the first five shall be dealt with in detail in this course. However, we may mention briefly the last two before closing this chapter.

1.2.9 Engineering economy in water resources planning

All Water Resources projects have to be cost evaluated. This is an essential part of planning. Since, generally, such projects would be funded by the respective State Governments, in which the project would be coming up it would be helpful for the State planners to collect the desired amount of money, like by issuing bonds to the public, taking loans from a bank, etc. Since a project involves money, it is essential that the minimum amount is spent, under the given constraints of project construction. Hence, a few feasible alternatives for a project are usually worked out. For example, a project involving a storage dam has to be located on a map of the river valley at more than one possible location, if the terrain permits. In this instance, the dam would generally be located at the narrowest part of the river valley to reduce cost of dam construction, but also a couple of more alternatives would be selected since there would be other features of a dam whose cost would dictate the total cost of the project. For example, the foundation could be weak for the first alternative and consequently require costly found treatment, raising thereby the total project cost. At times, a economically lucrative project site may be causing submergence of a costly property, say an industry, whose relocation cost would offset the benefit of the alternative. On the other hand, the beneficial returns may also vary. For example, the volume of water stored behind a dam for one alternative of layout may not be the same as that behind another. Hence, what is required is to evaluate the so-called Benefit-Cost Ratio defined as below:

$$Benefit - CostRatio = \frac{AnnualBenefits(B)}{AnnualCost(C)}$$

The annual cost and benefits are worked out as under.

Annual Cost (C): The investment for a project is done in the initial years during construction and then on operation and maintenance during the project's lifetime. The initial cost may be met by certain sources like borrowing, etc. but has to be repaid over a certain number of years, usually with an interest, to the lender. This is called the Annual Recovery Cost, which, together with the yearly maintenance cost would give the total Annual Costs. It must be noted that there are many non-tangible costs, which arise due to the effect of the project on the environment that has to be quantified properly and included in the annual costs.

1.2.10 Assessment of effect on environment and society

This is a very important issue and all projects need to have clearance from the Ministry of Environment and Forests on aspects of impact that the project is likely to have on the environment as well as on the social fabric. Some of the adverse (negative) impacts, for which steps have to be taken, are as follows:

- Loss of flora and fauna due to submergence.
- Loss of land having agricultural, residential, industrial, religious, archaeological importance.
- Rehabilitation of displaced persons.
- Reservoir induced seismicity.
- Ill-effect on riverine habitats of fish due to blockage of the free river passage

There would also be some beneficial (positive) impacts of the project, like improvement of public health due to availability of assured, clean and safe drinking water, assured agricultural production, etc. There could even be an improvement in the micro-climate of the region due to the presence of a water body.

Module 1

Principles of Water Resources Engineering

Lesson 3

National Policy For Water Resources Development

Instructional Objectives

On completion of this lesson, the student shall be able to:

1. Appreciate the policy envisaged by the nation to develop water resources within the country
2. Conventional and non-conventional methods in planning water resources projects
3. Priorities in terms of allocation of water for various purposes
4. Planning strategies and alternatives that should be considered while developing a particular project
5. Management strategies for excess and deficit water imbalances
6. Guidelines for projects to supply water for drinking and irrigation
7. Participatory approach to water management
8. Importance of monitoring and maintaining water quality of surface and ground water sources.
9. Research and development which areas of water resources engineering need active
10. Agencies responsible for implementing water resources projects in our country
11. Constitutional provision guiding water resource development in the county
12. Agencies responsible for monitoring the water wealth of the country and plan scientific development based on the National Policy on water

1.3.0 Introduction

Water, though commonly occurring in nature, is invaluable! It supports all forms of life in conjunction with air. However, the demand of water for human use has been steadily increasing over the past few decades due to increase in population. In contrast, the total reserve of water cannot increase. Hence each nation, and especially those with rapidly increasing population like India, has to think ahead for future such that there is equitable water for all in the years to come. This is rather difficult to achieve as the water wealth varies widely within a country with vast geographical expanse, like India. Moreover, many rivers originate in India and flow through other nations (Pakistan and Bangladesh) and the demands of water in those counties have to be honored before taking up a project on such a river. Similarly there are rivers which originate form other counties (Nepal, Bhutan and China) and flow through India.

All these constraints have led to the formulation of the national water policy which was drafted in 1987 keeping in mind national perspective on water resource planning, development and management. The policy has been revised in 2002, keeping in mind latest objectives. It is important to know the essentials of the national policy as it has significant bearing on the technology or engineering that would be applied in developing and managing water resources projects.

This section elucidates the broad guidelines laid own in the National Water Policy (2002) which should be kept in mind while planning any water resource project in our country.

1.3.1 Water Resources Planning

Water resources development and management will have to be planned for a hydrological unit such as a drainage basin as a whole or a sub-basin. Apart from traditional methods, non-conventional methods for utilization of water should be considered, like

- Inter-basin transfer
- Artificial recharge of ground water
- Desalination of brackish sea water
- Roof-top rain water harvesting

The above options are described below in some detail:

Inter-basin transfer: Basically, it's the movement of surface water from one river basin into another. The actual transfer is the amount of water not returned to its source basin. The most typical situation occurs when a water system has an intake and wastewater discharge in different basins. But other situations also cause transfers. One is where a system's service area covers more than one basin. Any water used up or consumed in a portion of the service area outside of the source basin would be considered part of a transfer (e.g. watering your yard). Transfers can also occur between interconnected systems, where a system in one basin purchases water from a system in another basin.

Artificial recharge of ground water: Artificial recharge provides ground water users an opportunity to increase the amount of water available during periods of high demand--typically summer months. Past interest in artificial recharge has focused on aquifers that have declined because of heavy use and from which existing users have been unable to obtain sufficient water to satisfy their needs.

Desalination of brackish sea water: Water seems to be a superabundant natural resource on the planet earth. However, only 0.3 per cent of the world's total amount of water can be used as clean drinking water. Man requires huge amounts of drinking water every day and extracts it from nature for innumerable purposes. As natural fresh water resources are limited, sea water plays an important part as a source for drinking water as well. In order to use this water, it has to be desalinated. Reverse osmosis and electro dialysis is the preferred methods for desalination of brackish sea water.

Roof-top rain water harvesting: In urban areas, the roof top rain water can be conserved and used for recharge of ground water. This approach requires connecting the outlets pipe from roof top to divert the water to either existing well/tube wells/bore wells or specially designed wells/ structures. The Urban housing complexes or institutional buildings have large roof area and can be utilized for harvesting the roof top rain water to recharge aquifer in urban areas.

One important concept useful in water resources planning is ***Conjunctive*** or combined use of both surface and ground water for a region has to be planned for sustainable development incorporating quantity and quality aspects as well as environmental considerations. Since there would be many factors influencing the decision of projects involving conjunctive use of surface and ground water, keeping in mind the underlying constraints, the entire system dynamics should be studied to as detail as practically possible. The uncertainties of rainfall, the primary source of water, and its variability in space and time has to be borne in mind while deciding upon the planning alternatives.

It is also important to pursue watershed management through the following methodologies:

- Soil conservation
  This includes a variety of methods used to reduce soil erosion, to prevent depletion of soil nutrients and soil moisture, and to enrich the nutrient status of a soil.

- Catchment area treatment
  Different methods like protection for degradation and treating the degraded areas of the catchment areas, forestation of catchment area.

- Construction of check-dams
  Check-dams are small barriers built across the direction of water flow on shallow rivers and streams for the purpose of water harvesting. The small dams retain excess water flow during monsoon rains in a small catchment area behind the structure. Pressure created in the catchment area helps force the impounded water into the ground. The major environmental benefit is the replenishment of nearby groundwater reserves and wells. The water entrapped by the dam, surface and subsurface, is primarily

intended for use in irrigation during the monsoon and later during the dry season, but can also be used for livestock and domestic needs.

1.3.2 Water allocation priorities

While planning and operation of water resource systems, water allocation priorities should be broadly as follows:

- Drinking water
- Irrigation
- Hydropower
- Ecology
- Industrial demand of water
- Navigation

The above demands of water to various sectors are explained in the following paragraphs.

Drinking water: Adequate safe drinking water facilities should be provided to the entire population both in urban and in rural areas. Irrigation and multipurpose projects should invariably include a drinking water component, wherever there is no alternative source of drinking water. Drinking water needs of human beings and animals should be the first charge on any available water.

Irrigation: Irrigation is the application of water to soil to assist in the production of crops. Irrigation water is supplied to supplement the water available from rainfall and ground water. In many areas of the world, the amount and timing of the rainfall are not adequate to meet the moisture requirements of crops. The pressure for survival and the need for additional food supplies are causing the rapid expansion of irrigation throughout the world.

Hydropower: Hydropower is a clean, renewable and reliable energy source that serves national environmental and energy policy objectives. Hydropower converts kinetic energy from falling water into electricity without consuming more water than is produced by nature.

Ecology: The study of the factors that influence the distribution and abundance of species.

Industrial demand of water: Industrial water consumption consists of a wide range of uses, including product-processing and small-scale equipment cooling, sanitation, and air conditioning. The presence of industries in or near the city has great impact on water demand. The quantity of water required depends on the type of the industry. For a city with moderate factories, a provision of 20 to 25 percent of per capita consumption may be made for this purpose.

Navigation: Navigation is the type of transportation of men and goods from one place to another place by means of water. The development of inland water transport or navigation is of crucial importance from the point of energy conservation as well.

1.3.3 Planning strategies for a particular project

Water resource development projects should be planned and developed (as far as possible) as ***multi-purpose projects*** . The study of likely impact of a project during construction and later on human lives, settlements, socio-economic, environment, etc., has to be carried out before hand. Planning of projects in the hilly areas should take into account the need to provide assured drinking water, possibilities of hydropower development and irrigation in such areas considering the physical features and constraints of the basin such as steep slopes, rapid runoff and possibility of soil erosion.

As for ground water development there should be a periodical reassessment of the ground water potential on a scientific basis, taking into consideration the quality of the water available and economic viability of its extraction. Exploitation of ground water resources should be so regulated as not to exceed the recharging possibilities, as also to ensure social equity. This engineering aspect of ground water development has been dealt with in Lesson 8.1.

Planning at river basin level requires considering a complex large set of components and their interrelationship. Mathematical modelling has become a widely used tool to handle such complexities for which simulations and optimization techniques are employed. One of the public domain software programs available for carrying out such tasks is provided by the United States Geological Survey at the following web-site http://water.usgs.gov/software/. The software packages in the web-site are arranged in the following categories:

- Ground Water
- Surface Water
- Geochemical
- General Use
- Statistics & Graphics

There are private companies who develop and sell software packages. Amongst these, the DHI of Denmark and Delft Hydraulics of Netherlands provide comprehensive packages for many water resources applications.

Note:
Multi-purpose projects: Many hydraulic projects can serve more than one of the basic purposes-water supply, irrigation, hydroelectric power, navigation, flood control, recreation, sanitation and wild life conservation. Multiple use of project of facilities may increase benefits without a proportional increase in costs and thus enhance the economic justification for the project. A project which is which is designed for single purpose but which produces incidental benefits for other purposes should not, however, be considered a multi-purpose project. Only those projects which are designed and operated to serve two or more purposes should be described as multi-purpose.

1.3.4 Guidelines for drinking and irrigation water projects

The general guidelines for water usage in different sectors are given below:

1.3.4.1 *Drinking water*

Adequate safe drinking water facilities should be provided to the entire population both in urban and rural areas. Irrigation and multi purpose projects should invariably include a drinking water component wherever there is no alternative source of drinking water.

Primarily, the water stored in a reservoir has to be extracted using a suitable pumping unit and then conveyed to a water treatment plant where the physical and chemical impurities are removed to the extent of human tolerance. The purified water is then pumped again to the demand area, that is, the urban or rural habitation clusters. The source of water, however, could as well be from ground water or directly from the river.

The aspect of water withdrawal for drinking and its subsequent purification and distribution to households is dealt with under the course Water and Waste Water Engineering. The following books may be useful to consult.

- Waster Water Engineering by B C Punmia and A K jain
- Water and waste water engineering by S P Garg

1.3.4.2 *Irrigation*

Irrigation planning either in an individual project or in a basin as whole should take into account the irrigability of land, cost of effective irrigation options possible from all available sources of water and appropriate irrigation techniques for optimizing water use efficiency. ***Irrigation intensity*** should be such as to extend the benefits of irrigation to as large as number of farm families as possible, keeping in view the need to maximize production.

- ***Water allocation*** in an irrigation system should be done with due regard to equity and social justice. Disparities in the availability of water between head-reach and tail-end farms and (in respect of canal irrigation) between large and small farms should be obviated by adoption of a ***rotational water distribution system*** and supply of water on a ***volumetric basis*** subject to certain ceilings and rational water pricing.

- Concerned efforts should be made to ensure that the ***irrigation potential*** created is fully utilized. For this purpose, the ***command area development*** approach should be adopted in all irrigation projects.

- Irrigation being the largest consumer of freshwater, the aim should be to get optimal productivity per unit of water. Scientific water management, farm practices and ***sprinkler*** and ***drip*** system of irrigation should be adopted wherever possible.

The engineering aspects of irrigation engineering have been discussed in Section 6.

Some terms defined in the above passages are explained below:

Water allocation: Research on institutional arrangements for water allocation covers three major types of water allocation: public allocation, user-based allocation, and market allocation. This work includes attention to water rights and to the organizations involved in water allocation and management, as well as a comparative study of the consequences of water reallocation from irrigation to other sectors. A key aspect of this research is the identification of different stakeholders' interests, and the consequences of alternative institutions for the livelihoods of the poor.

Rotational water distribution system: Water allocated to the forms one after the other in a repeated manner.

Volumetric basis: Water allocated to each farm a specified volume based on the area of the farm, type of crop etc.

Irrigation Potential: Irrigation is the process by which water is diverted from a river or pumped from a well and used for the purpose of agricultural production. Areas under irrigation thus include areas equipped for full and partial control irrigation, spate irrigation areas, equipped wetland and inland valley bottoms, irrespective of their size or management type. It does not consider techniques related to on-farm water conservation like water harvesting. The area which can potentially be irrigated depends on the physical resources 'soil' and 'water', combined with the irrigation water requirements as determined by the cropping patterns and climate. However, environmental and socioeconomic constraints also have to be taken into consideration in order to guarantee a sustainable use of the available physical resources. This means that in most cases the possibilities for irrigation development would be less than the physical irrigation potential.

Command area development: The command area development programme aims mainly at reducing the gap between the potential created for irrigation to achieve higher agriculture production thereof. This is to be achieved through the integrated development of irrigated tracks to ensure efficient soil land use and water management for ensuring planned increased productivity.

Sprinkler irrigation: Sprinkler irrigation offers a means of irrigating areas which are so irregular that they prevent use of any surface irrigation methods. By using a low supply rate, deep percolation or surface runoff and erosion can be minimized. Offsetting these advantages is the relatively high cost of the sprinkling equipment and the permanent installations necessary to supply water to the sprinkler lines. Very low delivery rates may also result in fairly high evaporation from the spray and the wetted vegetation. It is impossible to get completely uniform distribution of water around a sprinkler head and spacing of the heads must be planned to overlap spray areas so that distribution is essentially uniform.

Drip: The drip method of irrigation, also called trickle irrigation, originally developed in Israel, is becoming popular in areas having water scarcity and salt problems. The method is one of the most recent developments in irrigation. It involves slow and frequent application of water to the plant root zone and enables the application of water and fertilizer at optimum rates to the root system. It minimizes the loss of water by deep percolation below the root zone or by evaporation from the soil surface. Drip irrigation is not only economical in water use but also gives higher yields with poor quality water.

1.3.5 Participatory approach to water resource management

Management of water resources for diverse uses should incorporate a participatory approach; by involving not only the various government agencies but also the users and other stakeholders in various aspects of planning, design, development and management of the water resources schemes. Even private sector participation should be encouraged, wherever feasible.

In fact, private participation has grown rapidly in many sectors in the recent years due to government encouragement. The concept of “Build-Own-Transfer (BOT)” has been popularized and shown promising results. The same concept may be actively propagated in water resources sector too. For example, in water scarce regions, recycling of waste water or desalinization of brackish water, which are more capital intensive (due to costly technological input), may be handed over to private entrepreneurs on BOT basis.

1.3.6 Water quality

The following points should be kept in mind regarding the quality of water:

1. Both surface water and ground water should be regularly monitored for quality.
2. Effluents should be treated to acceptable levels and standards before discharging them into natural steams.
3. Minimum flow should be ensured in the perennial streams for maintaining ecology and social considerations.

Since each of these aspects form an important segment of water resources engineering, this has been dealt separately in course under water and waste water engineering.

The technical aspects of water quality monitoring and remediation are dealt with in the course of Water and Waste – Water Engineering. Knowledge of it is essential for the water resources engineer to know the issues involved since, even polluted water returns to global or national water content.

Monitoring of surface and ground water quality is routinely done by the Central and State Pollution Control Boards. Normally the physical, chemical and biological parameters are checked which gives an indication towards the acceptability of the water for drinking or irrigation. Unacceptable pollutants may require remediation, provided it is cost effective. Else, a separate source may have to be investigated. Even industrial water also require a standard to be met, for example, in order to avoid scale formation within boilers in thermal power projects hard water sources are avoided.

The requirement of effluent treatment lies with the users of water and they should ensure that the waste water discharged back to the natural streams should be within acceptable limits. It must be remembered that the same river may act as source of drinking water for the inhabitants located down the river. The following case study may provoke some soul searching in terms of the peoples' responsibility towards preserving the quality of water, in our country:

Under the Ganga Action Plan (GAP) initiated by the government to clean the heavily polluted river, number of Sewage Treatment Plants (STPs) have been constructed all along the river Ganga. The government is also laying the main sewer lines within towns that discharge effluents into the river. It is up to the individual house holders to connect their residence sewer lines up to the trunk sewer, at some places with government subsidy. However, public apathy in many places has resulted in only a fraction of the houses being connected to the trunk sewer line which has resulted in the STPs being run much below their capacity.

Lastly, it must be appreciated that a minimum flow in the rivers and streams, even during the low rainfall periods is essential to maintain the ecology of the river and its surrounding as well as the demands of the inhabitants located on the downstream. It is a fact that excessive and indiscriminate withdrawal of water has been the cause of drying up of many hill streams, as for example, in the Mussourie area. It is essential that the decision makers on water usage should ensure that the present usage should not be at the cost of a future sacrifice. Hence, the policy should be towards a sustainable water resource development.

1.3.7 Management strategies for excess and deficit water imbalances

Water is essential for life. However, if it is present in excess or deficit quantities than that required for normal life sustenance, it may cause either ***flood*** or ***drought***. This section deals with some issues related to the above imbalance of water, and strategies to mitigate consequential implications. Much detailed discussions is presented in Lesson 6.2.

1.3.7.1 Flood control and management

- There should be a master plan for flood control and management for each flood prone basin.
- Adequate ***flood-cushioning*** should be provided in water storage projects, wherever feasible, to facilitate better flood management.
- While physical flood protection works like ***embankments and dykes*** will continue to be necessary, increased emphasis should be laid on non-structural measures such as ***flood forecasting and warning***, ***flood plain zoning***, and ***flood proofing*** for minimization of losses and to reduce the recurring expenditure on flood relief.

1.3.7.2 Drought prone area development

- Drought-prone areas should be made less vulnerable to drought associated problems through ***soil conservation measures***, ***water harvesting practices***, ***minimization of evaporation losses***, and ***development of ground water potential*** including ***recharging*** and ***transfer of surface water*** from surplus areas where feasible and appropriate.

Terms referred to above are explained below:

Flood cushioning: The reservoirs created behind dams may be emptied to some extent, depending on the forecast of impending flood, so that as and when the flood arrives, some of the water gets stored in the reservoir, thus reducing the severity of the flood.

Embankments and dykes: Embankments & dykes also known as levees are earthen banks constructed parallel to the course of river to confine it to a fixed course and limited cross-sectional width. The heights of levees will be higher than the design flood level with sufficient free board. The confinement of the river to a fixed path frees large tracts of land from inundation and consequent damage.

Flood forecast and warning: Forecasting of floods in advance enables a warning to be given to the people likely to be affected and further enables civil-defence measures to be organized. It thus forms a very important and relatively inexpensive nonstructural flood-control measure. However, it must be realized that a flood warning is meaningful if it is given sufficiently in advance. Also, erroneous warnings will cause the populace to loose faith in the system. Thus the dual requirements of reliability and advance notice are the essential ingredients of a flood-forecasting system.

Flood plain zoning: One of the best ways to prevent trouble is to avoid it and one of the best ways to avoid flood damage is to stay out of the flood plain of streams. One of the forms of the zoning is to control the type, construction and use of buildings within their limits by zoning ordinances. Similar ordinances might prescribe areas within which structures which would suffer from floods may not be built. An indirect form of zoning is the creation of parks along streams where frequent flooding makes other uses impracticable.

Flood proofing: In instances where only isolated units of high value are threatened by flooding, they may sometimes by individually flood proofed. An industrial plant comprising buildings, storage yards, roads, etc., may be protected by a ring levee or flood wall. Individual buildings sufficiently strong to resist the dynamic forces of the flood water are sometimes protected by building the lower stories (below the expected high-water mark) without windows and providing some means of watertight closure for the doors. Thus, even though the building may be surrounded by water, the property within it is protected from damage and many normal functions may be carried on.

Soil conservation measures: Soil conservation measures in the catchment when properly planned and effected lead to an all-round improvement in the catchment characteristics affecting abstractions. Increased infiltration, greater evapotranspiration and reduced soil erosion are some of its easily identifiable results. It is believed that while small and medium floods are reduced by soil conservation measures, the magnitude of extreme floods are unlikely to be affected by these measures.

Water harvesting practices: Technically speaking, water harvesting means capturing the rain where it falls, or capturing the run-off in one's own village or town. Experts suggest various ways of harvesting water:

- Capturing run-off from rooftops;
- Capturing run-off from local catchments;
- Capturing seasonal flood water from local streams; and
- Conserving water through watershed management.

Apart from increasing the availability of water, local water harvesting systems developed by local communities and households can reduce the pressure on the state to provide all the financial resources needed for water supply. Also, involving people will give them a sense of ownership and reduce the burden on government funds.

Minimization of evaporation losses: The rate of evaporation is dependent on the vapour pressures at the water surface and air above, air and water temperatures, wind speed, atmospheric pressure, quality of water, and size of the water body. Evaporation losses can be minimized by constructing deep reservoirs, growing tall trees on the windward side of the reservoir, plantation in the area adjoining the reservoir, removing weeds and water plants from the reservoir periphery and surface, releasing warm water and spraying chemicals or fatty acids over the water surface.

Development of groundwater potential: A precise quantitative inventory regarding the ground-water reserves is not available. Organization such as the Geographical Survey of India, the Central Ground-Water Board and the State Tube-Wells and the Ground-Water Boards are engaged in this task. It has been estimated by the Central Ground-Water Board that the total ground water reserves are on the order of 55,000,000 million cubic meters out of which 425,740 million cubic meters have been assessed as the annual recharge from rain and canal seepage. The Task Force on Ground-Water Reserves of the Planning Commission has also endorsed these estimates. All recharge to the ground-water is not available for withdrawal, since part of it is lost as sub-surface flow. After accounting from these losses, the gross available ground-water recharge is about 269,960 million cubic meters per annum. A part of this recharge (2,460 million cubic meters) is in the saline regions of the country and is unsuitable for use in agriculture owing to its poor quality. The net recharge available for ground-water development in India, therefore, is of the magnitude of about 267,500 million cubic meters per annum. The Working Group of the Planning Commission Task Force Ground-Water Reserves estimated that the usable ground-water potential would be only 75 to 80 per cent of the net ground-water recharge available and recommended a figure of 203,600 million cubic meters per annum as the long-term potential for ground-water development in India.

Recharging: Artificial recharge provides ground water users an opportunity to increase the amount of water available during periods of high demand--typically summer months. Past interest in artificial recharge has focused on aquifers that have declined because of heavy use and from which existing users have been unable to obtain sufficient water to satisfy their needs.

Transfer of surface water: Basically, it's the movement of surface water from one river basin into another. The actual transfer is the amount of water not returned to its source basin. The most typical situation occurs when a water system has an intake and wastewater discharge in different basins. But other situations also cause transfers. One is where a system's service area covers more than one basin. Any water used up or consumed in a portion of the service area outside of the source basin would be considered part of a transfer (e.g. watering your yard). Transfers can also occur between interconnected systems, where a system in one basin purchases water from a system in another basin.

1.3.8 Implementation of water resources projects

Water being a state subject, the state governments has primary responsibility for use and control of this resource. The administrative control and responsibility for development of water rests with the various state departments and corporations. Major and medium irrigation is handled by the irrigation / water resources departments. Minor irrigation is looked after partly by water resources department, minor irrigation corporations and zilla parishads / panchayats and by other departments such as agriculture. Urban water supply is generally the responsibility of public health departments and panchayatas take care of rural water supply. Government tube-wells are constructed and managed by the irrigation/water resources department or by the tube-well corporations set up for the purpose. Hydropower is the responsibility of the state electricity boards.

Due to the shared responsibilities, as mentioned above, for the development of water resources projects there have been instances of conflicting interests amongst various state holders.

1.3.9 Constitutional provisions for water resources development

India is a union of states. The Constitutional provisions in respect of allocation of responsibilities between the State and Center fall into three categories: the Union List (List-I), the State List (List-II) and the Concurrent List (List-III). Article 246 of the Constitution deals with subject matter of laws to be made by the Parliament and by Legislature of the States. As most of the rivers in the country are inter-State, the regulation and development of waters of these rivers is a source of inter-State differences and disputes. In the Constitution, water is a matter included in entry 17 of List-II i.e., State List. This entry is subject to provision of entry 56 of List-I i.e., Union List. The specific provisions in this regard are as under:

- Article 246
  Notwithstanding anything in clauses (2) and (3), Parliament has exclusive power to make laws with respect to any of the matters enumerated in List-I in the seventh schedule (in this Constitution referred to as the "Union List").
  1) Notwithstanding anything in clauses (3), Parliament, and, subject to clause (1), the Legislature of any State also, have power to make laws with respect to any of the matters enumerated in List-III in the seventh schedule (in this Constitution referred to as the "Concurrent List").
  2) Subject to clauses (1) and (2), the Legislature of any state has exclusive power to make laws for such state or any part thereof with respect to any of the matters enumerated in List-II in the seventh schedule (in this Constitution referred to as the "State List").
  3) Parliament has power to make laws with respect to any matter for any part of the territory of India not included in a State notwithstanding that such matter is a matter enumerated in the State List.

- Article 262
  In case of disputes relating to waters, article 262 provides:
  1) Parliament may by law provide for the adjudication of any dispute or complaint with respect to the use, distribution or control of the waters of, or in, any inter-State river or river-valley.
  2) Notwithstanding anything in this Constitution, Parliament may, by law provide that neither the Supreme Court nor any other Court shall exercise jurisdiction in respect of any such dispute or complaint as is referred to in clause (1).

- Entry 56 of list I of seventh schedule
  Entry 56 of List I of seventh schedule provides that "Regulation and development of inter-State rivers and river valleys to the extent to which

such regulation and development under the control of the Union are declared by Parliament by law to be expedient in the public interest".

- Entry 17 under list II of seventh schedule
  Entry 17 under List II of seventh schedule provides that "Water, that is to say, water supplies, irrigation and canals, drainage and embankments, water storage and water power subjects to the provisions of entry 56 of List I".
  As such, the Central Government is conferred with powers to regulate and develop inter-State rivers under entry 56 of List I of seventh schedule to the extent declared by the Parliament by law to be expedient in the public interest.
  It also has the power to make laws for the adjudication of any dispute relating to waters of Inter-State River or river valley under article 262 of the Constitution.

1.3.10 Central agencies in water resources sector

Some of the important offices working under the Ministry of Water Resources, Government of India (website of the ministry: http://wrmin.nic.in) which plays key role in assessing, planning and developing the water resources of the country are as follows:

- Central Water Commission (CWC)
- Central Ground Water Board (CGWB)
- National Water Development Agency (NWDA)
- Brahmaputra Board
- Central Water and Power Research Station (CWPRS)
- Central Soil and Materials Research Station (CSMRS)
- National Institute of Hydrology (NIH)
- Ganga Flood Control Commission (GFCC)
- Water and Power Consultancy Services (India) ltd (WAPCOS)
- National Projects Construction Corporation ltd (NPCC)

Detailed activities of the above departments may be obtained from the Ministry of Water Resources web-site.

Although not directly under the ministry of water resources, the National Hydropower Corporation (NHPC) as well as Rail India Technical Engineers Services (RITES) also actively participate in water resources development projects.

Module 1

Principles of Water Resources Engineering

Lesson 4

Planning and Assessment of Data for Project Formulation

Instructional Objectives

On completion of this lesson, the student shall be able to learn:

1. The range of water resources project and the general planning philosophy
2. Planning arrangements for drinking water supply project and related data requirement
3. Planning arrangements for irrigation water supply project and related data requirement
4. Planning arrangements for hydropower generation project and related data requirement
5. Planning arrangements for flood control project and related data requirement
6. Investigations for data assessment for constructing water resource engineering structures
7. Water availability computations
8. Data collection for environment, socio-economic and demographic informations
9. Data collection methods for topography, geology, rainfall and stream flow.

1.4.0 Introduction

A water resources systems planner is faced with the challenge of conceptualizing a project to meet the specific needs at a minimum cost. For a demand intensive project, the size of the project is limited by the availability of water. The planner then has to choose amongst the alternatives and determine the optimum scale of the project. If it is a multi-purpose project, an allocation of costs has to be made to those who benefit from the project. An important aspect of planning is that it has to prepare for a future date – its effects in terms of physical quantities and costs over a period of time spanning the useful life of project has to be evaluated. The return expected over the project period has to be calculated.

All this requires broader decisions, which affect the design details of the project. This chapter looks into the different aspects of preparing a project plan likely to face a water resources system planner, including the basic assessment of data that is primary to any project plan formulation.

1.4.1 Meeting the challenges

The major projects which water resources systems planner has to conceptualize are shown in Figure 1. Although the figure shows each project to be separate entity, quite a few real projects may actually serve more than one purpose. For example, the Hirakud or the Bhakra dams cater to flood control, irrigation and hydropower generation. On the other hand more than one project is necessary (and which actually forms a system of projects) to achieve a specific purpose.

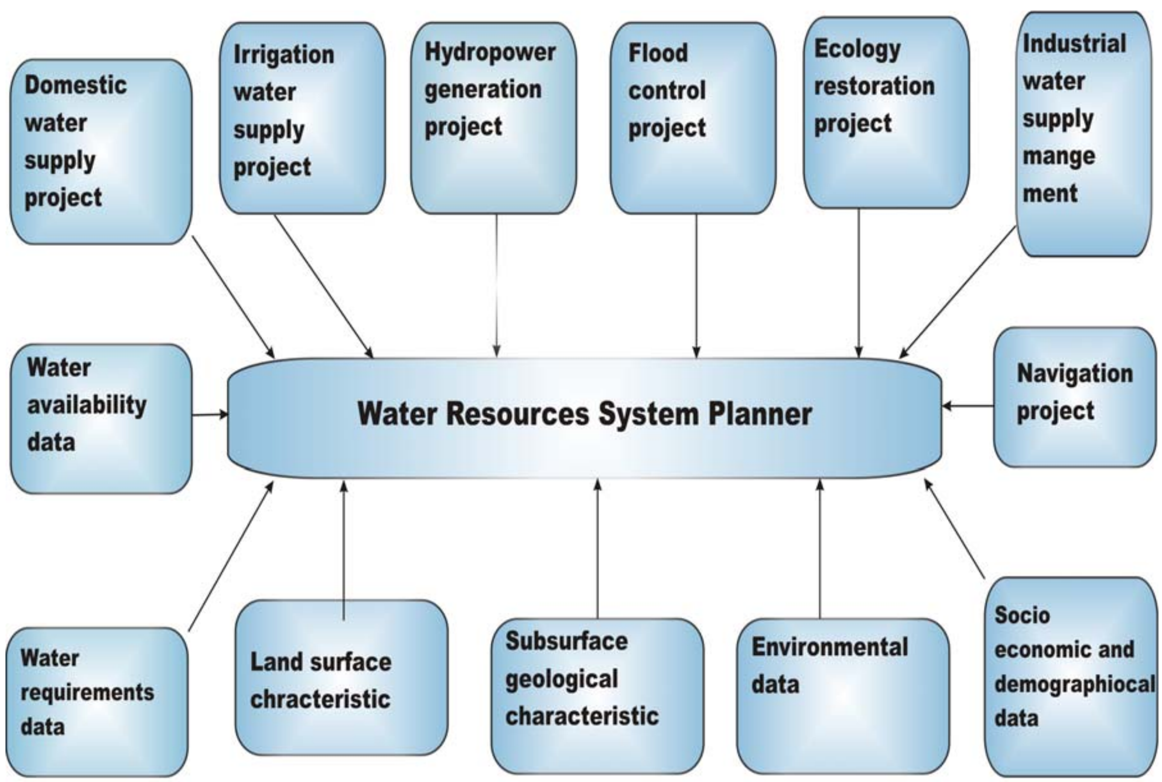


Figure 1 Possible water resources projects requiring planning and necessary data requirement

For example, to control the floods in the Damodar River, which earlier used to havoc in the districts of Bardhaman, Hooghly and Howrah in West Bengal, a number of dams were constructed on the Damodar and its tributaries between 1950s and 1970. For irrigation projects, a dam may be constructed across a river to store water in the upstream reach and a barrage may be constructed in the downstream reach to divert and regulate the water through an off taking canal.

1.4.2 Project planning for domestic water supply

The project for supplying drinking water to a township would usually consist of a network of pipelines to reach the demand area. The source of water could be underground or from a surface water body, usually a river. At times, it could be a judicious combination of the two. A water resources systems planner has to design the whole system from the source up to the distribution network. However, the scope of water resources engineering is generally be limited to the intake system design. The storage of water, its treatment and finally distribution to the consumers are looked after by the authorities of the township. Further details may be obtained in a course on Water and Waste Water Engineering.

Typical intake systems could possibly be one of the following, depending and the convenience of planning.

1. Construction of a water intake plant directly from the river

   Example: Water intake system at Palta for Kolkata from river Hooghly.

2. Construction of a dam across a river and drawing water from the reservoir behind.

   Example: Dam at Mawphlang on river Umiam for water supply to Shillong.

3. Construction of a barrage across a river and drawing water from the pool behind

   Example: Wazirabad barrage across river Yamuna for water supply to Delhi.

4. Construction of infiltration wells near a river to draw riverbed ground water

   Example: For water supply to IIT Kharagpur campus from river Kangsabati.

5. Construction of deep wells to draw water from lower strata of ground water

   Example: Water intake system for the city of Barddhaman.

A simple line sketch is shown in Figure 2 to show the processes for intake, storage, treatment and distribution of a typical drinking water project.

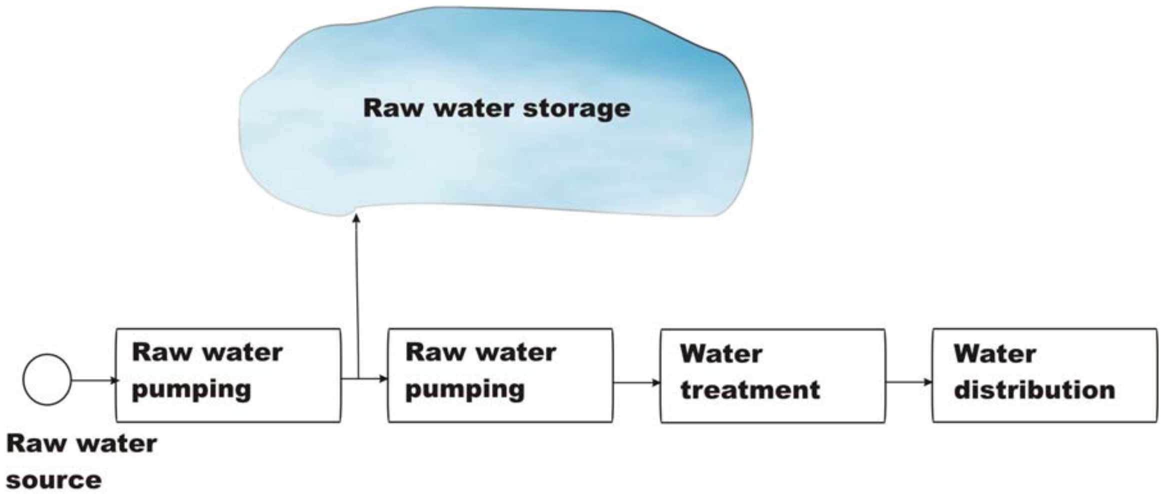


Figure . 2 A line diagram for intake, storage, treatment and distribution of a typical drinking water project

1.4.3 Data requirement for domestic water supply project

The following data is required for planning and designing a typical water supply system.

1.4.3.1 *Demand of water*

As discussed in lesson 1.2, according to the norms laid out in the National Building Code, and revised under National Water policy (2002), the following demand of domestic water consumption may be adopted:

Rural water supply:

- 40 litres per capita per day or one hand pump 250 persons within walking distance of 1.6 km or elevation difference of 100m in hills
- 30 lpcd additional for cattle in desert development programmed areas

Urban water supply:

- 40 lpcd where only sources are available
- 70 lpcd where piped water supply is available but no sewerage system
- 125 lpcd where piped water supply and sewerage system are both available.
- 150 lpcd for main cities

- Additional water for other demands like commercial, institutional, firefighting, gardening, etc.

Since the water supply project would serve a future population, a realistic projection has to be made based on scientific projection methods like

- ***Arithmetic increase method***.
- ***Geometric increase method***.
- ***Incremental increase method***.

Water supply projects, under normal circumstances, may be designed for a period of thirty years. This period may be modified in regard to certain components of the project, depending upon:

- The useful life of the component facility
- Ease in carrying out extensions, when required.
- Rate of interest.

1.4.3.2 Availability of water and other data

The availability of water has been discussed in a subsequent section of this lesson, which would be used to design the capacities of the intake by the water resources engineer, by comparing with the demand. The data for constructing the structures would usually be topography for locating the structure, geology for finding foundation characteristics and materials required for construction of the structure.

1.4.4 Project planning for irrigation water supply

The project may consist of supplying water to irrigate an area through a network of canals, by diverting some of the water from a river by constructing a barrage for water diversion and head regulator for water control. The water through canals mostly flows by gravity (except for pumped canal projects), the area under cultivation by the water of the canal is called the Command Area. This area is decided by the prevailing slope of the land. Although the main source of water for irrigating an area could be surface water, it could be supplemented with ground water. This combination of surface and ground water for irrigation is known as Conjunctive use.

The principal component of an irrigation scheme is a diversion structure – a weir or a barrage – though the latter is preferred in a modern irrigation project. Since the height of such a structure is rather small compared to that of a dam, the volume of water stored behind a barrage (the barrage pool) is small compared to that stored behind a dam (the dam reservoir). The elevated water surface of the barrage pool causes the water to be diverted into the canal, the entry of which is regulated through a ***canal head works***. If the river is perennial, and the minimum flow of the river is sufficient to cater to the flow through the canal, this arrangement is perfectly fine to irrigate a command area using a barrage and an irrigation canal system. However, if the river is non-perennial, or the minimum flow of the river is less than the canal water demand, then a dam may be constructed at a suitable upstream location of the river. This would be useful in storing larger volumes, especially the flood water, of water which may be released gradually during the low-flow months of the river.

A conceptual scheme of a diversion scheme for irrigation is shown in Figure 3.

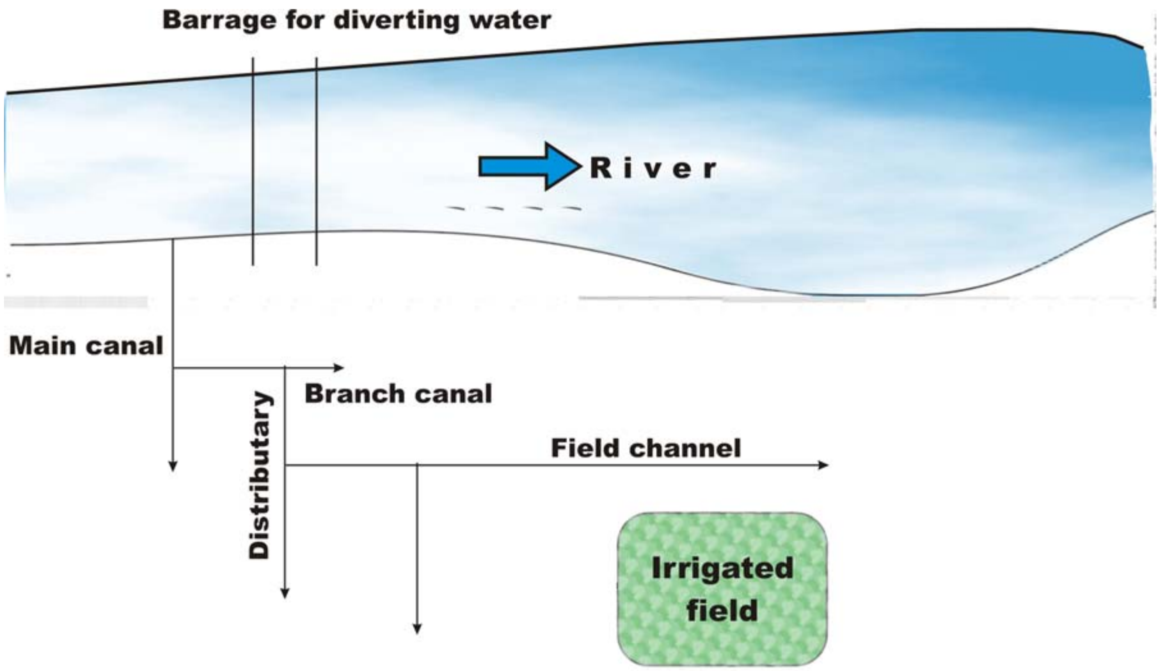


Figure 3 Diversion scheme for irrigation

1.4.5 Data requirement for water supply to an irrigation project

The following data is required for planning and designing a typical irrigation system.

1.4.5.1 *Demand of water for irrigation water supply*

The demand of water for an irrigation scheme is to be calculated from the cropping schedule that is proposed in the Command Area. Different crops have different water requirements and their demand also varies with the growth of the plants. Further, Command Area may be able to cultivate more than one crop within since many of the crops have maturity duration of few months.

The field requirement decides the design discharge for the distributaries and so on up to the canal regulator. Of course, most canals are prone to losses with water seeping through the canal sides. Exceptions are the lined canals, though in this case, the loss of infiltrating water is very small. Thus the net demand at the head of the canal system, as a function of time, is calculated. Lessons of Module 3 deal in detail about the irrigation system demand of water.

1.4.5.2 Availability of water and other data

This has discussed in a subsequent section of this lesson. The data for demand and availability of water would be used to design the reservoir upstream of the dam for storage. This water, when released in a regulated way, would be diverted by a barrage and passed through a canal head regulator and water distribution network consisting of canals and other structures such as ***regulators*** and ***falls***. The data requirements for construction of the structures are usually: Topography, geology or riverbed soil characteristics, and materials.

1.4.6 Project planning for hydropower generation

A hydroelectricity generation project or a hydropower project in short, would essentially require water diversion form a continuous surface water source like a river. The diversion, as shown, could be using a dam or a barrage. A dam has the advantages of creating a high head and provides sufficient storage in the reservoir that is created behind. When the stream inflow to a reservoir is less, the stored water may be released to generate power.

A barrage, on the other hand, does not store much water in the pool. Hence, the power generation would be according to the available flow in the river. It also does not create a high head and hence this type of arrangement is usually practiced in the hilly areas, where a long power channel ensures sufficient head for power generation. This is because the slope of the power channel would be rather small compared to the general slope of land. A system with no sufficient storage is called the run-of-the-river project.

Figure 4 shows a typical schematic diagram for a project with a dam for diverting water to generate hydropower.

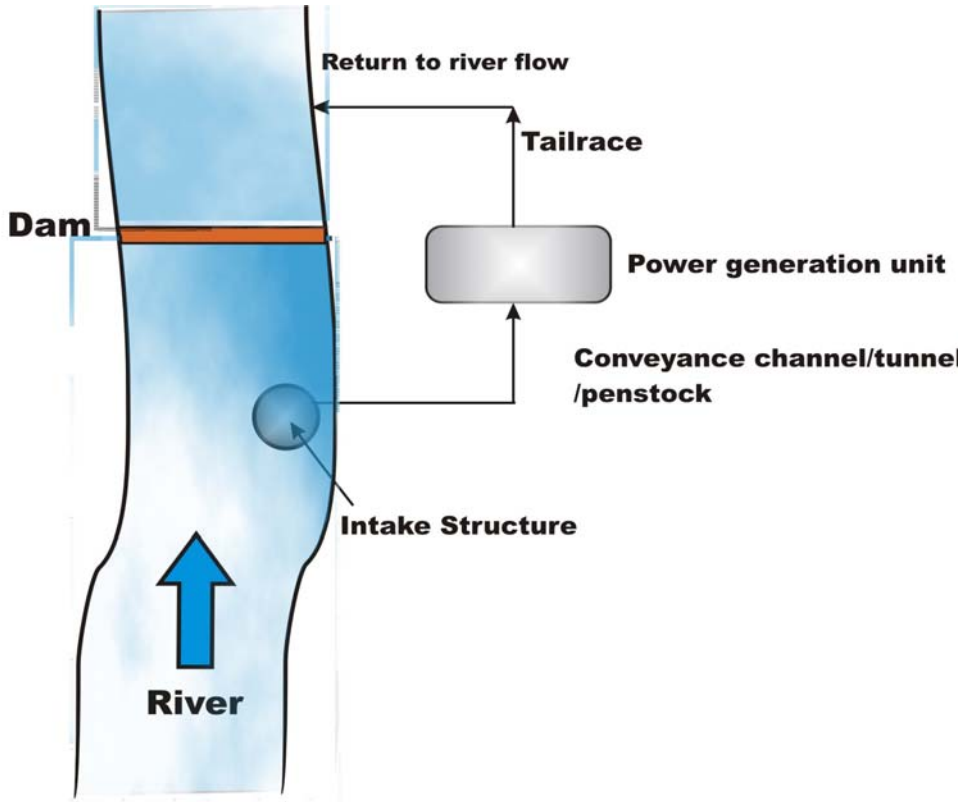


Figure 4 Diversion with a dam for power generation

Under some situation, a barrage may also be used to divert water through a power channel to generate hydropower. This is shown in Figure 5.

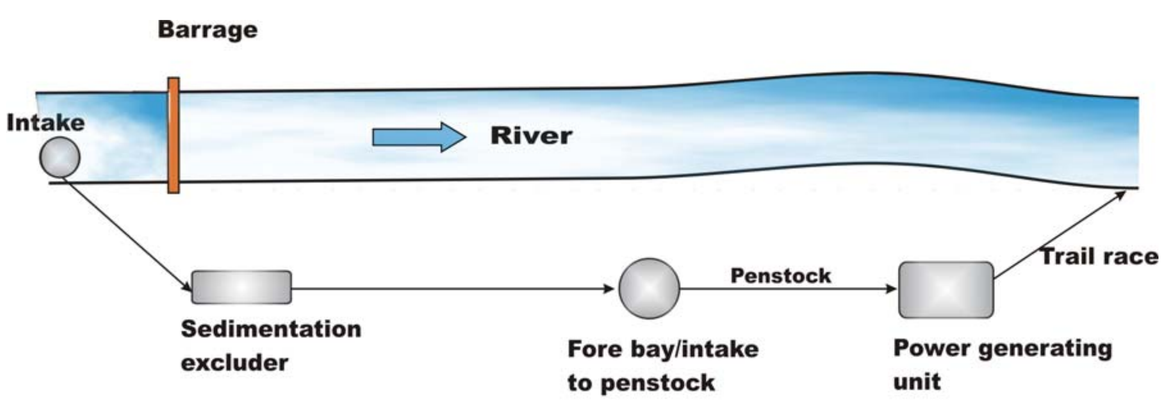


Figure 5 Diversion with a barrage for power generation

1.4.7 Data requirement for hydropower generation project

The following data is required for planning and designing a typical hydropower system.

1.4.7.1 *Demand of water*

Power generated 'P' is proportional to the discharge 'Q' passing through the turbine generator units and the piezometric head of water 'H'. Also, the demand of power varies with the time of the day (Figure 6) and some times on the days of the week. Hence the demand of water that is required to drive the turbines would vary too.

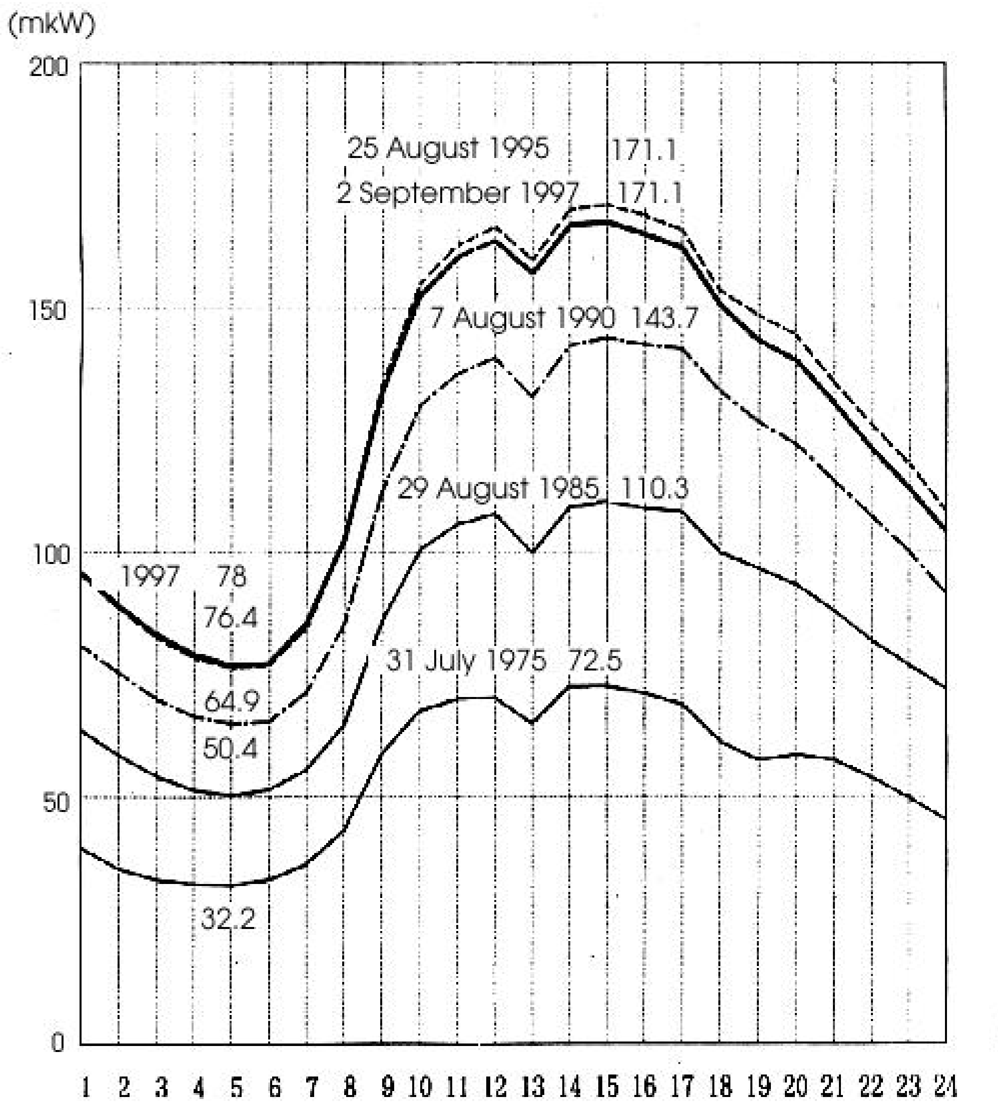


FIGURE 6 . Typical power demand curve variation over a month

However, when a hydropower plant is initially planned, the main constraint comes from the stream flow availability. Demand, on the other hand, is not really limited since more generation of power is always welcome. Hence, the maximum installed capacity of a hydropower plant would be limited to the reliable all-year-round available flow.

1.4.7.2 *Availability of water*

This has been discussed in a subsequent section of this lesson. The data for demand and availability would decide the height of dam or a barrage and the size of the appurtenant structures required for conveying water up to the power generation unit and the corresponding exit channel. The data requirement for construction of the structures is the same as mentioned before, that is, Topography, Geology and Materials.

1.4.8 Project planning for flood control

Truly speaking, controlling a flood is generally not possible, but with different combinations, it can be managed in such a way that the resulting damages are minimized. There are several options, but broadly, these may be classified as being structural or non-structural. Construction of a large dam across a river to hold the incoming flood and the release of the regulated flow would fall under structural measures. On the other hand, if the residents of the flood prone area are warned before hand by making suitable predictions of the impending flood using a flood forecasting technique, then it falls under a non-structural measure.

Lesson 6.2 deals with different types of flood management techniques, but presently, the discussion is limited to the construction of dams for management of floods, as illustrated in Figure 7.

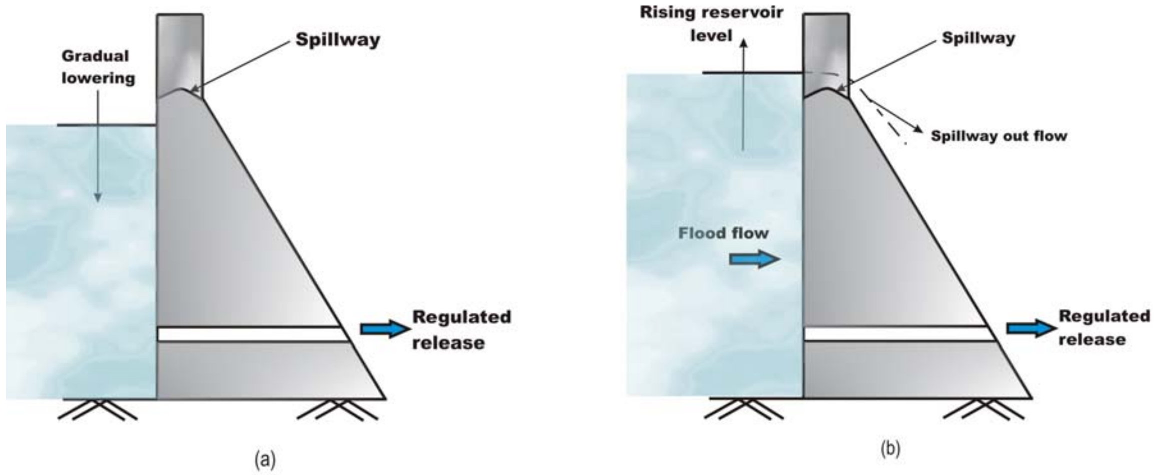


Figure 7 Section of dam a) Showing level of water before inflow of flood
b) Showing reservoir storage for flood inflow and regulated release

1.4.9 Data requirement for flood control project

Theoretically a dam constructed to reduce a flood peak should require the maximum possible stream flow ***hydrograph***.

However, this is neither possible to be determined exactly, nor is desirable as it would too costly to build a huge dam. Rather, a flood of a certain probability of occurrences (say 1 in 100 years) is estimated from past peak stream flow records and a corresponding hydrograph constructed. This is generally used to design the height of the dam (which determines the size of the reservoir) and the ***spillway***.

Hence, if a dam is used to moderate the flood of a river, then the data collection should be aimed at that required for constructing a dam. They usually concern topography, geology and materials. If other structural measures like embankment are constructed, then also the above mentioned parameters appropriate to the construction of embankment would be required to be collected.

1.4.10 Planning for other miscellaneous projects

Other major types of water resources project include those for

- Ecology restoration

- Industrial water supply
- Navigation

In each of the above, a certain demand of water is first estimated, for example.

- How much water is required for restoration of a marshy or aquatic habitat and how is it spread overtime.
- How much water is required to be supplied to an industry (relatively larger demands being required for the cooling of thermal energy producing plants).
- How much water is required to flush out sediment from a navigable channel or to what height the river water level should be raised to increase the draft necessary for moving vessels.

Accordingly, a dam or a barrage and possibly a water conveying canal would be required, to achieve the above objectives. The construction details for each of these components have been dealt with in the lessons of Module 4. We shall look now into the data requirement and its source.

1.4.11 Investigations for data assessment

The main structural components that are proposed for any water resource project include the following:

- A storage structure like Dams
- A diversion structure like Barrages
- A water conveyance structure like Canals

The primary job of the water resources engineer would be to locate or site the structure and for that the land surface elevation, or topography, is required. Once a structure is sited (or a few alternatives sited), then the next phase would be investigate the suitability of the foundation. Thus, geological characteristics determination forms an important data requirement.

For demand intensive projects, where the demand is more than the supply, the maximum possible flow that can be diverted for useful function is limited by the stream flow availability. Hence, the water availability studies form the third set of important data assessment

In the national level, the survey and investigation wing of the ***Central Water Commission (CWC)*** takes up these assessment jobs for surface water projects in concurrence with concerned state governments or central government. The CWC monitors most of the country's ***Major and Medium Projects*** and the detailed project reports (DPRs) have been prepared and submitted to concerned authorities.

1.4.12 Topographic details

These are the elevation contour maps the area where a project is proposed to be executed. The ***Survey of India*** has the responsibility to prepare and publish such maps for the nation. The maps (called ***toposheet***) in the scale of 1:50,000 have been completed for almost all regions the country. The contour interval in these maps is 20 meters and each sheet covers 15 minutes of latitude and longitude. Some areas have been surveyed in greater detail in the scale of 1:25000 in which the contour interval is 5 meters.

The survey of India also conducts specific surveys for particular project sites to serve the needs of project authorities. The scale and contour interval depends upon the nature of the terrain (country) and the purpose of the survey. The ***National Remote Sensing Agency*** has also acquired a ***Lidar*** for precision survey work with a topographic precision of 0.01m.

The elevation contour map of a region is useful to decide among others

- Height of storage structures (dam) and elevation of its spillway.
- Extent of inundation due to reservoir formation behind a dam.
- Amount of storage possible in the reservoir.
- Alignment of canals and their branches.

1.4.13 Geological characteristics

Usually hydraulic structures like dams or barrages for major water resources projects are massive. Unless the foundation properties are correctly found from geologic features and their interpretation, chances of structural failure would increase. Even for barrages, which are comparatively lighter structures, the underlying foundation strata of the river bed needs to be properly investigated. The ***Geological Survey of India*** has produced maps showing geological structure of the country. However, whenever a project is planned, a detailed geological investigation is carried out by drilling ***Bore Holes*** at required number of places and taking a ***Boring Log***. The ***Strength Parameters*** of the underlying rock/soil layers are investigated by extracting cores of samples and taken to laboratory for ***Strength Tests***. Sometimes, ***In-situ Laboratory Tests*** are conducted that avoids disturbing the foundation material in its original form.

The geological tests of the foundation material of the proposed project allow the determination of the following major parameters.

- Base width of a dam or a barrage so that the ***Bearing Pressure*** is within safe limits.
- Degree of protection required for prevention of seepage below the hydraulic structure. (grout holes for dams and sheet piles for barrages)

4.1.14 Water availability data

Lesson 1.1 gives details about the average water availability of the country in general for a specific project dependant on surface water sources, however more detailed data of the amount of water availability needs to be established. In fact, the success of a water resources project depends on how accurate has been the estimation of the total quantity of water available and its variation with time – over days, weeks, months and years. This would require collection of data and its analysis by suitable methods.

Project	Dependable water availability*
Irrigation	75%
Drinking water	100%
Hydro power project	90%

*n% dependable availability means that the minimum water required for the project would be available for 'n' units of time (say days or weeks, 10 day period, monthly) from within 100 equivalent units.

Database*:*
For computations of water availability, the following rainfall and stream runoff data should be collected in order of preference as given below. Daily observed data collected for ten consecutive days is more commonly used and mentioned here as ten-daily data

- Runoff data at the proposed site for at least 40 – 50 years.

Or

- Rainfall data of the catchment for 40-50 years and Runoff data for at least 5-10 years.

Or

- Rainfall data of the catchment for 40-50 years and Runoff data and concurrent rainfall data at existing project on upstream or down stream of the proposed site for at least 5-10 years.

Or

- Rainfall data of the catchment for 40-50 years and Runoff data concurrent rainfall data at existing project of a nearby river for at least 5 to 10 years provided ***Orographic*** conditions of the catchment at the work site are similar to that of the proposed site.

4.1.15 Water availability computations

Depending on the type of data available, the water availability can be computed from the following methods:

Direct observation method:
This method is applied when observed runoff data at the proposed site is available for the last 50 years or so. The method has been discussed in Lesson 2.4.

Rainfall-Runoff series method:
The method consists in extending the runoff data with the help of rainfall data by means of rainfall-runoff relationships (Lessons 2.2 and 2.3**)**.Depending upon the availability of rainfall and runoff data, following three cases arise

- Long term precipitation record along with a stream flow data for a few years is available.
- Long term precipitation record is available for the catchment along with a few years of stream flow data at a neighboring site on the same river.
- Long term precipitation record is available for the catchment rainfall-runoff data on a nearby river.

Langbein's log-deviation method:
This method is used when short term runoff data is available at the proposed site along with long term runoff at a nearby gauging station.

There are other methods which are discussed in the advanced texts, as the following:

Mutreja, K N (1995) *Applied Hydrology*, Tata McGraw Hill.

4.1.16 Environmental data

Any water resources project would be affecting the environment in one way or other. Construction of a dam or barrage may not allow free movement of fish along the river, the ponded water behind may cause submergence of valuable forest and even human habitation. Construction of flood protection environment may cause water logging in the area behind the embankment unless proper drainage is provided, thus leading to breeding of mosquitoes and other disease carrying vectors.

It is, therefore, always mandatory to check the impact on the environment due to construction of a water resource project. For this purpose, the relevant data on environment and ecology has to be collected for analysis.

4.1.17 Socio-economic and demographic data

Dam and barrage projects constructed at one point on a river benefits people downstream largely. However, the construction affects the people residing on the upstream as the ponded water causes submergence of villages and force people to migrate. It is pertinent, therefore, to study the effect of the project on the people and impact on the socio-economic fabric of the region benefited or affected by the project.

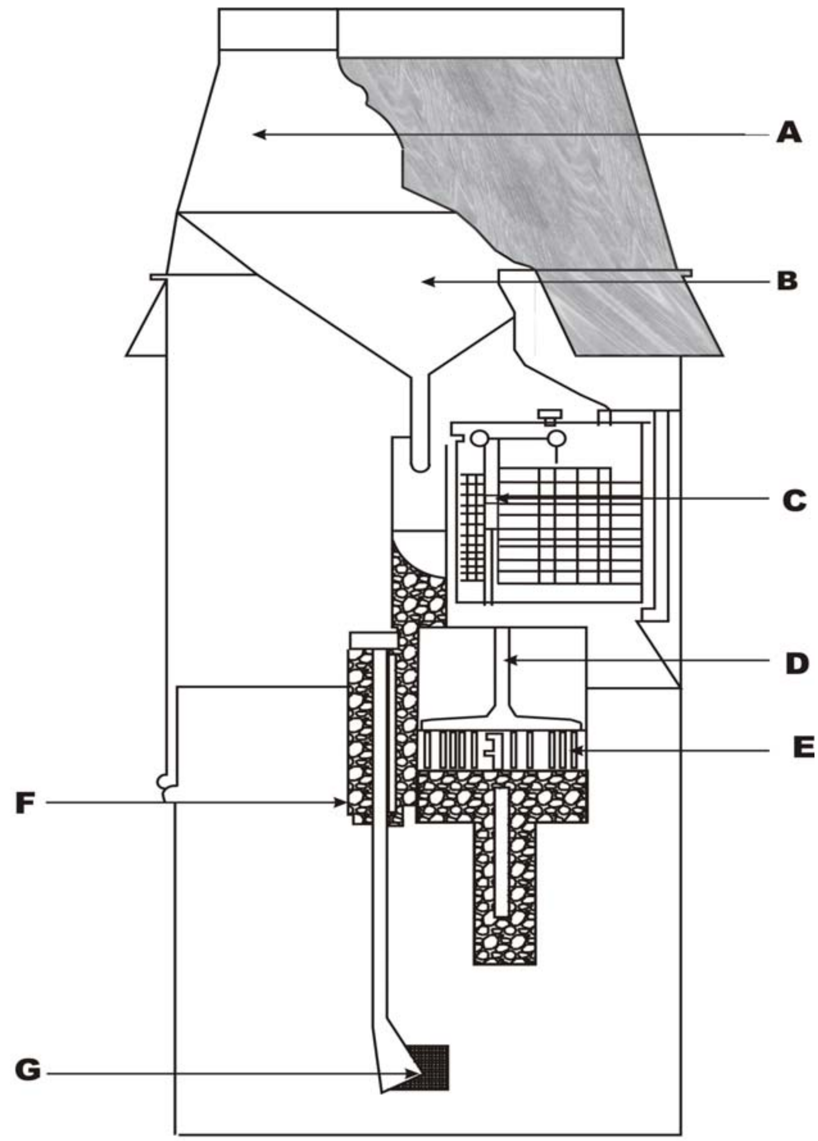
4.1.18 Data collection methods

Rainfall:
This is measured with rain gauges, which may be of ***Recording*** (Figure 8) ***or Non-Recording*** (Figure 9) types. The specifications regarding these gauges may be found in the following Indian Standard codes of practices:

- ***IS: 5225*** (1998) - Specifications for non-recording rain gauges.
- ***IS: 4986*** (2002) - Installation of non-recording rain gauges and rain measurement
- ***IS: 5235*** (1998) - Specifications for recording rain gauges.
- ***IS: 8389*** (2003) - Installation and use of recording rain gauges.

The rain gauges may be distributed within the catchment as specified in the following IS code:

- ***IS: 4987*** (1994) - Recommendations for establishing network of rain gauge stations



LEGEND

A . COLLECTOR
B . FUNNEL
C . RECORDING DRUM
D . FLOAT AND FLOAT ROD
E . FLOAT CHAMBER
F . SIPHON CHAMBER
G . DISCHARGE TUBE

NATURAL -SIPHON RAIN GAUGE

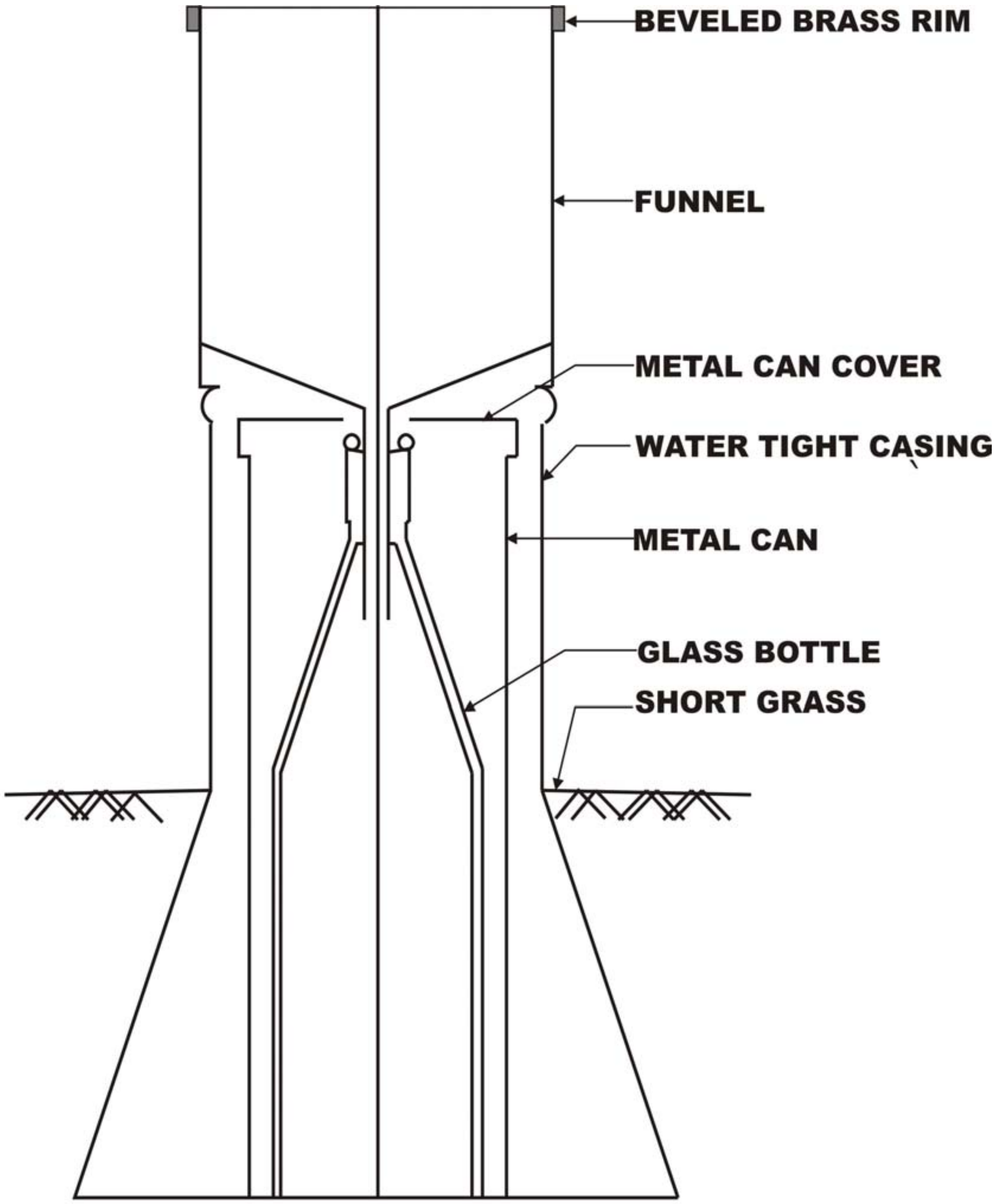


Figure 9. Non-recording rain gauge

Stream Runoff (Discharge):

The discharge of a stream or a river at a point varies with time. Usually, the discharge is measured by calculating the average velocity of the stream and multiplying by the cross sectional area. Since the velocity of a stream varies across the cross section, it is usual to divide the cross section hypothetically into several vertical strips (Figure 10).

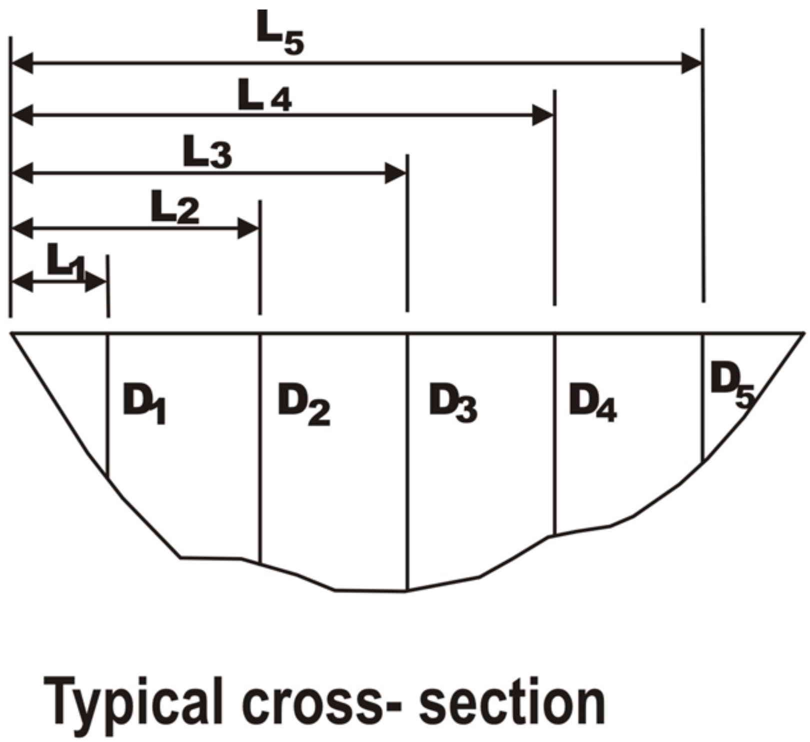


Figure 10. Division of river into strips

Calculation of the discharge passing through each strip is then done by multiplying the average velocity of the strip by the area of the strip (approximated as a trapezium). The velocity is measured with a ***current meter*** (Figures 11 and 12) which dipped in the flowing water to a distance of 0.6 times the depth of water at that point, since the velocity at this point is seen to represent the average velocity well for most streams. There are many different types of current meters, of which the "Price" cup-type current meter attached to a round wading rod is illustrated in Figure 8. Discussions on the principles of measurement of stream flow, including the types of current meters may be obtained from the United States Bureau of Reclamation online document "Water measurement manual" which may be found in the following web-site:

http://www.usbr.gov/pmts/hydraulics_lab/pubs/wmm/indexframe.html

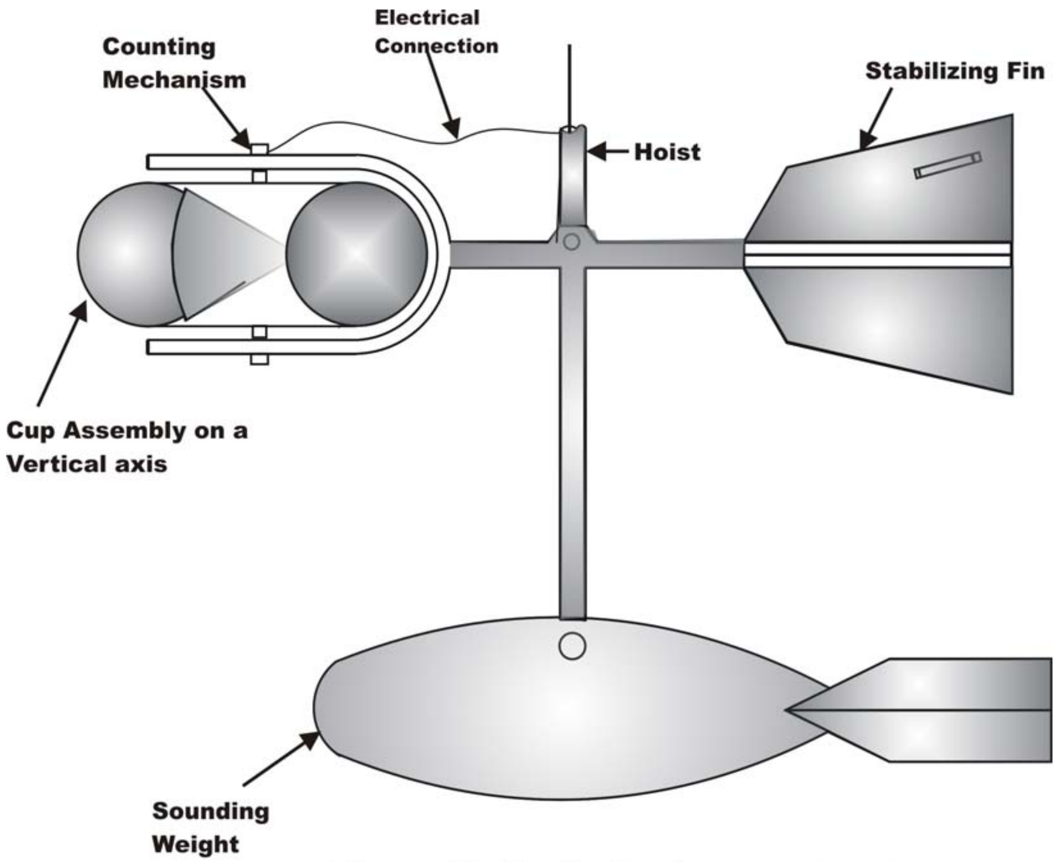


Figure 11. Vertical axis current meter

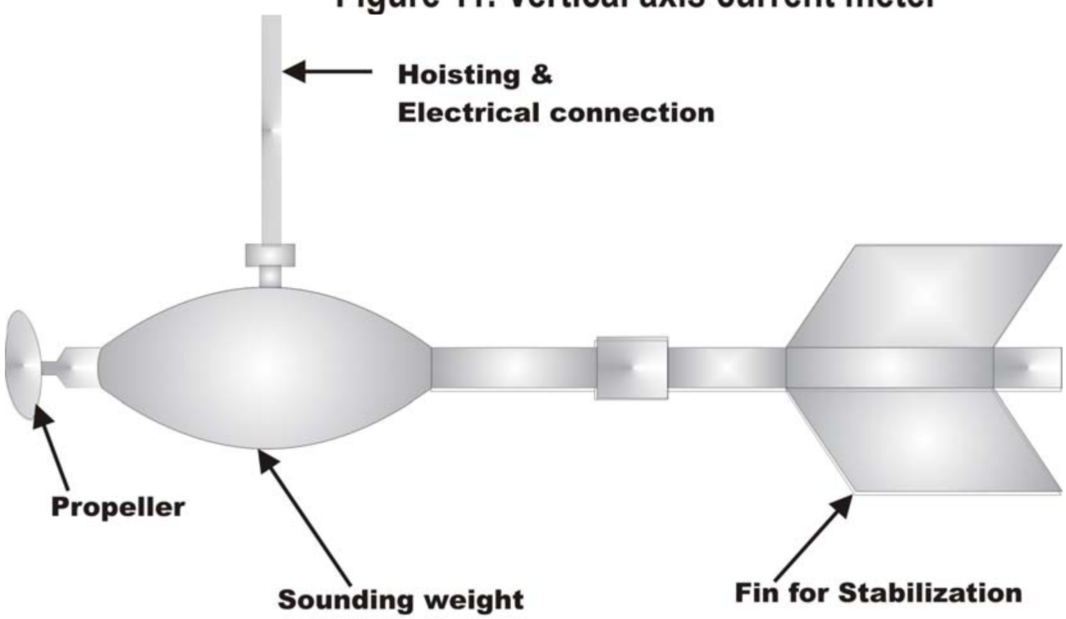


Figure 12. Horizontal- axis current meter

4.1.19 Important terms

Arithmetic increase method

In this method it is assumed that, the population increases at constant a rate. Therefore, population after *n* decades $P_n = P_O + n\bar{x}$

Where,

P_O → Present population.

P_n → Forecasted population after *n* decades.

$\bar{x}$ → Arithmetic mean of population increase in the known decades.

n → No of decades.

Geometric increase method

In this method, the decade wise percentage increase or percent growth rate is assumed to be constant. Thus, population after *n* decades $P_n = P_O\left(1+\frac{r}{100}\right)^n$

Where,

P_O → Present population.

P_n → Forecasted population after *n* decades.

r → Percent of increase in population in the known decades.

n → No of decades.

Incremental increase method

In this method it is assumed that per decade growth rate is not constant, but is progressively increasing or decreasing. Hence, population after *n* decades $P_n = P_O + n\bar{x} + \frac{n(n+1)}{2}\bar{y}$

Where,

P_O → Present population.

P_n → Forecasted population after *n* decades.

$\bar{x}$ → Average increase in the known decades.

$\bar{y}$ → Average incremental increase in known decades.

n → No of decades.

Hydrograph: This is a plot of the discharge of a stream versus time.

Spillway: Spillway is the sluiceway/passage that carries excess water from the water body over a dam or any other obstructions.

Major, Medium and Minor Projects: This is a classification of the irrigation projects in India according to the area of land cultivated.

Toposheet: The Survey of India has published maps of the entire country in different scales. Usually, the ones in scale 1:25,000 or 1:50,000 have the elevation contours marked out in meters. These maps are called topography sheets, or toposheets, in short.

Lidar: LIDAR is an acronym for Light Detection and Ranging. This instrument can:

- Measure distance
- Measure speed
- Measure rotation
- Measure chemical composition and concentration of a remote target where the target can be a clearly defined object, such as a vehicle, or a diffuse object such as a smoke plume or clouds

For more information, one may visit: www.lidar.com

Bore Holes

The sub-soil investigation report will contain the data obtained from boreholes. The report should give the recommendations about the suitable type of foundation, allowable soil pressure and expected settlements. All relevant data for the borehole is recorded in a boring log. Depending upon the type of soil the purpose of boring, the following methods are used for drilling the holes.

- Auger drilling
- Wash boring
- Rotary drilling
- Percussion drilling
- Core boring

Boring Log

It is essential to give a complete and accurate record of data collected. Each borehole should be identified by a code number. All relevant data for the borehole is recorded in a boring log. A boring log gives the description or classification of various strata encountered at different depths. Any additional information that is obtained in the field, such as soil consistency, unconfined compression strength, standard penetration test, cone penetration test, is also indicated on the boring log. It also shows the water table. If the laboratory tests have been conducted, the information about the index properties, compressibility, shear strength, permeability, etc. should also be provided in this log.

Strength Parameters: These are the physical strength characteristics of soils and the important ones are:

- Shear strength (τ)

- Internal angle of friction or angle of shearing resistance (Φ)
- Cohesion intercept (c)
- Effective stress (σ')

Shear strength (τ) of a soil is its maximum resistance to shear stresses just before the failure. Shear failure occurs of a soil mass occurs when the shear stresses induced due to applied compressive loads exceed the shear strength of the soil. Soils are seldom subjected to direct shear. However the shear stresses develop when the soil is subjected to direct compression. Shear strength is the principal engineering property which controls the stability of soil mass under loads. It governs the bearing capacity of the soils, the stability of slopes in soils, and the earth pressure against retaining structures.
Shear strength of a soil at a point on a particular plane was expressed by coulomb as a linear function of normal stress an that plane, as

$$\tau = c + \sigma \tan\phi$$

Where,

c = cohesion interception

ϕ = angle which the envelop makes with σ–axis called angle of internal friction

Effective stress (σ') at any point in the soil mass is equal to the total stress minus pore water pressure. Total stress (σ) on the base of a prism is equal to the force per unit area which is given

$$\sigma = P/A = \gamma_{sat}\,h$$
$$(\sigma') = \sigma - u = \gamma_{sat}\,h - \gamma_w\,h$$
$$\sigma' = (\gamma_{sat} - \gamma_w)h = \gamma' h$$

Strength Tests: The following tests are used to measure the shear strength of soil.

- Direct shear test.
- Triaxial compression test
- Unconfined compression test
- Vane shear test

Direct shear test: This test is performed to determine the consolidated-drained shear strength of a sandy to silty soil. The shear strength is one of the most important engineering properties of a soil, because it is required whenever a structure is dependent on the soil's shearing resistance. The shear strength is needed for engineering situations such as determining the stability of slopes or cuts, finding the bearing capacity for foundations, and calculating the pressure exerted by a soil on a retaining wall.

The direct shear test is one of the oldest strength tests for soils. Direct shear device will be used to determine the shear strength of a cohesionless soil (i.e. angle of internal friction (f)). From the plot of the shear stress versus the horizontal displacement, the maximum shear stress is obtained for a specific vertical confining stress. After the experiment is run several times for various vertical-confining stresses, a plot of the maxi mum shear stresses versus the vertical (normal) confining stresses for each of the tests is produced. From the plot, a straight-line approximation of the Mohr-Coulomb failure envelope curve can be drawn, f may be determined, and, for cohesionless soils (c = 0), the shear strength can be computed from the following equation:

S = S*Tan (f)

- Direct shear device
- Load and deformation dial gauges
- Balance

Triaxial compression test: Trial test is used for determination of shear characteristics of all types of soils under different drainage conditions. The test has been explained in the Indian standard code (IS: 2720-1997).

Unconfined compression test: The unconfined compression test is a special form of a triaxial test in which the confining pressure is zero. The test can be conducted only on clayey soils which can withstand confinement. The test is generally performed on intact, saturated clay specimens.

Vane Shear Test: The undrained shear strength of soft clays or rocks can be determined in the laboratory by vane shear test. The test can also be conducted in the field on the soil at the bottom of the borehole. The field test can be performed even without drilling a bore hole by the direct penetration of the vane from the ground surface.

In-situ Laboratory Tests

The strength parameters of soil or rock layers are investigated by extracting cores of samples and taken to the laboratory for testing. Insitu laboratory tests are conducted to avoid disturbing of foundation material. These Insitu laboratory tests mainly include plate jack test for soils and hydro fracture test for rocks.

The hydro-fracture test is done to determine the strength of underlying strata, in case of site where huge structures, such as dams, etc are built. In this test, water is injected into the soil at huge pressures and checked if the soil is able to bear the pressure and even the magnitude of fractured rock can be estimated. In the hydro-fracture test the magnitude of the minimum principal stress is determined and back analysis is done from monitored deformations, when suitable excavations are made for other purposes and economical monitoring can be used. D5607-02 gives standard test method for performing laboratory direct shear strength tests of rock specimens under constant normal force. The Insitu shear test or the plate jack test for the soils is explained in IS: 2720-Part39/sec2.

Bearing Pressure

Foundations for structures are generally classified as deep and shallow. Deep foundations generally refer to piled foundations, whereas shallow foundations include pad foundations, raft foundations, and strip footings. The performance and functional viability of a foundation depends on the interaction between the structure which is supported and on the founding material. The behavior of the soil depends on the bearing pressure and width of the foundation, hence the bearing capacity is not simply a function of the soil, but rather is also a function of the specific foundation arrangement. Bearing pressure is the maximum pressure at which the supporting ground is expected to fail in shear.

Orographic: Denotes effects that are related to the presence of mountains or high ground on, say, rainfall. Orography is the study of the physical geography of mountains and mountain ranges.

Non- Recording and Recording rain gauges

The **non-recording rain gauge** that is extensively used in India is the *Symon's gauge*. It essentially consists of a circular collecting area connected to a funnel. The rim of the collector is set in a horizontal plane at a suitable height above the ground level. The funnel discharges the rainfall catch into a receiving vessel. The funnel and receiving vessel are housed in a metallic container. Water contained in the receiving vessel is measured by a suitably graduated measuring glass, with accuracy up to 0.1mm. Recently India Meteorological Department (IMD) has changed over to the use of *fiberglass reinforced polyester raingauges*, which is an improvement over the Symon's gauge. These come in different combinations of collector and bottles.

Recording rain gauges produce a continuous plot against time and provide valuable data of intensity and duration of rainfall for hydrologic analysis of storms. Following are some of the commonly used recording rain gauges.

1. Tipping bucket type
2. Weighing bucket type
3. Natural siphon type
4. Telemetering Rain gauges.

For a detailed list of commercial rain gauges usually manufactured, one may refer to the web-site of one of the manufacturers Nova Lynx at the following web-site:
http://www.novalynx.com/products-rain-gauges.html

Current meter

Current meters are velocity measuring devices that that are used to measure the velocity of a stream at a point. Each point velocity measurement is then assigned to a meaningful part of the entire cross section passing flow. Several classes of current meters are used in water measurement.

- Anemometer and propeller velocity meter
- Electromagnetic velocity meters
- Doppler velocity meters
- Optical strobe velocity meters

One may consult the United States Bureau of Reclamation online document "Water measurement manual" for more information which may be found in the following web-site:
http://www.usbr.gov/pmts/hydraulics_lab/pubs/wmm/indexframe.html

4.1.19 Important organizations

Central Water Commission

Central Water Commission is a premier Technical Organization in the country in the field of Water Resources since 1945 and is presently functioning as an attached office of the Ministry of Water resources. The Commission is charged with the general responsibilities of initiating, coordinating and furthering in consultation of the State Governments concerned, schemes for control, conservation and utilization of water resources throughout the country, for purpose of Flood Control, Irrigation, Navigation, Drinking Water, Power Development & water supply. It also undertakes the investigations, construction and execution of any such schemes as required.
Web-site: http://cwc.nic.in/

Survey of India

Survey of India, The National Survey and Mapping Organization of the country under the Department of Science & Technology, is the oldest scientific department of the govt. of India. It was set up in 1767 and has evolved rich traditions over the years. In its assigned role as the Nation's principal mapping agency, Survey of India bears a special responsibility to ensure that the country's domain is explored and mapped suitably to provide base maps for expeditious and integrated development and ensure that all resources contribute their full measure to the progress, prosperity and security of our country now and for generations to come.
Web-site: http://dst.gov.in/scservices/soi.htm

National Remote Sensing Agency

National Remote Sensing Agency (NRSA) is an autonomous organization under Department of Space, Govt. of India engaged in operational remote sensing activities. The operational use of remote sensing applications is in the fields of water resources, agriculture, soil and land degradation, mineral exploration, groundwater targeting, geomorphologic mapping, coastal and ocean resources monitoring, environment, ecology and forest mapping, land use and land cover mapping and urban area studies, large scale mapping, etc.

The chief activities are satellite data and aerial data reception, data processing, data dissemination; applications for providing value added services and training.

Web-site: http://www.nrsa.gov.in/

Geological Survey of India

This is the premier organization of Earth Science Studies in the sub-continent with strength of 2900 geoscientists and technical professionals. The GSI has a network of Offices located in all the states of India. It is the custodian of Geoscientific database developed over a period of 150 years and is capable of handling time-bound jobs in different sub disciplines of earth science: from geological mapping to deposit modeling. It is also equipped with modern laboratories run by professionals. It possesses organizational setup to impart training in the fields of earth science and holds the key to mineral exploration.

Web-site: www.gsi.gov.in

Module
2
The Science of Surface and Ground Water

Lesson
1
Precipitation And Evapotranspiration

Instructional Objectives

On completion of this lesson, the student shall learn:

1. The role of precipitation and evapotranspiration with the hydrologic cycle.
2. The factors that cause precipitation.
3. The means of measuring rainfall.
4. The way rain varies in time and space.
5. The methods to calculate average rainfall over an area.
6. What are Depth – Area – Duration curves.
7. What are the Intensity – Duration – Frequency curves.
8. The causes of anomalous rainfall record and its connective measures.
9. What are Probable Maximum Precipitation (PMP) and Standard Project Storm (SPS).
10. What are Actual and Potential evapotranspiration.
11. How can direct measurement of evapotranspiration be made.
12. How can evapotranspiration be estimated based on climatological data.

2.1.0 Introduction

Precipitation is any form of solid or liquid water that falls from the atmosphere to the earth's surface. Rain, drizzle, hail and snow are examples of precipitation. In India, rain is the most common form of precipitation.

Evapotranspiration is the process which returns water to the atmosphere and thus completes the hydrologic cycle. Evapotranspiration consists of two parts, Evaporation and Transpiration. Evaporation is the loss of water molecules from soil masses and water bodies. Transpiration is the loss of water from plants in the form of vapour. We proceed on to discuss precipitation, and its most important component in India context, the rainfall.

2.1.1 Causes of precipitation

For the formation of clouds and subsequent precipitation, it is for necessary that the moist air masses to cool in order to condense. This is generally accomplished by adiabatic cooling of moist air through a process of being lifted to higher altitudes. The precipitation types can be categorized as.

- **Frontal precipitation**
  This is the precipitation that is caused by the expansion of air on ascent along or near a frontal surface.

- **Convective precipitation**
  Precipitation caused by the upward movement of air which is warmer than its surroundings. This precipitation is generally showery nature with rapid changes of intensities.

- **Orographic precipitation**
  Precipitation caused by the air masses which strike the mountain barriers and rise up, causing condensation and precipitation. The greatest amount of precipitation will fall on the windward side of the barrier and little amount of precipitation will fall on leave ward side.

For the Indian climate, the south-west monsoon is the principal rainy season when over 75% of the annual rainfall is received over a major portion of the country. Excepting the south-eastern part of the Indian peninsula and Jammu and Kashmir, for the rest of the country the south-west monsoon is the principal source of rain.

From the point of view of water resources engineering, it is essential to quantify rainfall over space and time and extract necessary analytical information.

2.1.2 Regional rainfall characteristics

Rain falling over a region is neither uniformly distributed nor is it constant over time. You might have experienced the sound of falling rain on a cloudy day approaching from distance. Gradually, the rain seems to surround you and after a good shower, it appears to recede. It is really difficult to predict when and how much of rain would fall. However it is possible to measure the amount of rain falling at any point and measurements from different point gives an idea of the rainfall pattern within an area.

In India, the rainfall is predominantly dictated by the monsoon climate. The monsoon in India arises from the reversal of the prevailing wind direction from Southwest to Northeast and results in three distinct seasons during the course of the year. The Southwest monsoon brings heavy rains over most of the country between June and October, and is referred to commonly as the 'wet' season. Moisture laden winds sweep in from the Indian Ocean as low-pressure areas develop over the subcontinent and release their moisture in the form of heavy rainfall. Most of the annual rainfall in India comes at this time with the exception of in Tamil Nadu, which receives over half of its rain during the Northeast monsoon from October to November.

The retreating monsoon brings relatively cool and dry weather to most of India as drier air from the Asian interior flows over the subcontinent. From

November until February, temperatures remain cool and precipitation low. In northern India it can become quite cold, with snow occurring in the Himalayas as weak cyclonic storms from the west settle over the mountains. Between March and June, the temperature and humidity begin to rise steadily in anticipation of the Southwest monsoon. This pre-monsoonal period is often seen as a third distinct season although the post-monsoon in October also presents unique characteristics in the form of slightly cooler temperatures and occasional light drizzling rain. These transitional periods are also associated with the arrival of cyclonic tropical storms that batter the coastal areas of India with high winds, intense rain and wave activity.

Rainfall and temperature vary greatly depending on season and geographic location. Further, the timing and intensity of the monsoon is highly unpredictable. This results in a vastly unequal and unpredictable distribution over time and space. In general, the northern half of the subcontinent sees greater extremes in temperature and rainfall with the former decreasing towards the north and the latter towards the west. Rainfall in the Thar Desert and areas of Rajasthan can be as low as 200mm per year, whereas on the Shillong Plateau in the Northeast, average annual rainfall can exceed 10,000 mm per year. The extreme southern portion of the country sees less variation in temperature and rainfall. In Kerala, the total annual rainfall is of the order of 3,000 mm.

In this lecture, we discuss about rainfall measurement and interpretation of the data.

2.1.3 Measurement of rainfall

One can measure the rain falling at a place by placing a measuring cylinder graduated in a length scale, commonly in mm. In this way, we are not measuring the volume of water that is stored in the cylinder, but the 'depth' of rainfall. The cylinder can be of any diameter, and we would expect the same 'depth' even for large diameter cylinders provided the rain that is falling is uniformly distributed in space.

Now think of a cylinder with a diameter as large as a town, or a district or a catchment of a river. Naturally, the rain falling on the entire area at any time would not be the same and what one would get would be an 'average depth'. Hence, to record the spatial variation of rain falling over an area, it is better to record the rain at a point using a standard sized measuring cylinder.

In practice, rain is mostly measured with the ***standard non-recording rain gauge*** the details of which are given in Bureau of Indian Standards code IS 4989: 2002. The rainfall variation at a point with time is measured with a ***recording rain-gauge***, the details of which may be found in IS 8389: 2003. Modern technology has helped to develop Radars, which measures rainfall over an entire region. However, this method is rather costly compared to the conventional recording and non-recording rain gauges which can be monitored easily with cheap labour.

2.1.4 Variation of rainfall

Rainfall measurement is commonly used to estimate the amount of water falling over the land surface, part of which infiltrates into the soil and part of which flows down to a stream or river. For a scientific study of the hydrologic cycle, a correlation is sought, between the amount of water falling within a catchment, the portion of which that adds to the ground water and the part that appears as streamflow. Some of the water that has fallen would evaporate or be extracted from the ground by plants.

FIGURE 1. A hypothetical catchment showing four raing gauge stations

In Figure 1, a catchment of a river is shown with four rain gauges, for which an assumed recorded value of rainfall depth have been shown in the table.

		Time (in hours)				Total Rainfall
		First	Second	Third	Fourth	
Rain(mm)	A	15	10	3	2	30
	B	12	15	8	5	40
	C	8	10	6	4	28
	D	5	8	2	2	17

It is on the basis of these discrete measurements of rainfall that an estimation of the average amount of rainfall that has probably fallen over a catchment has to be made. Three methods are commonly used, which are discussed in the following section.

2.1.5 Average rainfall depth

The time of rainfall record can vary and may typically range from 1 minute to 1 day for non – recording gauges, Recording gauges, on the other hand, continuously record the rainfall and may do so from 1 day 1 week, depending on the make of instrument. For any time duration, the average depth of rainfall falling over a catchment can be found by the following three methods.

- The Arithmetic Mean Method
- The Thiessen Polygon Method
- The Isohyetal Method

Arithmetic Mean Method

The simplest of all is the Arithmetic Mean Method, which taken an average of all the rainfall depths as shown in Figure 2.

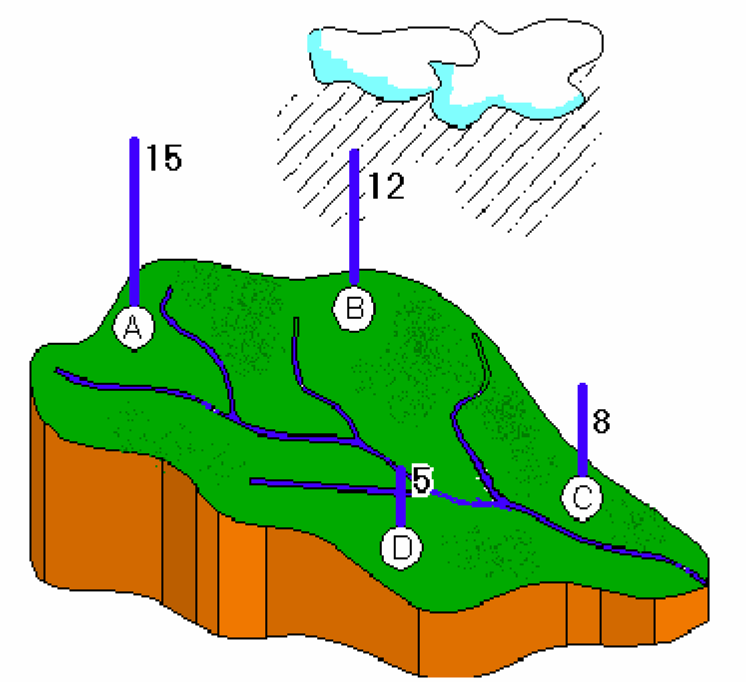


FIGURE 2. Representation of the rainfall recorded in the four rain gauges (values in mm)

Average rainfall as the arithmetic mean of all the records of the four rain gauges, as shown below:

$$\frac{15+12+8+5}{4} = 10.0\,\text{mm}$$

The Theissen polygon method

This method, first proposed by Thiessen in 1911, considers the representative area for each rain gauge. These could also be thought of as the areas of influence of each rain gauge, as shown in Figure 3.

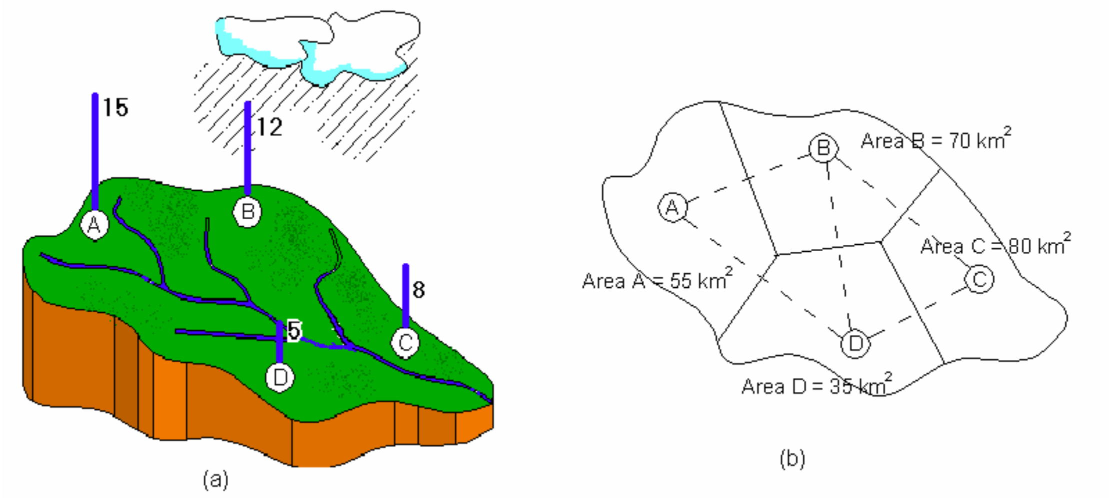


FIGURE 3. Rainfall measurement by Thiessen Polygon method. (a) Rainfall recorded; (b) Areas of influences

These areas are found out using a method consisting of the following three steps:

1. Joining the rain gauge station locations by straight lines to form triangles
2. Bisecting the edges of the triangles to form the so-called "Thiessen polygons"
3. Calculate the area enclosed around each rain gauge station bounded by the polygon edges (and the catchment boundary, wherever appropriate) to find the area of influence corresponding to the rain gauge.

For the given example, the "weighted" average rainfall over the catchment is determined as,

$$\frac{65\times15+70\times12+35\times8+80\times5}{(55+70+35+80)} = 10.40\ \text{mm}$$

The Isohyetal method

This is considered as one of the most accurate methods, but it is dependent on the skill and experience of the analyst. The method requires the plotting of ***isohyets*** as shown in the figure and calculating the areas enclosed either between the isohyets or between an isohyet and the catchment boundary. The areas may be measured with a ***planimeter*** if the catchment map is drawn to a scale.

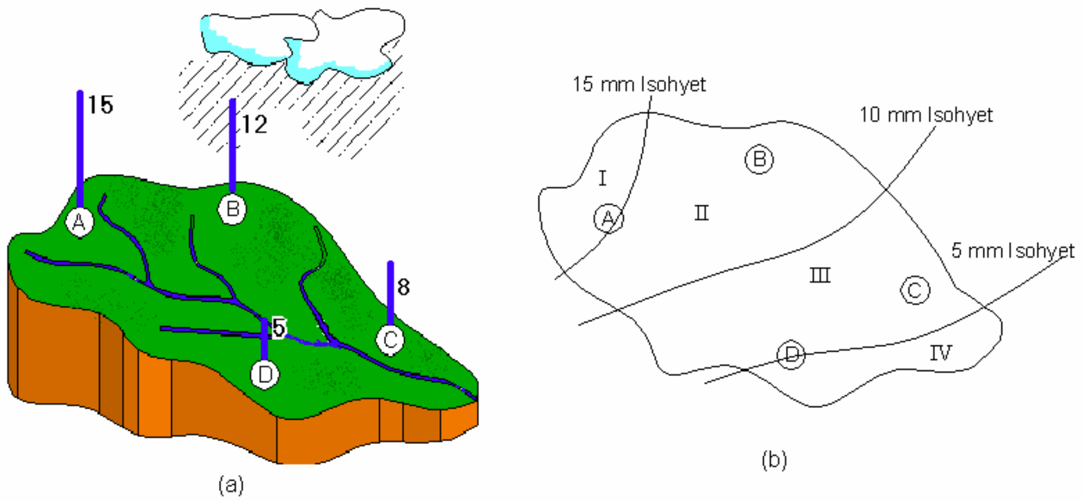


FIGURE 4. Rainfall measurement by the Isohyetal method. (a) Recorded rainfall; (b) Isohyets and the areas enclosed bewteen two consecutive isohyets.

For the problem shown in Figure 4, the following may be assumed to be the areas enclosed between two consecutive isohyets and are calculated as under:

Area I = 40 km^2
Area II = 80 km^2
Area III = 70 km^2
Area IV = 50 km^2
Total catchment area = 240 km^2

The areas II and III fall between two isohyets each. Hence, these areas may be thought of as corresponding to the following rainfall depths:

Area II : Corresponds to (10 + 15)/2 = 12.5 mm rainfall depth
Area III : Corresponds to (5 + 10)/2 = 7.5 mm rainfall depth

For Area I, we would expect rainfall to be more than 15mm but since there is no record, a rainfall depth of 15mm is accepted. Similarly, for Area IV, a rainfall depth of 5mm has to be taken.

Hence, the average precipitation by the isohyetal method is calculated to be

$$\frac{40\times15+80\times12.5+70\times7.5+50\times5}{240}$$

= 9.89 mm

Please note the following terms used in this section:

Isohyets: Lines drawn on a map passing through places having equal amount of rainfall recorded during the same period at these places (these lines are drawn after giving consideration to the topography of the region).

Planimeter: This is a drafting instrument used to measure the area of a graphically represented planar region.

2.1.6 Mean rainfall

This is the average or representative rainfall at a place. The mean annual rainfall is determined by averaging the total rainfall of several consecutive years at a place. Since the annual rainfall varies at the station over the years, a record number of years are required to get a correct estimate.

Similarly, the mean monthly rainfall at a place is determined by averaging the monthly total rainfall for several consecutive years. For example, the mean rainfall along with the mean number of rainy days for New Delhi (as obtained from World Meteorological Organisation – WMO) is as follows:

Month	Mean Total Rainfall (mm)	Mean Number of Rain Days
Jan	20.3	1.7
Feb	15.0	1.3
Mar	15.8	1.2
Apr	6.7	0.9
May	17.5	1.4
Jun	54.9	3.6
Jul	231.5	10.0
Aug	258.7	11.3
Sep	127.8	5.4
Oct	36.3	1.6
Nov	5.0	0.1
Dec	7.8	0.6

In comparison, that for the city of Kolkata, obtained from the same source, is as follows:

Month	Mean Total Rainfall (mm)	Mean Number of Rain Days
Jan	16.8	0.9
Feb	22.9	1.5
Mar	32.8	2.3
Apr	47.7	3.0
May	101.7	5.9
Jun	259.9	12.3
Jul	331.8	16.8
Aug	328.8	17.2
Sep	295.9	13.4

Oct	151.3	7.4
Nov	17.2	1.1
Dec	7.4	0.4

2.1.7 Depth-Area-Duration curves

In designing structures for water resources, one has to know the areal spread of rainfall within watershed. However, it is often required to know the amount of high rainfall that may be expected over the catchment. It may be observed that usually a storm event would start with a heavy downpour and may gradually reduce as time passes. Hence, the rainfall depth is not proportional to the time duration of rainfall observation. Similarly, rainfall over a small area may be more or less uniform. But if the area is large, then due to the variation of rain falling in different parts, the average rainfall would be less than that recorded over a small portion below the high rain fall occurring within the area. Due to these facts, a Depth-Area-Duration (DAD) analysis is carried out based on records of several storms on an area and, the maximum areal precipitation for different durations corresponding to different areal extents.

The result of a DAD analysis is the DAD curves which would look as shown in Figure 5.

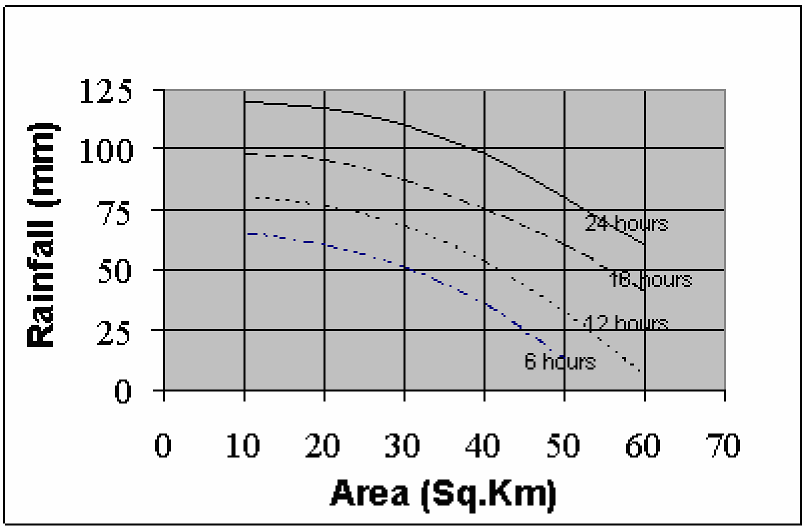


FIGURE 5. A typical Depth-Area-Duration (DAD) curve

2.1.8 Intensity-Duration-Frequency curves

The analysis of continuous rainfall events, usually lasting for periods of less than a day, requires the evaluation of rainfall intensities. The assessment of such values may be made from records of several part storms over the area and presented in a graphical form as shown in Figure 6.

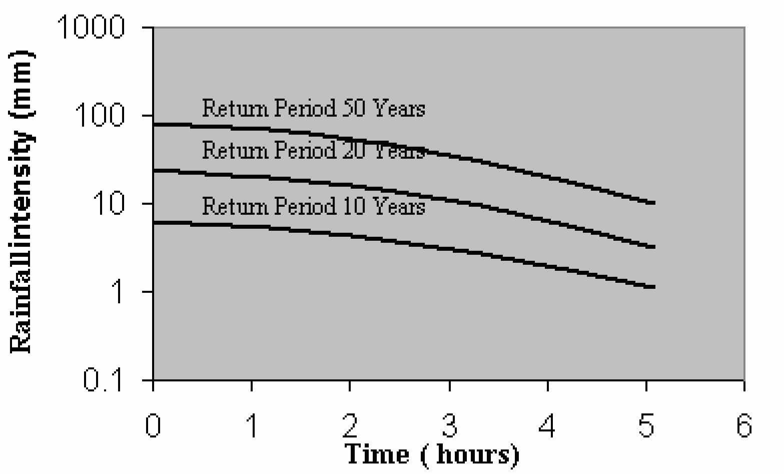


FIGURE 6. A typical rainfall intensity-duration-frequency (IDF) curve

Two new concepts are introduced here, which are:

- **Rainfall intensity**
  This is the amount of rainfall for a given rainfall event recorded at a station divided by the time of record, counted from the beginning of the event.

- **Return period**
  This is the time interval after which a storm of given magnitude is likely to recur. This is determined by analyzing past rainfalls from several events recorded at a station. A related term, the frequency of the rainfall event (also called the storm event) is the inverse of the return period. Often this amount is multiplied by 100 and expressed as a percentage. Frequency (expressed as percentage) of a rainfall of a given magnitude means the number of times the given event may be expected to be equaled or exceeded in 100 years.

2.1.9 Analysis for anomalous rainfall records

Rainfall recorded at various rain gauges within a catchment should be monitored regularly for any anomalies. For example of a number of recording rain gauges located nearby, one may have stopped functioning at a certain point of time, thus breaking the record of the gauge from that time onwards. Sometimes, a perfectly working recording rain gauge might have been shifted to a neighbourhood location, causing a different trend in the recorded rainfall compared to the past data. Such difference in trend of recorded rainfall can also be brought about by a change in the neighbourhood or a change in the ecosystem, etc. These two major types of anomalies in rainfall are categorized as

- Missing rainfall record
- Inconsistency in rainfall record

Missing rainfall record

The rainfall record at a certain station may become discontinued due to operational reasons. One way of approximating the missing rainfall record would be using the records of the three rain gauge stations closet to the affected station by the "Normal Ratio Method" as given below:

$$P_4 = \frac{1}{3}\left[\frac{N_4}{N_1}P_1 + \frac{N_4}{N_2}P_2 + \frac{N_4}{N_3}P_3\right] \qquad (1)$$

Where P_4 is the precipitation at the missing location, N_1, N_2, N_3 and N_4 are the normal annual precipitation of the four stations and P_1, P_2 and P_3 are the rainfalls recorded at the three stations 1, 2 and 3 respectively.

Inconsistency in rainfall record

This may arise due to change in location of rain gauge, its degree of exposure to rainfall or change in instrument, etc. The consistency check for a rainfall record is done by comparing the accumulated annual (or seasonal) precipitation of the suspected station with that of a standard or reference station using a double mass curve as shown in Figure 7.

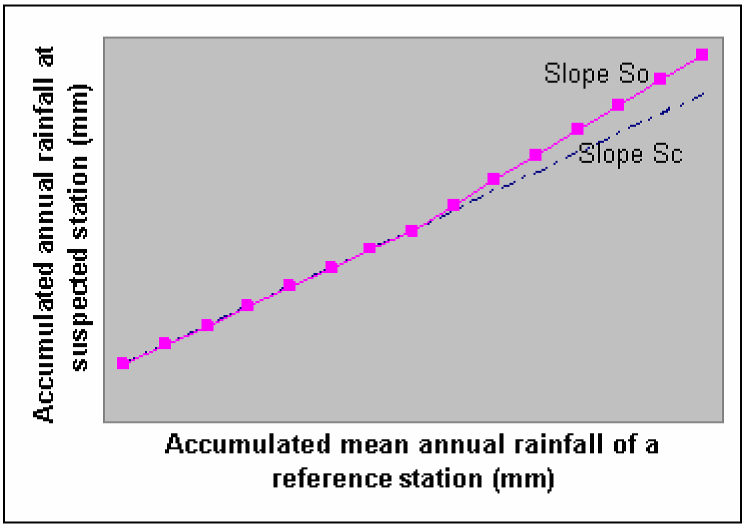


FIGURE 7. A typical example of inconsistent rainfall record

From the calculated slopes S_0 and S_c from the plotted graph, we may write

$$P_c = P_0\left(\frac{S_c}{S_0}\right) \quad (2)$$

Where P_c and P_0 are the corrected and original rainfalls at suspected station at any time. S_c and S_0 are the corrected and original slopes of the double mass-curve.

2.1.10 Probable extreme rainfall events

Two values of extreme rainfall events are important from the point of view of water resources engineering. These are:

Probable Maximum Precipitation (PMP)

This is the amount of rainfall over a region which cannot be exceeded over at that place. The PMP is obtained by studying all the storms that have occurred over the region and maximizing them for the most critical atmospheric conditions. The PMP will of course vary over the Earth's surface according to the local climatic factors. Naturally, it would be expected to be much higher in the hot humid equatorial regions than in the colder regions of the mid-latitudes when the atmospheric is not able to hold as much moisture. PMP also varies within India, between the extremes of the dry deserts of Rajasthan to the ever humid regions of South Meghalaya plateau.

Standard Project Storm (SPS)

This is the storm which is reasonably capable of occurring over the basin under consideration, and is generally the heaviest rainstorm, which has occurred in the region of the basin during the period of rainfall records. It is not maximized for the most critical atmospheric conditions but it may be transposed from an adjacent region to the catchment under considerations.

The methods to obtain ***PMP*** and ***SPS*** are involved and the interested reader mayfind help in text books on hydrology, such as the following:

- Mutreja, K N (1995) Applied Hydrology, Tata McGraw Hill
- Subramanya, K (2002) Engineering Hydrology, Tata McGraw Hill

2.1.11 Evapotranspiration

As discussed earlier, evapotranspiration consists of evaporation from soil and water bodies and loss of water from plant leaves, which is called transpiration. It is a major component of the hydrologic cycle and its information is needed to design irrigation projects and for managing water quality and other environmental concerns. In urban development, evapotranspiration calculations are used to determine safe yields from aquifers and to plan for flood control. The term consumptive use is also sometimes used to denote the loss of water molecules to atmosphere by evapotranspiration. For a given set of atmospheric conditions, evapotranspiration depends on the availability of water. If sufficient moisture is always available to completely meet the needs of vegetation fully covering the area, the resulting evapotranspiration is called potential evapotranspiration (PET). The real evapotranspiration occurring in a specific situation is called actual evapotranspiration (AET).

2.1.12 Measurement of evapotranspiration

There are several methods available for measuring evaporation or evapotranspiration, some of which are given in the following sub-sections.

2.1.12.1 Potential Evapotranspiration (PET)

- **Pan evaporation**

  The evaporation rate from pans filled with water is easily obtained. In the absence of rain, the amount of water evaporated during a period (mm/day) corresponds with the decrease in water depth in that period. Pans provide a measurement of the integrated effect of radiation, wind, temperature and humidity on the evaporation from an open water surface. Although the pan responds in a similar fashion to the same climatic factors affecting crop transpiration, several factors produce significant differences in loss of water from a water surface and from a cropped surface. Reflection of solar radiation from water in the shallow pan might be different from the assumed 23% for the grass reference surface. Storage of heat within the pan can be appreciable and may cause significant evaporation during the night while most crops transpire only during the daytime. There are also differences in turbulence, temperature and humidity of the air immediately above the respective surfaces. Heat transfer through the sides of the pan occurs and affects the energy balance.

  Notwithstanding the difference between pan-evaporation and the evapotranspiration of cropped surfaces, the use of pans to predict ET_o for periods of 10 days or longer may be warranted. The pan evaporation is related to the reference evapotranspiration by an empirically derived pan coefficient:

$$ET_o = K_p \, E_{pan}$$

Where

ET_o reference evapotranspiration [mm/day],
K_p pan coefficient [-],
E_{pan} pan evaporation [mm/day].

- **Evapotranspiration gauges**

  The modified Bellani plate atmometer has been offered as an alternative and simpler technique to combination-based equations to estimate evapotranspiration (ET) rate from green grass surface.

2.1.12.2 Actual Evapotranspiration (AET)

- **Simple methods**
  - Soil water depletion method

    Evapotranspiration can be measured by using soil water depletion method. This method is usually suitable for areas where soil is fairly uniform. Soil moisture measured at various time intervals. Evapotranspiration can be measured from the difference of soil moisture at various time levels.

  - Water balance method

    The method is essentially a book-keeping procedure which estimates the balance between the inflow and outflow of water. In a standard soil water balance calculation, the volume of water required to saturate the soil is expressed as an equivalent depth of water and is called the soil water deficit. The soil water balance can be represented by:

$$Ea = P - Gr + \Delta S - Ro$$

Where, Gr = recharge;
P = precipitation;
Ea = actual evapotranspiration;
ΔS = change in soil water storage; and
Ro = run-off.

- **Complex methods**
  - Lysimeters

    A lysimeter is a special watertight tank containing a block of soil and set in a field of growing plants. The plants grown in the lysimeter are the same as in the surrounding field. Evapotranspiration is estimated in terms of the amount of water required to maintain constant moisture conditions within the tank measured either volumetrically or gravimetrically through an arrangement made in the lysimeter. Lysimeters should be designed to accurately reproduce the soil conditions, moisture content, type and size of the vegetation of the surrounding area. They should be so hurried that the soil is at the same level inside and outside the container. Lysimeter studies are time-consuming and expensive.

  - Energy balance method

    The energy balance consists of four major components: net radiation input, energy exchange with soil, energy exchange to heat the air (sensible heat) and energy exchange to evaporate water (latent energy). Latent energy is thus the budget involved in the process of evapotranspiration:

Net Radiation -Ground Heat Flux = Sensible Heat + Latent Energy

The energy balance method of determining Evapotranspiration can be used for hourly values during daylight hours but accurate night time values are difficult to obtain. Eddy diffusion equations can be used and combinations of these procedures can be used also to calculate evapotranspiration. The method used is governed often by the data available, the accuracy needed, and the computational capability.

- Mass transfer method
  This is one of the analytical methods for the determination of lake evaporation. This method is based on theories of turbulent mass transfer in boundary layer to calculate the mass water vapour transfer from the surface to the surrounding atmosphere.

2.1.13 Estimation of Evapotranspiration

The lack of reliable measured data from field in actual projects has given rise to a number of methods to predict Potential Evapotranspiration (PET) using climatological data. The more commonly used methods to estimate evapotranspiration are the following:

- Blaney-Criddle method
- Modified Penman Method
- Jansen-Haise method
- Hargreaves method
- Thornwaite method

Some of the more popular of these methods have been discussed in detail in lesson 5.4 “Estimating irrigation demand”. Interested readers may consult Modi, P N (2000) Water Resources Engineering for detailed discussions on this issue.

Module 2

The Science of Surface and Ground Water

Lesson 2

Runoff and Infiltration

Instructional Objectives

At the end of this lesson, the student shall be able to learn:

1. The importance of runoff and infiltration in the hydrologic cycle.
2. What is the difference between overland flow, interflow and base flow components contributing to stream flow generation.
3. What are hydrograph and hyetographs.
4. Methods to separate infiltration from rainfall hyetographs effective rainfall.
5. Methods to separate base flow from stream hydrograph to find out the Direct Runoff Hydrograph.

2.2.0 Introduction

The amount of precipitation flowing over the land surface and the evapotranspiration losses from land and water bodies were discussed in Lesson 2.1. This water ultimately is returned to the sea through various routes either overland or below ground. Evaporation from the ocean, which is actually a large water body, contributes to the bulk of water vapour to the atmosphere, driven by the energy of the sun. This process completes the ***hydrologic cycle*** (Figure 1), which keeps the water content of the Earth in a continuous dynamic state.

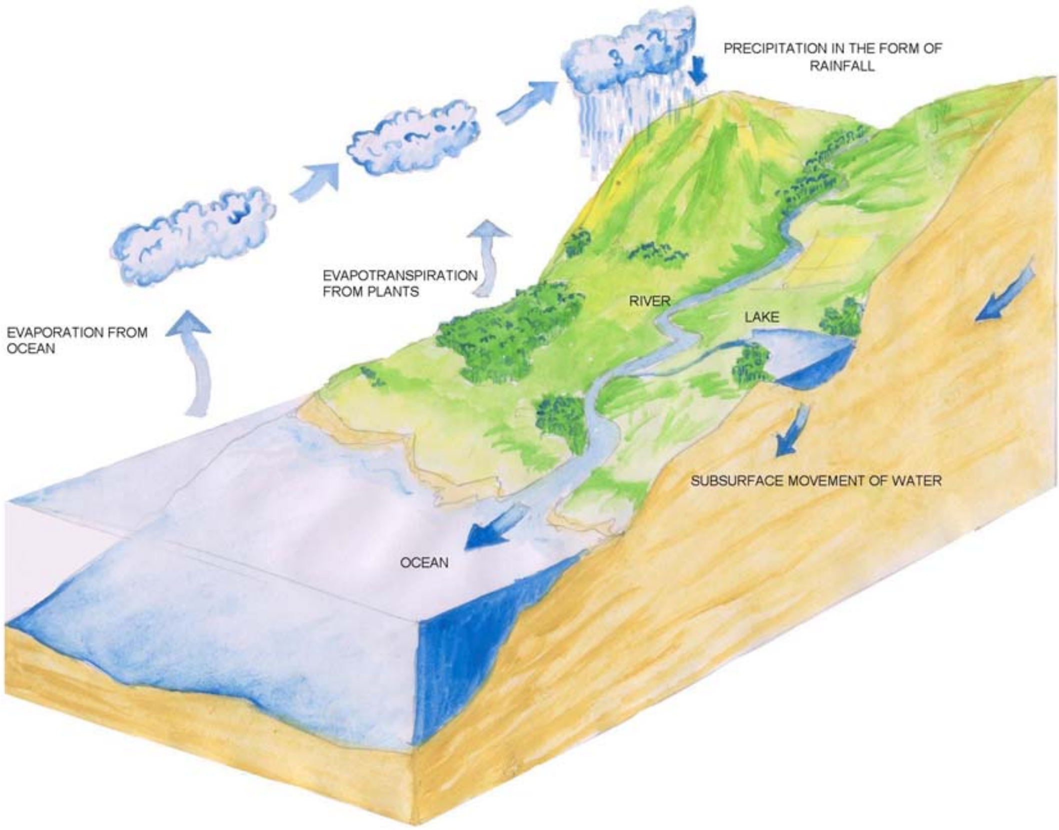


FIGURE 1. HYDROLOGIC CYCLE

In this lesson, we would study the fate of the raindrops as they fall on the earth and flow down the land surface to meet streams and rivers. Part of the water, as it flows down the land surface, infiltrates into the soil and ultimately contributes to the ground water reserve.

2.2.1 Overland flow and inter flow

During a precipitation event, some of the rainfall is intercepted by vegetation before it reaches the ground and this phenomenon is known as ***interception***. At places without any vegetation, the rain directly touches the land surface. This water can infiltrate into the soils, form puddles called the ***depression storage***, or flow as a thin sheet of water across the land surface. The water trapped in puddles ultimately evaporates or infiltrates. If the soil is initially quite dry, then most of the water infiltrates into the ground. The amount of rainfall in excess of the infiltrated quantity flows over the ground surface following the land slope. This is the ***overland flow***. The portion that infiltrates moves through an unsaturated portion of the soil in a vertical direction for some depth till it meets the water table, which is the free surface of a fully saturated region with water (the ground water reserve). Part of the water in the ***unsaturated zone*** of the soil (also called the ***vadose zone***) moves in a lateral direction, especially if the ***hydraulic conductivity*** in the horizontal direction is more than that in vertical direction and emerges at the soil surface at some location away from the point of entry into the soil. This phenomenon is known as ***interflow***. Figure 2 illustrates the flow components schematically.

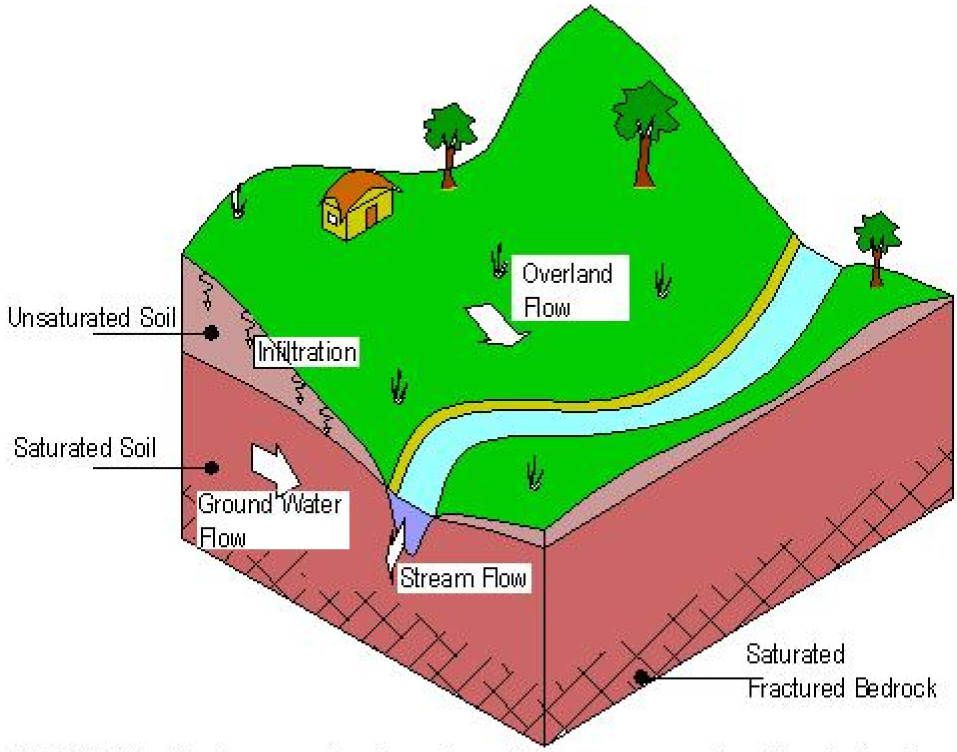


FIGURE 1. Surface and sub-surface flow components of hydrologic cycle

Please note the meaning of the term ***Hydraulic conductivity:***

Hydraulic conductivity is a measure of the ability of a fluid to flow through a porous medium and is determined by the size and shape of the pore spaces in the medium and their degree of interconnection and also by the viscosity of the fluid. Hydraulic conductivity can be expressed as the volume of fluid that will move in unit time under a unit hydraulic gradient through a unit area measured at right angles to the direction of flow.

2.2.2 Stream flow and groundwater flow

If the unsaturated zone of the soil is uniformly permeable, most of the infiltrated water percolates vertically. Infiltrated water that reaches the ground water reserve raises the water table. This creates a difference in potential and the inclination of the water table defines the variation of the ***piezometric head*** in horizontal direction. This difference in energy drives the ground water from the higher to the lower head and some of it ultimately reaches the stream flowing through the valley. This contribution of the stream flow is known as Base flow, which usually is the source of dry-weather flow in perennial streams.

During a storm event, the overland flow contributes most of the immediate flow of the stream. The total flow of the stream, however, is the sum of overland flow, interflow and ***base flow***. It must be remembered that the rates at which these three components of runoff move varies widely. Stream flow moves fastest, followed by interflow and then ground water flow, which may take months and sometimes even years to reach the stream.

Note that for some streams, the water table lies quite some distance below the bottom of the stream. For these streams, there is a loss of water from the river bed percolating into the ground ultimately reaching the water table. The reason for a low water table could possibly be due to natural geographic conditions, or a dry climate, or due to heavy pumping of water in a nearby area.

2.2.3 The hydrograph and hyetograph

As the name implies, Hydrograph is the plot of the stream flow at a particular location as a function of time. Although the flow comprises of the contributions from overland flow, interflow and groundwater flow, it is useful to separate only the groundwater flow (the base flow) for hydrograph analysis, which is discussed in Lesson 2.3.

In Lesson 2.1, precipitation was discussed. The hyetograph is the graphical plot of the rainfall plotted against time. Traditionally, the hyetograph is plotted upside down as shown in Figure 3, which also shows a typical hydrograph and its components. Splitting up of a complete stream flow hydrograph into its components requires the knowledge of the geology of the area and of the factors like surface slope, etc. Nevertheless, some of the simpler methods to separate base flow are described subsequently.

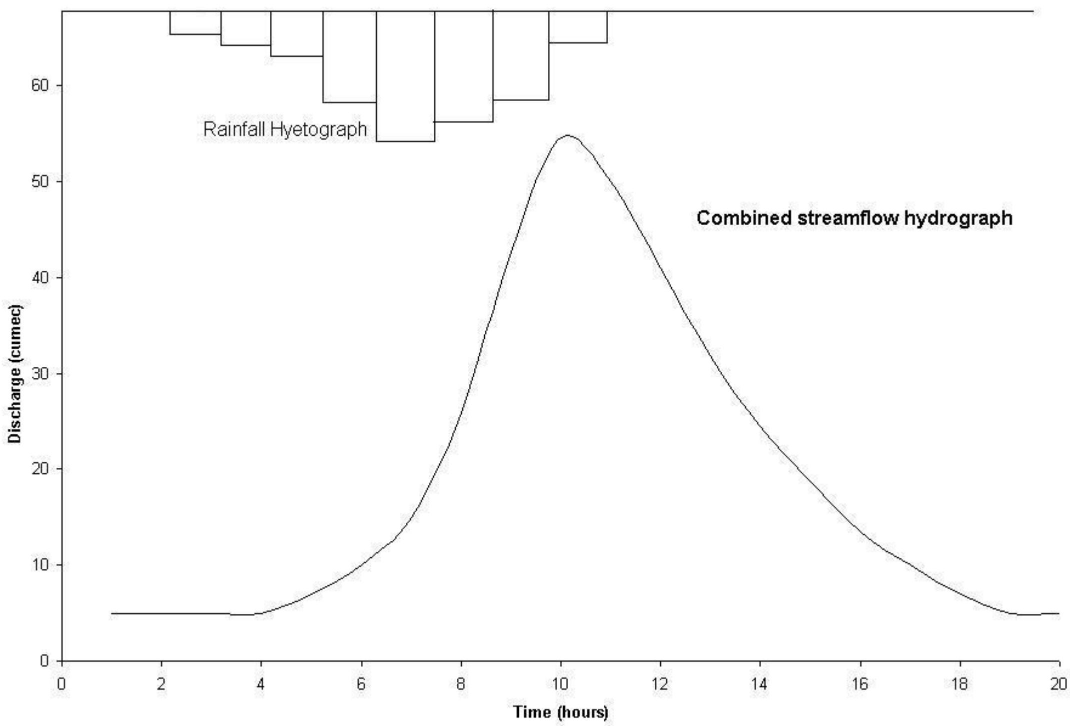


FIGURE 2. Rainfall hyetograph and corresponding flow hydrograph

The combined hydrograph can be split up into two parts: The base flow (Figure 4) and the overland flow added to interflow (Figure 5)

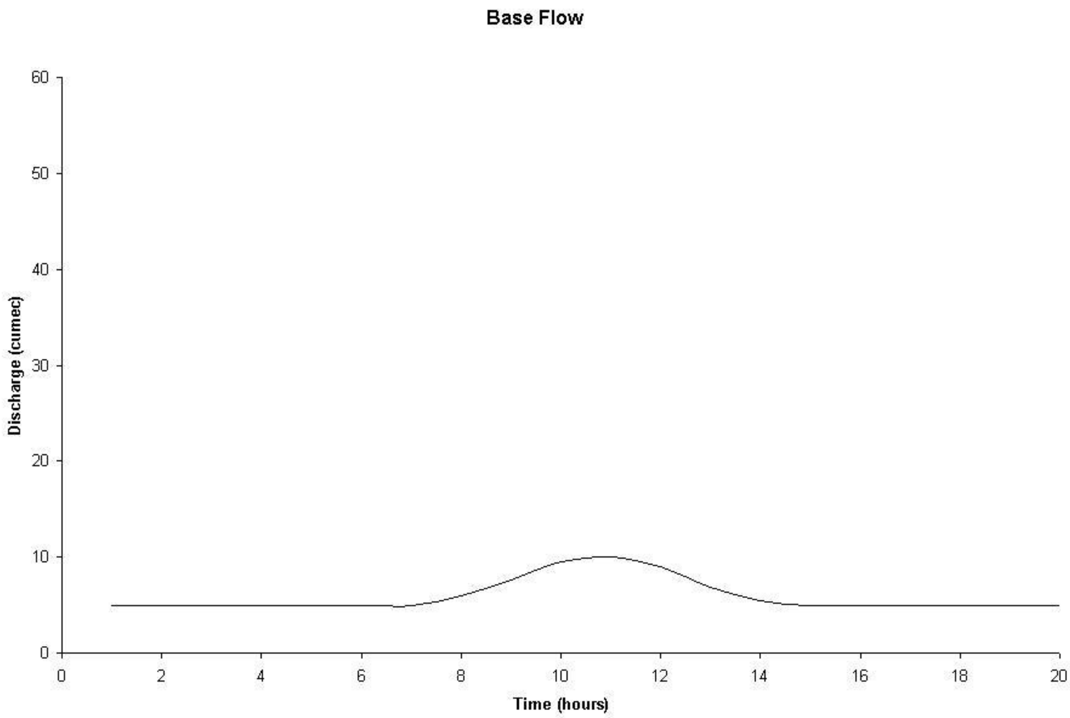


FIGURE 3. Typical baseflow discharge hydrograph

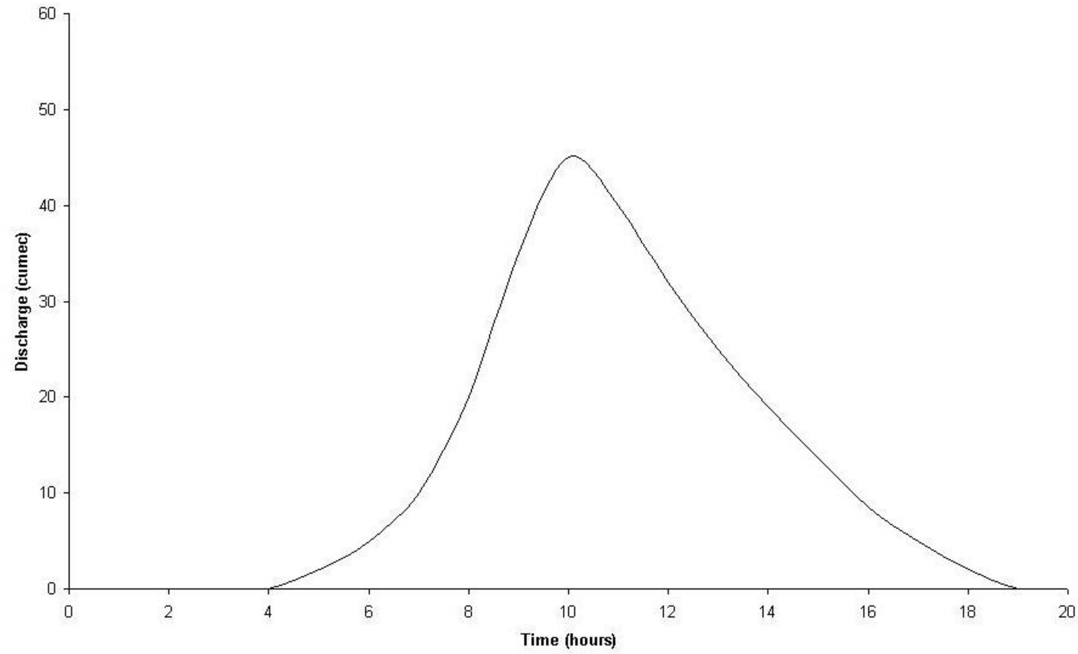


FIGURE 5. Overland flow and interlfow combined hydrogrph

2.2.4 Effective rainfall

A part of the rainfall reaching the earth's surface infiltrates into the ground and finally joins the ground water reservoirs or moves laterally as interflow. Of the interflow, only the quick response or prompt interflow contributes to the immediate rise of the stream flow hydrograph. Hence, the rainfall component causing perceptible change in the stream flow is only a portion of the total rainfall recorded over the catchment. This rainfall is called the effective rainfall.

The infiltration capacity varies from soil to soil and is also different for the same soil in its moist and dry states. If a soil is initially dry, the infiltration rate (or the infiltration capacity of the soil) is high. If the precipitation is lower than the infiltration capacity of the soil, there will be no overland flow, though interflow may still occur. As the rainfall persists, the soil become moist and infiltration rate decreases, causing the balance precipitation to produce surface runoff. Mathematical representation of the infiltration capacity and the methods to deduct infiltration for finding effective rainfall is described later in this lesson.

2.2.5 Methods of base flow separation

Consider the total runoff hydrograph shown in Figure 3, for which the corresponding effective rainfall hyetograph over the catchment is known. In this example, the flow in the stream starts rising at about 4 hours, and the peak is seen to reach at about 10.5 hours. The direct runoff is presumed to end at about 19.5 hours. Though we have separately shown the base flow and the direct runoff in Figures 4 and 5, it is only a guess, as what is observed flowing in the stream is the total discharge. A couple of procedures are explained in the following sub-sections to separate the two flows. For this, we consider another hydrograph (Figure 6), where the total flow is seen to be reducing initially, and then a sudden rise takes place, probably due to a sudden burst of rainfall.

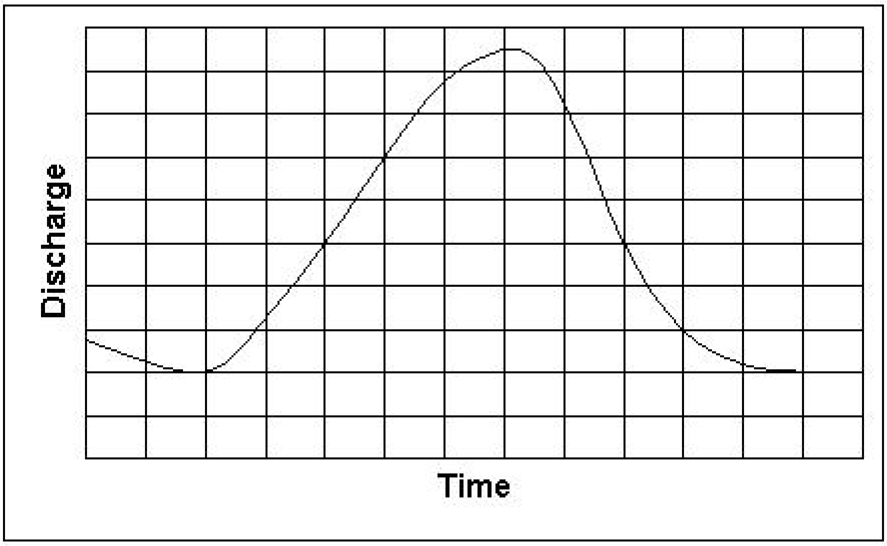


FIGURE 6. A typical hydrograph requiring base flow separation

Method 1

One method to separate the base flow from the total runoff hydrograph is to join points X and Z as shown in Figure 7. This method is considered not very accurate, though.

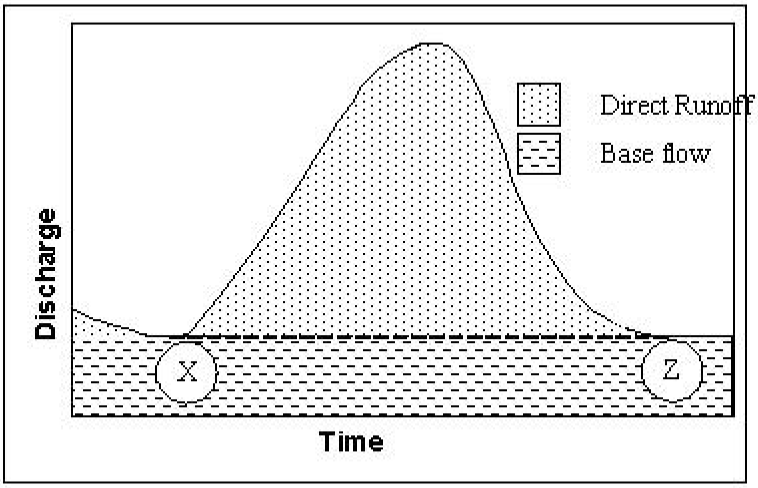


FIGURE 7. Method 1 to separate base flow

Method 2

This method suggests the extension of the base flow graph (Figure 8) along its general trend before the rise of the hydrograph up to a point P directly below the runoff hydrograph peak. From P, a straight line PQ is drawn to meet the hydrograph at point Q, which as separated from P in the time scale by an empirical relation given as:

$$N \text{ (in days)} = 0.862\ A^{0.2} \qquad (1)$$

Where, A is the area of the drainage basin in square kilometers.

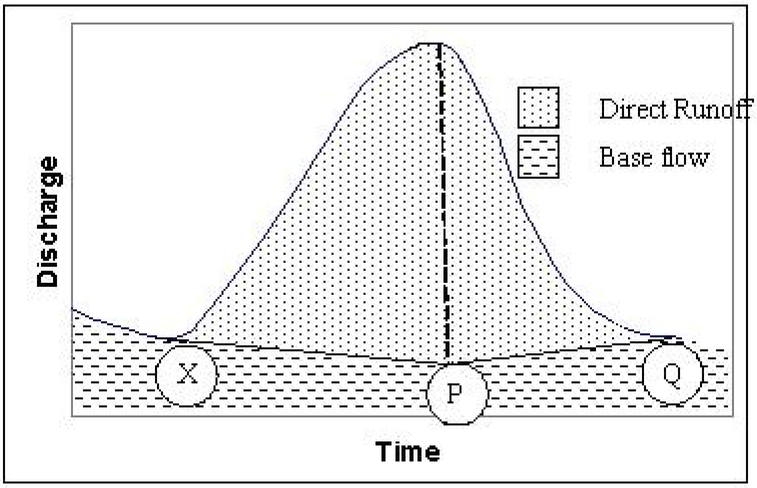


FIGURE 8. Method 2 to separate base flow

Method 3

The third method makes use of composite base flow recession curve, as shown in Figure 9. The following points are to be kept in mind:

X – A follows the trend of the initial base flow recession curve prior to the start of the direct runoff hydrograph
B – Q follows the trend of the later stage base flow recession curve.
B is chosen to lie below the point of inflection (C) of the hydrograph.

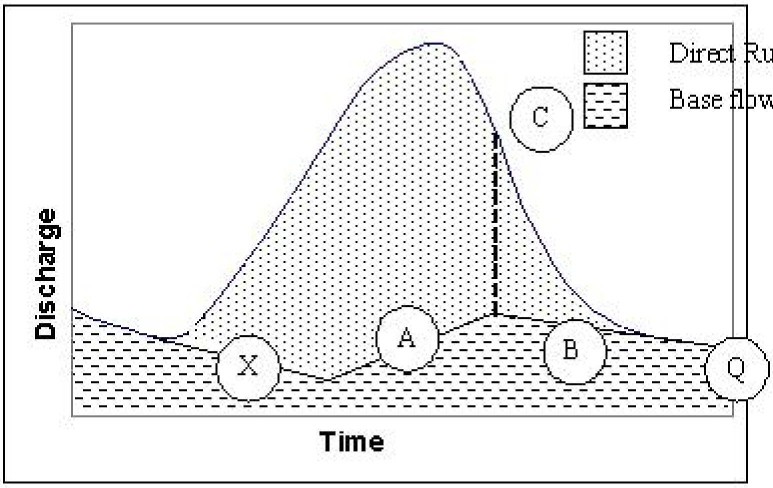


FIGURE 9. Method 3 for base flow separation

The hydrograph after separating and the base flow results in what is called the ***Direct Runoff Hydrograph***.

2.2.6 Estimation of infiltration

The rate at which water infiltrates into a ground is called the ***infiltration capacity.*** When a soil is dry, the infiltration rate is usually high compared to when the soil is moist. For an initially dry soil subjected to rain, the infiltration capacity curve shows an exponentially decaying trend as shown in Figure 10. The observed trend is due to the fact that when the soil is initially dry, the rate of infiltration is high but soon decreases, as most of the soil gets moist. The rate of infiltration reaches a uniform rate after some time.

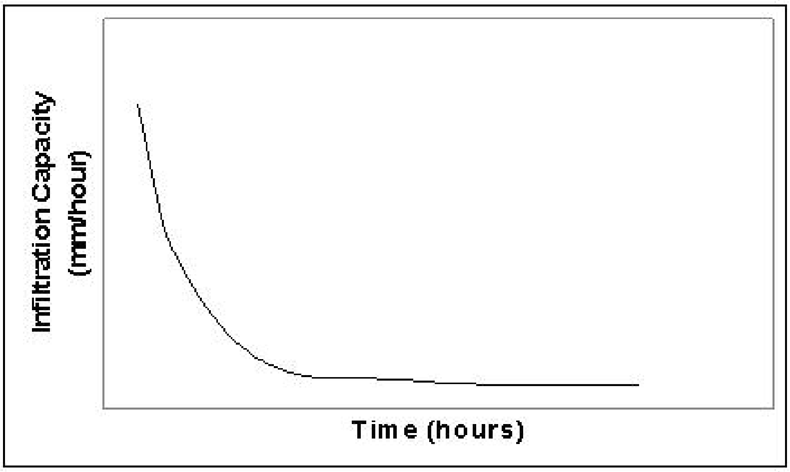


FIGURE 10. Infiltration rate decreasing as more water infiltrates

Interestingly, if the supply of continuous water from the surface is cutoff, then the infiltration capacity starts rising from the point of discontinuity as shown in below.

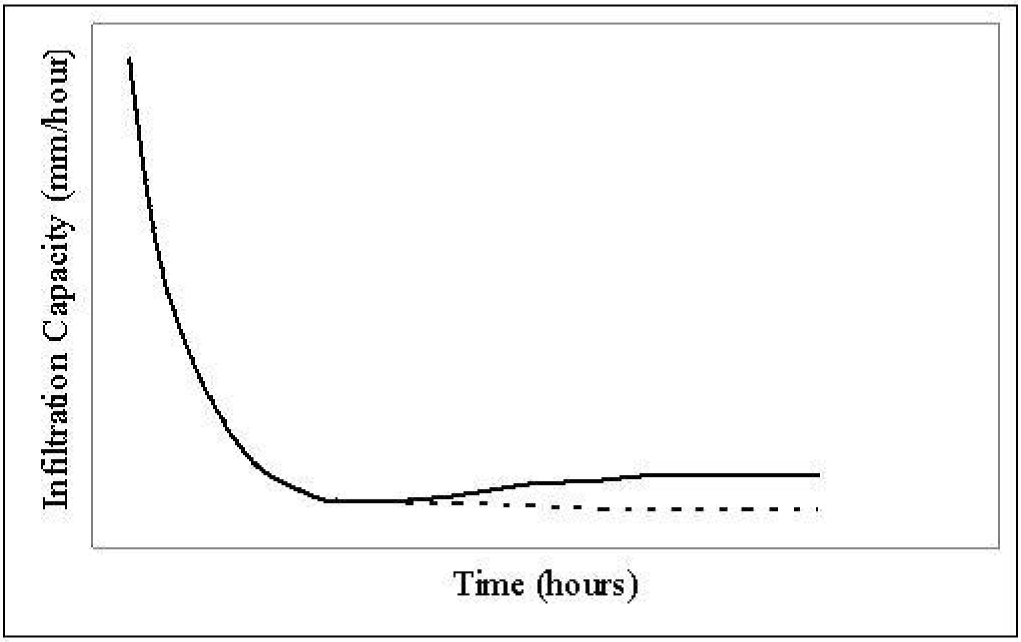


FIGURE 11. Infiltration capacity rising after supply from top is cut off

For consistency in hydrological calculations, a constant value of infiltration rate for the entire storm duration is adopted. The average infiltration rate is called the Infiltration Index and the two types of indices commonly used are explained in the next section.

2.2.7 Infiltration indices

The two commonly used infiltration indices are the following:

ϕ – index

W – index

2.2.7.1 The ϕ - index

This is defined as the rate of infiltration above which the rainfall volume equals runoff volume, as shown in Figure 12.

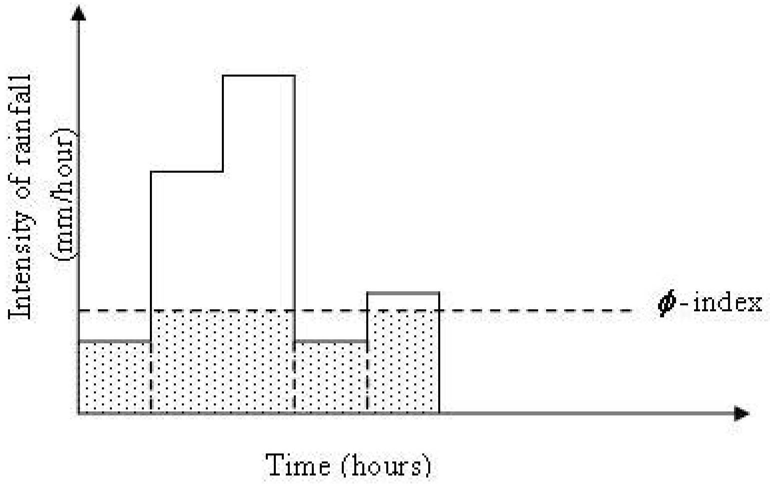


FIGURE 12. Illustrating the ϕ- index

The method to determine the - index would usually involve some trial. Since the infiltration capacity decreases with a prolonged storm, the use of an average loss rate in the form of - index is best suited for design storms occurring on wet soils in which case the loss rate reaches a final constant rate prior to or early in the storm. Although the - index is sometimes criticized as being too simple a measure for infiltration, the concept is quite meaningful in the study of storm runoff from large watersheds. The evaluation of the infiltration process is less precise for large watersheds. The data is never sufficient to derive an infiltration curve. Under the circumstances, the - index is the only feasible alternative to predict the infiltration from the storm.

2.2.7.2 The W – index

This is the average infiltration rate during the time when the rainfall intensity exceeds the infiltration rate.

Thus, ***W*** may be mathematically calculated by dividing the total infiltration (expressed as a depth of water) divided by the time during which the rainfall intensity exceeds the infiltration rate. Total infiltration may be fund out as under:

Total infiltration = Total precipitation – Surface runoff – Effective storm retention

The ***W*** – index can be derived from the observed rainfall and runoff data. It differs from the - index in that it excludes surface storage and retention. The index does not have any real physical significance when computed for a multiple complex watershed. Like the phi-index the - index, too is usually used for large watersheds.

Module
2
The Science of Surface and Ground Water

Lesson 3

Rainfall Runoff Relationships

Instructional Objectives

At the end of this lesson, the student shall learn

1. How hydrograph varies with the catchment characteristics
2. How hydrograph varies with the rainfall characteristics
3. What is Unit Hydrograph, its assumptions and limitations
4. Application of the Unit Hydrograph to find the Direct Runoff Hydrograph
5. What is S – Curve and its applications
6. Derivation of the Unit Hydrograph for gauged catchments
7. How to estimate Unit Hydrograph for ungauged catchments
8. Conceptual and Physically based catchment rainfall – runoff models

2.3.0 Introduction

Lesson 2.2 it was explained what a hydrograph is and that it indicates the response of water flow of a given catchment to a rainfall input. It consists of flow from different phases of runoff, like the overland flow, interflow and base flow. Methods to separate base flow from the total stream flow hydrograph to obtain the direct runoff hydrograph as well as infiltration loss from the total rainfall hyetograph to determine the effective rainfall have been discussed. In this lesson, a relationship between the direct runoff hydrograph of a catchment observed at a location (the catchment outlet) and the effective rainfall over the catchment causing the runoff are proposed to be dealt with.

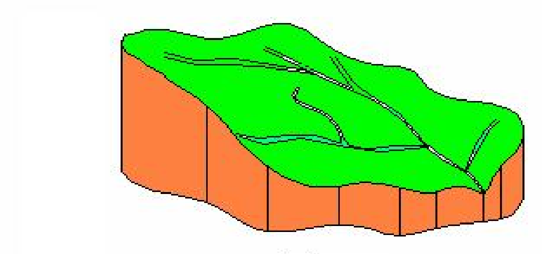
We start with discussing how the various aspects of a catchment's characteristics affects the shape of the hydrograph.

2.3.1 Hydrograph and the catchment's characteristics

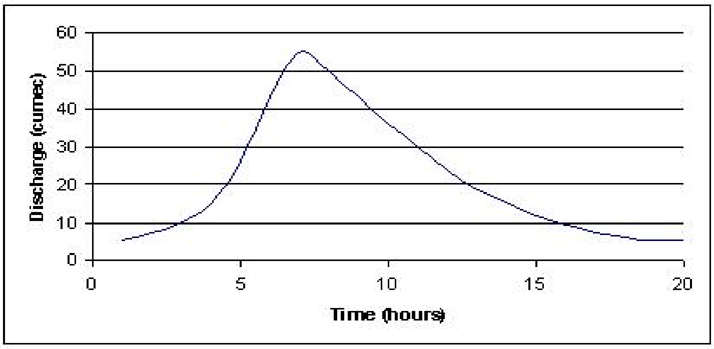
The shape of the hydrograph depends on the characteristics of the catchment. The major factors are listed below.

2.3.1.1 Shape of the catchment

A catchment that is shaped in the form of a pear, with the narrow end towards the upstream and the broader end nearer the catchment outlet (Figure 1a) shall have a hydrograph that is fast rising and has a rather concentrated high peak (Figure 1b).



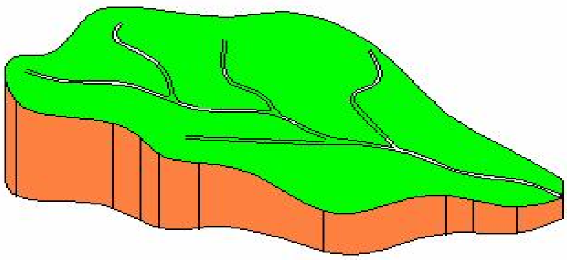
(a)



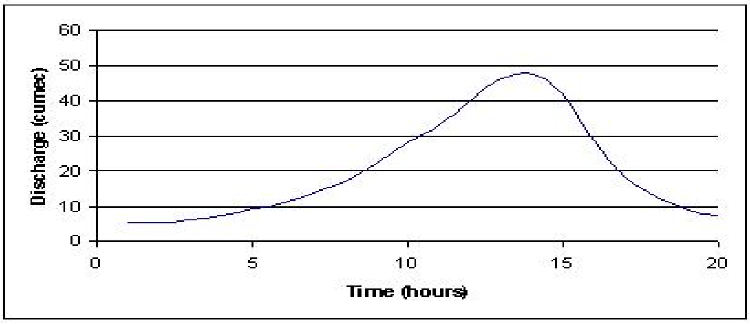
(b)

Figure 1. (a) Pear shaped catchment with narrow end towards upstream and blunt end towards outlet
(b) Corresponding hydrograph for a hypothetical rainfall

A catchment with the same area as in Figure 1 but shaped with its narrow end towards the outlet has a hydrograph that is slow rising and with a somewhat lower peak (Figure 2) for the same amount of rainfall.



(a)



(b)

FIGURE 2. (a) Catchment with narrow end towards outlet
(b) Corresponding hydrograph for a hypothetical rainfall

Though the volume of water that passes through the outlets of both the catchments is same (as areas and effective rainfall have been assumed same for both), the peak in case of the latter is ***attenuated.***

2.3.1.2 Size of the catchment

Naturally, the volume of runoff expected for a given rainfall input would be proportional to the size of the catchment. But this apart, the response characteristics of large catchment (say, a large river basin) is found to be significantly different from a small catchment (like agricultural plot) due to the relative importance of the different phases of runoff (overland flow, inter flow, base flow, etc.) for these two catchments. Further, it can be shown from the mathematical calculations of surface runoff on two impervious catchments (like urban areas, where infiltration becomes negligible), that the non-linearity between rainfall and runoff becomes perceptible for smaller catchments.

2.3.1.3 Slope

Slope of the main stream cutting across the catchment and that of the valley sides or general land slope affects the shape of the hydrograph. Larger slopes generate more velocity than smaller slopes and hence can dispose off runoff faster. Hence, for smaller slopes, the balance between rainfall input and the runoff rate gets stored temporally over the area and is able to drain out gradually over time. Hence, for the same rainfall input to two catchments of the same area but with with different slopes, the one with a steeper slope would generate a hydrograph with steeper rising and falling limits.

Here, two catchments are presented, both with the same are, but with different slopes. A similar amount of rainfall over the flatter catchment (Figure 3) produces a slow-rising moderated hydrograph than that produced by the steeper catchment (Figure 4).

2.3.2 Effect of rainfall intensity and duration on hydrograph

If the rainfall intensity is constant, then the rainfall duration determines in part the peak flow and time period of the surface runoff.

The concept of ***Isochrones*** might be helpful for explaining the effective of the duration of a uniform rainfall on the shape of hydrograph. Isochrones are imaginary lines across the catchment (see Figure 5) from where water particles traveling downward take the same time to reach the catchment outlet.

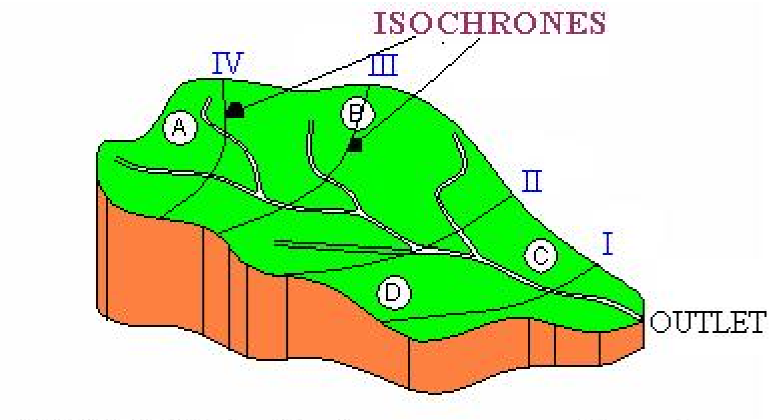


FIGURE 5. Typical isochrones over a catchment

If the rainfall event starts at time zero, then the hydrograph at the catchment outlet will go on rising and after a time'Δt', the flow from the isochrone ***I*** would have reached the catchment outlet. Thus, after a gap of time Δt, all the area A_1 contributes to the outflow hydrograph.

Continuing in this fashion, it can be concluded that after a lapse of time '$4\Delta t$', all the catchment area would be contributing to the catchment outflow, provided the rain continues to fall for atleast up to a time $4\Delta t$. If rainfall continues further, then the hydrograph would not increase further and thus would reach a plateau.

2.3.3 Effect of spatial distribution of rainfall on hydrograph

The effect of spatial distribution of rainfall, that is, the distribution in space, may be explained with the catchment image showing the isochrones as in Figure 6. Assume that the regions between the isochrones receive different amounts of rainfall (shown by the different shades of blue in the figure).

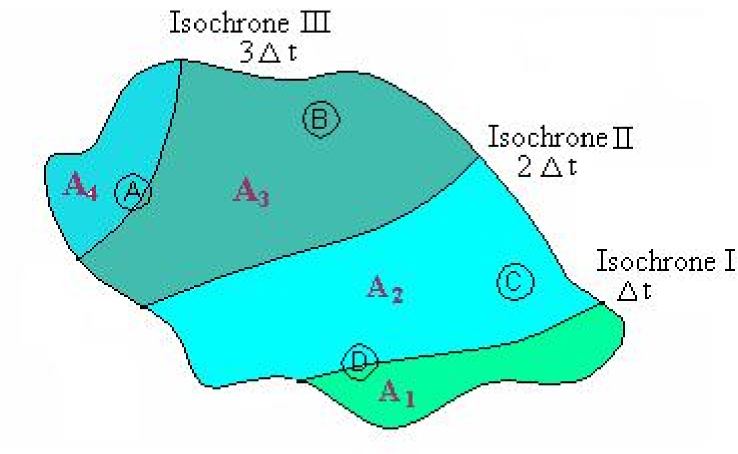


FIGURE 6. Areas of catchment subjected to different amounts of rainfall

If it is assumed now that only area A_1 receives rainfall but the other areas do not, then since this region is nearest to the catchment outlet, the resulting hydrograph immediately rises. If the rainfall continues for a time more than 'Δt', then the hydrograph would reach a saturation equal to $r_e.A_1$, where r_e is the intensity of the effective rainfall.

Assume now that a rainfall of constant intensity is falling only within area A_4, which is farthest from the catchment outlet. Since the lower boundary of A_4 is the Isochrone III, there would be no resulting hydrograph till time '$3\Delta t$'.

If the rain continues beyond a time '$4\Delta t$', then the hydrograph would reach a saturation level equal to r_e A_4 where r_e is the effective rainfall intensity.

2.3.4 Direction of storm movement

The direction of the storm movement with respect to the orientation of the catchments drainage network affects both the magnitude of peak flow and the duration of the hydrograph. The storm direction has the greatest effect on elongated catchments, where storms moving upstream tend to produce lower peaks and broader time base of surface runoff than storms that move downstream towards the catchment outlet. This is due to the fact that for an upstream moving storm, by the time the contribution from the upper catchment reaches the outlet, there is almost no contribution from the lower watershed.

2.3.5 Rainfall intensity

Increase in rainfall intensity increases the peak discharge and volume of runoff for a given infiltration rate. In the initial phases of the storm, when the soil is dry, a rainfall intensity less than infiltration rate produces no surface runoff. Gradually, as the rain progresses, the soil saturates and the infiltration rate reduces to a steady rate.

The relation between rainfall intensity and the discharge, strictly speaking, is not linear, which means that doubling the rainfall intensity does not produce a doubling of the hydrograph peak value. However, this phenomenon is more pronounced for small watersheds, such as an urban area. However in the catchment scale, due to the uncertainty of all the hydrological parameters, it might be assumed that the rainfall runoff relation follows a linear relationship. This assumption is made use of in the unit hydrograph concept, which is explained in the next section.

2.3.6 The Unit Hydrograph

The Unit Hydrograph (abbreviated as UH) of a drainage basin is defined as a hydrograph of direct runoff resulting from one unit of effective rainfall which is uniformly distributed over the basin at a uniform rate during the specified period of time known as unit time or unit duration. The unit quantity of effective rainfall is generally taken as 1mm or 1cm and the outflow hydrograph is expressed by the discharge ordinates. The unit duration may be 1 hour, 2 hour, 3 hours or so depending upon the size of the catchment and storm characteristics. However, the unit duration cannot be more than the time of concentration, which is the time that is taken by the water from the furthest point of the catchment to reach the outlet.

Figure 7 shows a typical unit hydrograph.

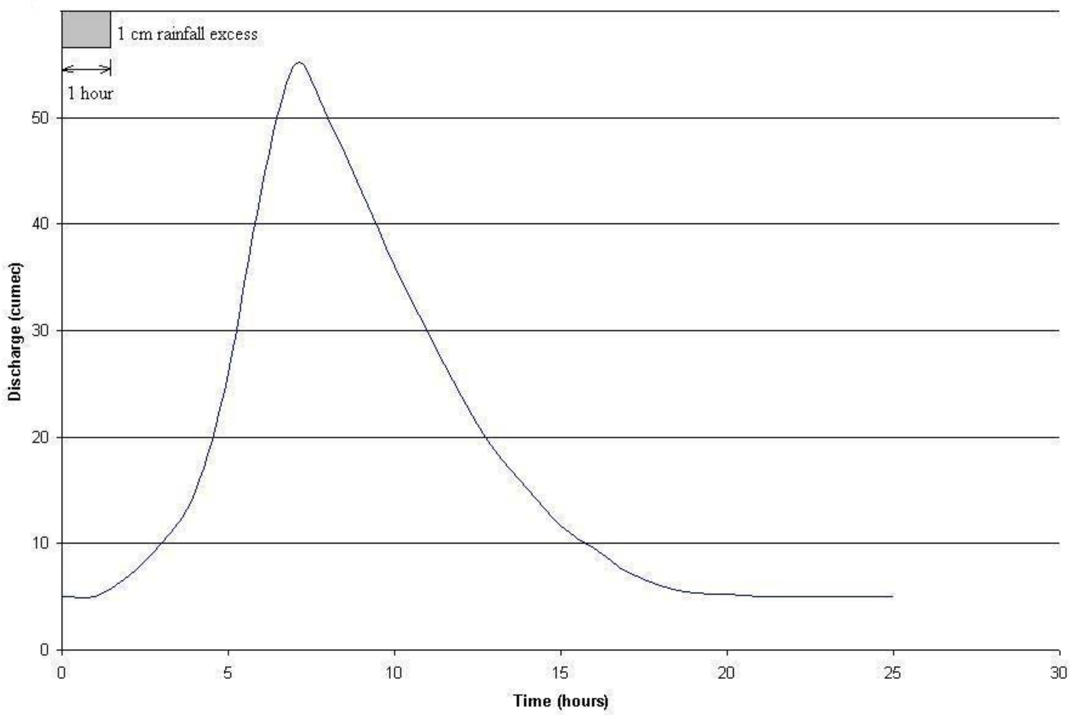


FIGURE 7. A typical unit hydrograph

2.3.6.1 *Unit hydrograph assumptions*

The following assumptions are made while using the unit hydrograph principle:

1. Effective rainfall should be uniformly distributed over the basin, that is, if there are 'N' rain gauges spread uniformly over the basin, then all the gauges should record almost same amount of rainfall during the specified time.

2. Effective rainfall is constant over the catchment during the unit time.

3. The direct runoff hydrograph for a given effective rainfall for a catchment is always the same irrespective of when it occurs. Hence, any previous rainfall event is not considered. This antecedent precipitation is otherwise important because of its effect on soil-infiltration rate, depressional and detention storage, and hence, on the resultant hydrograph.

4. The ordinates of the unit hydrograph are directly proportional to the effective rainfall hyetograph ordinate. Hence, if a 6-h unit hydrograph due to 1 cm rainfall is given, then a 6-h hydrograph due to 2 cm rainfall would just mean doubling the unit hydrograph ordinates. Hence, the base of the resulting hydrograph (from the start or rise up to the time when discharge becomes zero) also remains the same.

2.3.6.2 *Unit hydrograph limitations*

Under the natural conditions of rainfall over drainage basins, the assumptions of the unit hydrograph cannot be satisfied perfectly. However, when the hydrologic data used in the unit hydrograph analysis are carefully selected so that they meet the assumptions closely, the results obtained by the unit hydrograph theory have been found acceptable for all practical purposes.

In theory, the principle of unit hydrograph is applicable to a basin of any size. However, in practice, to meet the basic assumption in the derivation of the unit hydrograph as closely as possible, it is essential to use storms which are uniformly distributed over the basin and producing rainfall excess at uniform rate. Such storms rarely occur over large areas. The size of the catchment is, therefore, limited although detention, valley storage, and infiltration all tend to minimize the effect of rainfall variability. The limit is generally considered to be about 5000 sq. km. beyond which the reliability of the unit hydrograph method diminishes. When the basin area exceeds this limit, it has to be divided into sub-basins and the unit hydrograph is developed for each sub-basin. The flood discharge at the basin outlet is then estimated by combining the sub-basin floods, using ***flood routing*** procedures.

Note:
Flood Routing: This term is used to denote the computation principles for estimating the values of flood discharge with time and in space, that is, along the length of a river. Details about flood routing procedures may be had from the following book:
M H Chaudhry (1993) Open channel hydraulics, Prentice Hall of India

2.3.7 Application of the unit hydrograph

Calculations of direct runoff hydrograph in catchment due to a given rainfall event (with recorded rainfall values), is easy if a unit hydrograph is readily available. Remember that a unit hydrograph is constructed for a unit rainfall falling for a certain T-hours, where T may be any conveniently chosen time duration. The effective rainfall hyetograph, for which the runoff is to be calculated using the unit hydrograph, is obtained by deducting initial and infiltration losses from the recorded rainfall. This effective rainfall hyetograph is divided into blocks of T-hour duration. The runoff generated by the effective rainfall for each T-hour duration is then obtained and summed up to produce the runoff due to the total duration.

2.3.8 Direct runoff calculations using unit hydrograph

Assume that a 6-hour unit hydrograph (UH) of a catchment has been derived, whose ordinates are given in the following table and a corresponding graphical representation is shown in Figure 8.

Time (hours)	Discharge (m^3/s)
0	0
6	5
12	15
18	50
24	120
30	201
36	173
42	130
48	97
54	66
60	40
66	21
72	9
78	3.5
84	2

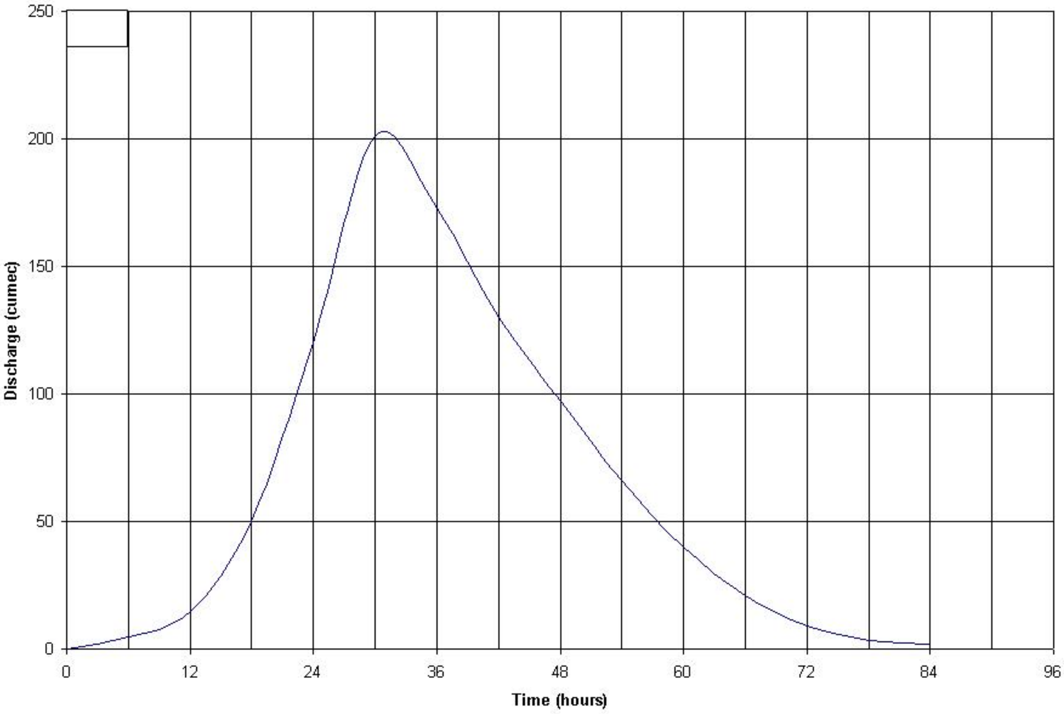


FIGURE 8. A 6-hour unit hydrograph

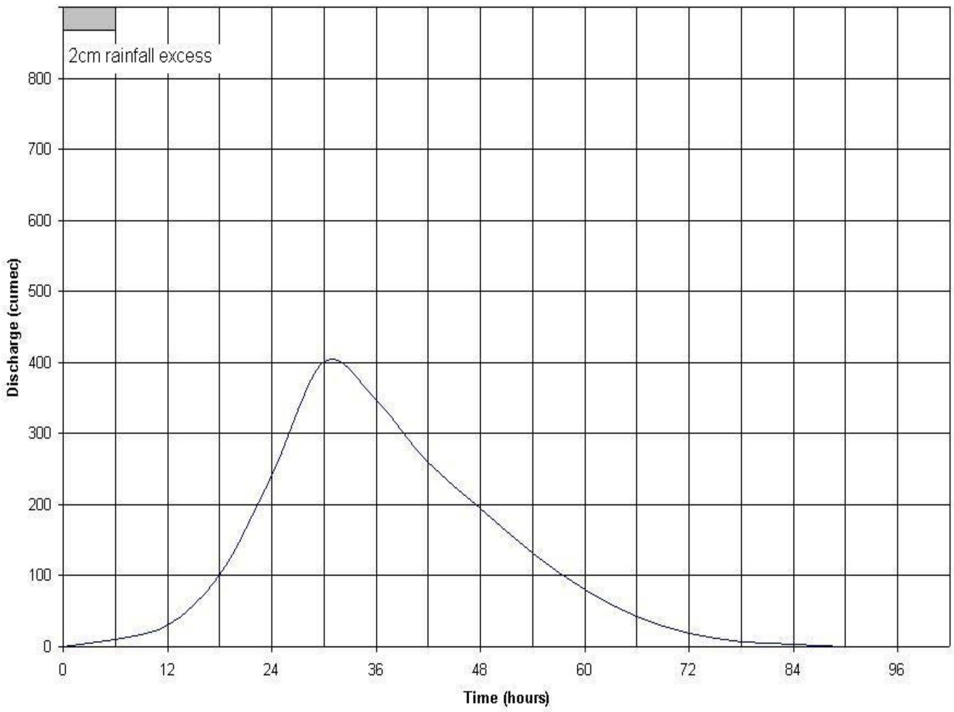
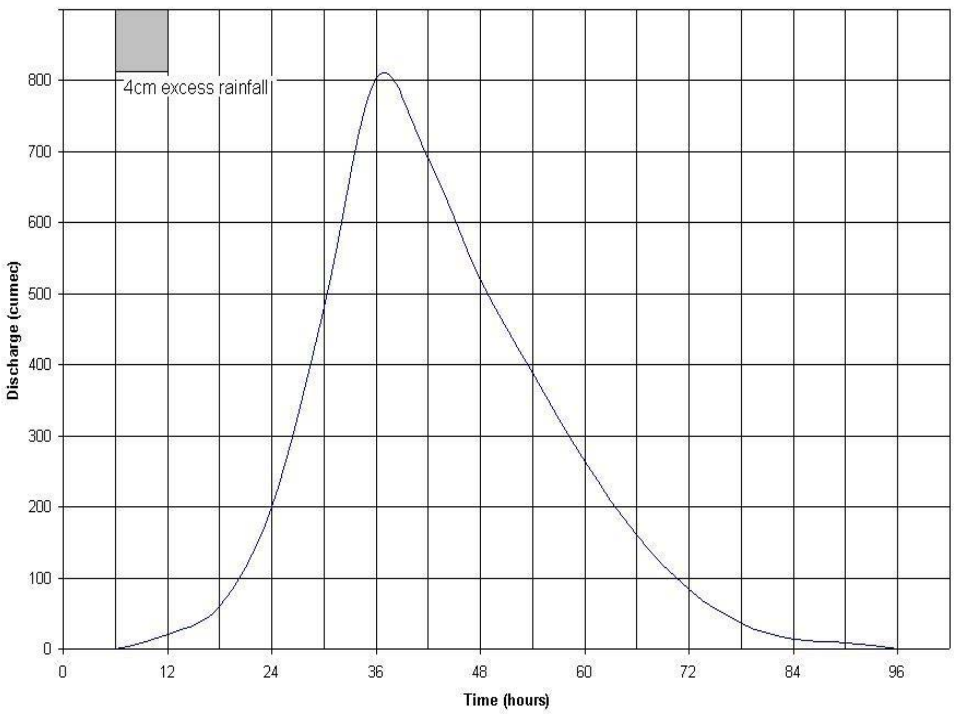
Assume further that the effective rainfall hyetograph (ERH) for a given storm on the region has been given as in the following table:

Time (hours)	Effective Rainfall (cm)
0	0
6	2
12	4
18	3

This means that in the first 6 hours, 2cm excess rainfall has been recorded, 4cm in the next 6 hours, and 3cm in the next.

The direct runoff hydrograph can then be calculated by the three separate hyetographs for the three excess rainfalls by multiplying the ordinates of the hydrograph by the corresponding rainfall amounts. Since the rainfalls of 2cm, 4cm and 3cm occur in successive 6-hour intervals, the derived DRH corresponding to each rainfall is delayed by 6 hours appropriately.

These have been shown in the figures indicated.

DRH for 2cm excess rainfall in 0-6 hours	 FIGURE 9. DRH corresponding to 2cm excess rainfall in 0 - 6 hours
DRH for 4cm excess rainfall in 6-12 hours	 FIGURE 10. DRH corresponding to 4cm excess rainfall during 6 - 12 hours

DRH for 3cm excess rainfall in 12-18 hours

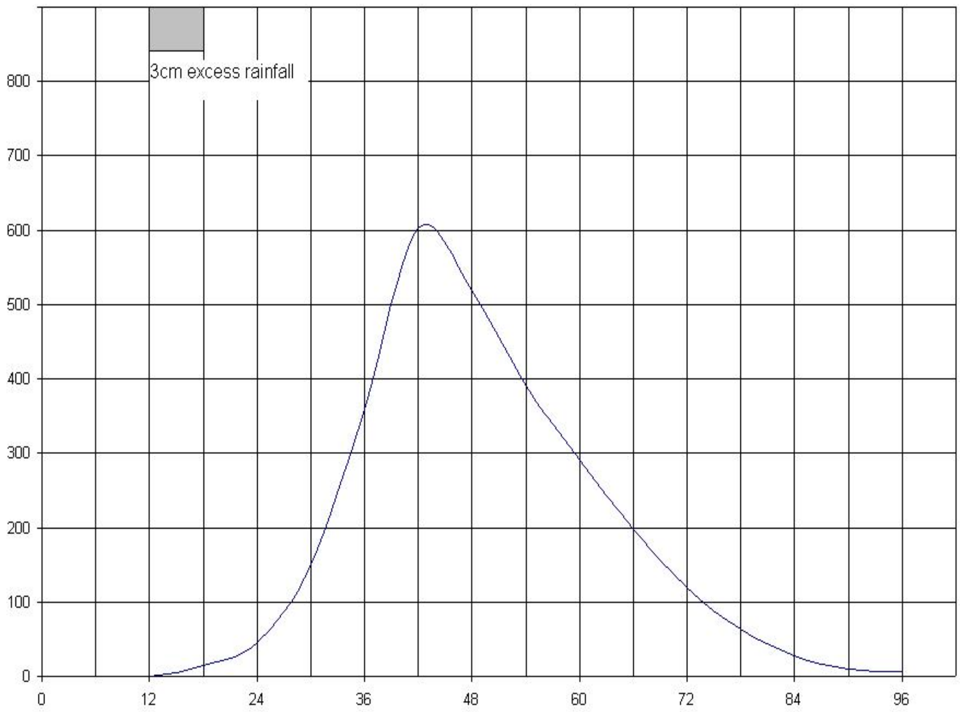


FIGURE 11. DRH corresponding to 3cm excess rainfall during 12 - 18 hours

The final hydrograph is found out by adding the three individual hydrographs, as shown in Figure 12.

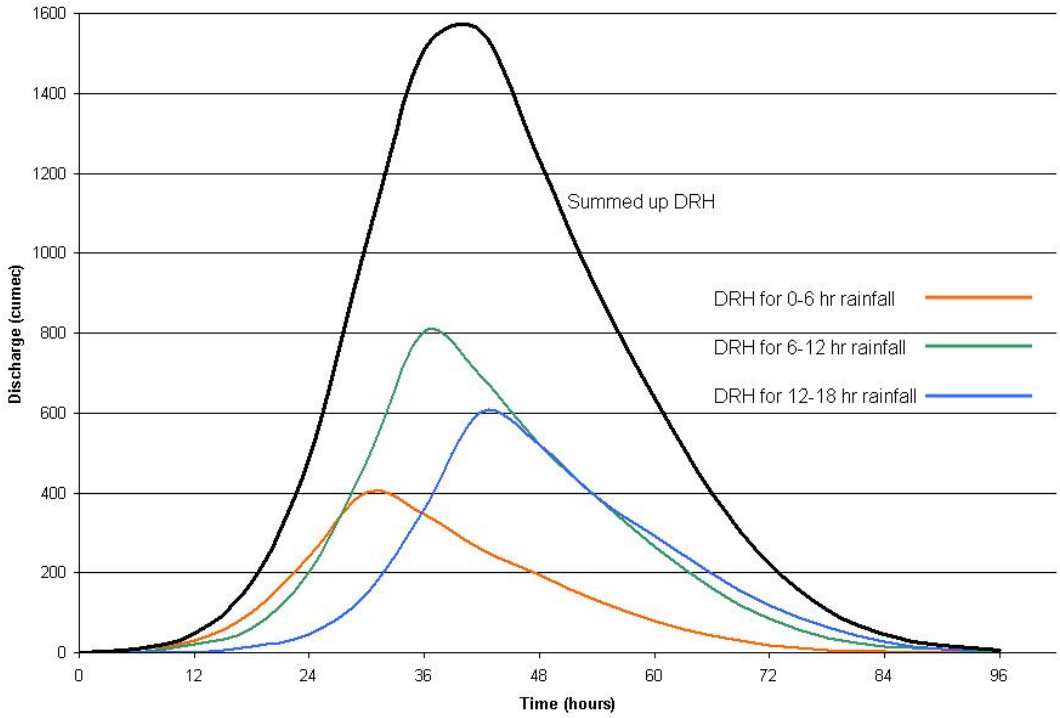


FIGURE 12. Final direct runoff hydrograph derived from summation of individual DRHs

The calculations to generate the direct runoff hydrograph (DRH) from a given UH and ERH can be conveniently done using a spreadsheet program, like the Microsoft XL.

A sample calculation for the example solved graphically is given in the following table. Note the 6 hour shift of the DRHs in the second and subsequent hours.

Time (hours)	Unit Hydrograph ordinates (m^3/s)	Direct runoff due to 2 cm excess rainfall in first 6 hours (m^3/s) (I)	Direct runoff due to 4 cm excess rainfall in second 6 hours (m^3/s) (II)	Direct runoff due to 3 cm excess rainfall in third 6 hours (m^3/s) (III)	Direct runoff Hydrograph (m^3/s) (I)+(II)+(III)
0	0	0	0	0	0
6	5	10	0	0	10
12	15	30	20	0	50
18	50	100	60	15	175
24	120	240	200	45	485
30	201	402	480	150	1032
36	173	346	804	360	1510
42	130	260	692	603	1555
48	97	194	520	519	1233
54	66	132	388	390	910
60	40	80	264	291	635
66	21	42	160	198	400
72	9	18	84	120	222
78	3.5	7	36	63	106
84	2	4	14	27	45
90		0	8	10.5	18.5
96		0	0	6	6

The last column in the above table gives the ordinates of the DRH produced by the ERH. If the base flow is known or estimated (Lesson 2.2), then this should be added to the DRH to obtain the 6-houly ordinates of the flood hydrograph.

2.3.9 The S – curve

This is a concept of the application of a hypothetical storm of 1 cm ERH of infinite duration spread over the entire catchment uniformly. This may be done by shifting the UH by the T-duration for a large number of periods. The resulting hydrograph (a typical one is shown in Figure 13) is called the S – hydrograph, or the S – curve due to the summation of an infinite series of T-hour unit hydrographs spaced T – hour apart. For the example of the UH given in the earlier section, the table below provides the necessary calculations.

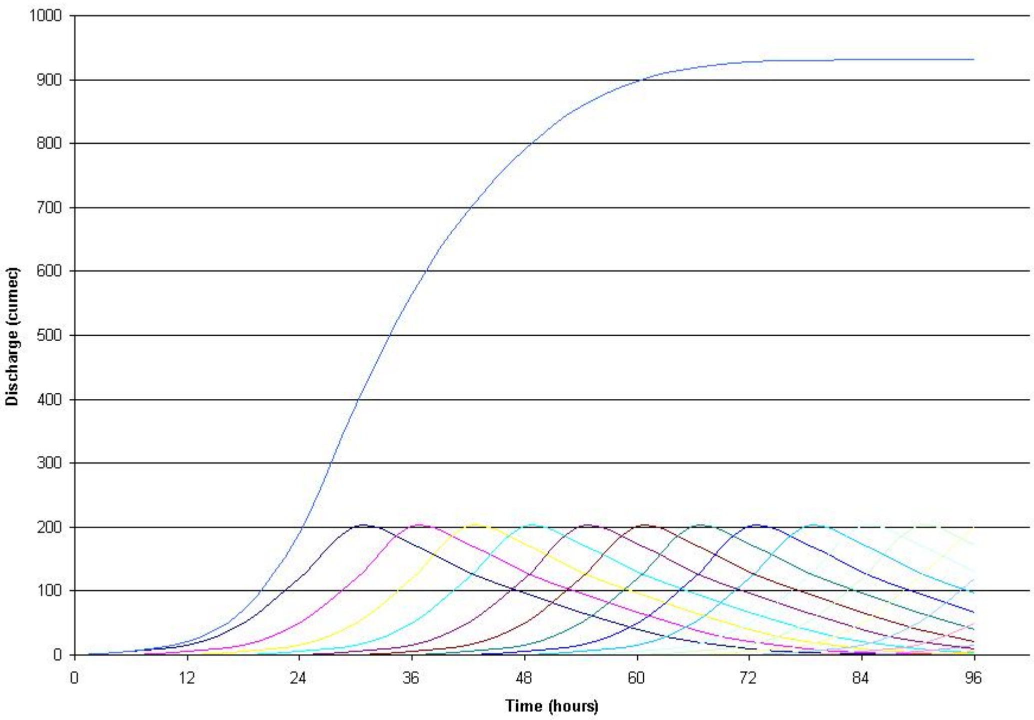


FIGURE 13. S - Curve, or Summed up Unit Hydrographs

Time (hr)	UH Ordi-Nates	UH Ordi-nates shifted by 6 hr	UH Ordi-nates shifted by 12 hr	UH Ordi-nates shifted by 18 hr	UH Ordi-nates shifted by 24 hr	…	…	…	…	…	…	…	…	…	…	…	Sum of all the UH ordi-nates
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
6	5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	5
12	15	5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	20
18	50	15	5	0	0	0	0	0	0	0	0	0	0	0	0	0	70
24	120	50	15	5	0	0	0	0	0	0	0	0	0	0	0	0	190
30	201	120	50	15	5	0	0	0	0	0	0	0	0	0	0	0	391
36	173	201	120	50	15	5	0	0	0	0	0	0	0	0	0	0	564
42	130	173	201	120	50	15	5	0	0	0	0	0	0	0	0	0	694
48	97	130	173	201	120	50	15	5	0	0	0	0	0	0	0	0	791
54	66	97	130	173	201	120	50	15	5	0	0	0	0	0	0	0	857
60	40	66	97	130	173	201	120	50	15	5	0	0	0	0	0	0	897
66	21	40	66	97	130	173	201	120	50	15	5	0	0	0	0	0	918
72	9	21	40	66	97	130	173	201	120	50	15	5	0	0	0	0	927
78	3.5	9	21	40	66	97	130	173	201	120	50	15	5	0	0	0	930.5
84	2	3.5	9	21	40	66	97	130	173	201	120	50	15	5	0	0	932.5
90	0	2	3.5	9	21	40	66	97	130	173	201	120	50	15	5	0	932.5
96	0	0	2	3.5	9	21	40	66	97	130	173	201	120	50	15	5	932.5

The average intensity of the effective rainfall producing the S – curve is 1/T (mm/h) and the equilibrium discharge is given as $(\frac{A}{T}X10^4)m^3/h$ where, A is the area of the catchment in Km^2 and T is the unit hydrograph duration in hours.

2.3.10 Application of the S – curve

Though the ***S – curve*** is a theoretical concept, it is an effective tool to derive a ***t – hour*** UH from a ***T – hour*** UH, when ***t*** is smaller that T or ***t*** is lager than T but not an exact multiple of T. In case ***t*** is a multiple of T, the corresponding UH can be obtained without the aid of a S – hydrograph by summing up the required number of UH, lagged behind by consecutive T – hours.

For all other cases shift the original S – hydrograph as derived for the T – hour UH by ***t*** hours to obtain a lagged S- hydrograph. Subtract the ordinates of the second curve from the first to obtain the ***t*** – hour graph. Next, scale the ordinates of the discharge hydrograph by a factor ***t/T***, to obtain the actual ***t*** – hour UH which would result due to a total 1 cm of rainfall over the catchment. This is illustrated by the S-curve derived in the previous section.

Recall that the S-curve was obtained from a 6-hour UH. Let us derive the UH for a 3-hour duration. Since we do not know the ordinates of the S-curve at every 3-hour interval, we interpolate and write them in a tabular form as given in the table below:

Time	S-curve ordinates as derived from 6-hr UH (I)	S-curve ordinates as derived from 6-hr UH but with inter-polated values (II)	S-curve ordinates as derived from 6-hr UH lagged by 3 hrs. (III)	Difference of the two S-curves (II) – (III) (IV)	3-hr UH ordinates Col. (IV) divided by (3hr/6hr) = (IV)*2
(hours)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)
0	0	0		0	
3		2.5	0	2.5	
6	5	5	2.5	2.5	
9		12.5	5	7.5	
12	20	20	12.5	7.5	
15		45	20	25	
18	70	70	45	25	
21		130	70	60	
24	190	190	130	60	
27		290.5	190	100.5	
30	391	391	290.5	100.5	

33		477.5	391	86.5	
36	564	564	477.5	86.5	
39		629	564	65	
42	694	694	629	65	
45		742.5	694	48.5	
48	791	791	742.5	48.5	
51		824	791	33	
54	857	857	824	33	
57		877	857	20	
60	897	897	877	20	
63		907.5	897	10.5	
66	918	918	907.5	10.5	
69		922.5	918	4.5	
72	927	927	922.5	4.5	
75		928.75	927	1.75	
78	930.5	930.5	928.75	1.75	
81		931.35	930.5	0.85	
84	932.5	932.5	931.35	1.15	
87		932.5	932.5	0	
90	932.5	932.5	932.5	0	
93		932.5	932.5	0	
96	932.5	932.5	932.5	0	

2.3.11 Derivation of unit hydrograph

An observed flood hydrograph at a streamflow gauging station could be a hydrograph resulting from an isolated intense short – duration storm of nearly uniform distribution in time and space, or it could be due to a complex rainfall event of varying intensities. In the former case, the observed hydrograph would mostly be single peaked whereas for the latter, the hydrograph could be multi peaked depending on the variation in the rainfall intensities. For the purpose of this course, we shall only consider rainfall to be more or less uniformly distributed in time and space for the purpose of demonstrating the derivation of unit hydrograph. The procedure may be broadly divided into the following steps:

1. Obtain as many rainfall records as possible for the study area to ensure that the amount and distribution of rainfall over the watershed is accurately known. Only those storms which are isolated events and with uniform spatial and temporal distribution are selected along with the observed hydrograph at the watershed outlet point.

2. Storms meeting the following criteria are generally preferred and selected out of the uniform storms data collected in Step 1.

3. Storms with rainfall duration of around 20 to 30 % of ***basin lag***,

4. Storms having rainfall excess between 1 cm and 4.5 cm.

5. From the observed total flood hydrograph for each storm separate the base flow (discussed in lecture 2.2) and plot the direct runoff hydrograph.

6. Measure the total volume of water that has passed the flow measuring point by finding the area under the DRH curve. Since area of the watershed under consideration is known, calculate the average uniform rainfall depth that produced the DRH by dividing the volume of flow (step 3) by the catchment area. This gives the effective rainfall (ER) corresponding to the storm. This procedure has to be repeated for each selected storm to obtain the respective ERs.

7. Express the hydrograph ordinate for each storm at T – hour is the duration of rainfall even. Divide each ordinate of the hydrograph by the respective storm ER to obtain the UH corresponding to each storm.

8. All UHs obtained from different storm events should be brought to the same duration by the S – curve method.

9. The final UH of specific duration is obtained by averaging the ordinates of he different UH obtained from step 6.

2.3.12 Unit hydrograph for ungauged catchments

For catchments with insufficient rainfall or corresponding concurrent runoff data, it is necessary to develop synthetic unit hydrograph. These are unit hydrographs constructed form basin characteristics. A number of methods like that of Snyder's had been used for the derivation of the Synthetic hydrographs. However, the present recommendations of the Central Water Commission discourage the use of the Snyder's method.

Instead, the Commission recommends the use of the Flood Estimation Reports brought out for the various ***sub–zones*** in deriving the unit hydrograph for the region. These sub–zones have been demarcated on the basis of similar hydro – meteorological conditions and a list of the basins may be found. The design flood is estimated by application of the design storm rainfall to the synthetic hydrograph developed by the methods outlined in the reports.

2.3.13 Catchment modelling

With the availability of personal computer high processing speed within easy reach of all, it is natural that efforts have been directed towards numerical modeling the catchment dynamics and its simulation. It is not possible to outline each model in detail, but the general concept followed is to represent each physical process by a conceptual mathematical model which can be represented by an equivalent differential or ordinary equation. These equations are solved by changing the equations to solvable form and writing algorithms in suitable computer language. However, the user of the programs generally input data through a ***Graphical User Interface*** (GUI) since there is a lot of spatial information to be included like land-use, land-cover, soil property, etc. Now a day, this information interaction between the user and the computer is through ***Geographic Information System*** (GIS) software. Once the information is processed, the output results are also displayed graphically. A list of notable conceptual models may be found in the following websites:

- http://www.nrc.gov/reading-rm/doc-collections/nuregs/contract/cr6656/cr6656.pdf
- www.hydrocomp.com/simoverview.html
- http://www.ems-i.com/gmshelp/numerical_models/modflow/modflow_conceptual_model/the_conceptual_model_approach.htm

2.3.14 Examples of catchment models

Though many of these models are sold commercially, there are quite a few developed by academic institutions and government agencies worldwide which are free and can be downloaded for non – commercial purposes through the internet. A few examples are given below.

- US Army corps of Engineers' ***HEC-HMS*** and ***HEC-GeoHMS***
- US Army corps of Engineers' ***GRASS***
- US Army corps of Engineers' ***TOPMODEL***

Water resources section of the Department of Civil Engineering, IIT Kharagpur has developed a watershed simulation model based on deterministic theory. A copy of the same may be made available on request for educational purposes.

2.3.15 Important terms

1. *Linearity*: A linear relation between rainfall and runoff form a catchment suggests that variations in rainfall over a catchment is related to the variations in runoff from the outlet of the catchment by a linear function.

2. *Basin lag*: Basin lag is the time between the peak flow and the centroid of rainfall.

3. *Graphical User Interface* (GUI): An interface that represents programs, files, and options with graphical images is called GUI. These images can include icons, menus, and dialog boxes. The user selects and activates these options by pointing and clicking with a mouse or with the keyboard. A particular GUI item (for example, a scroll bar) works the same way in all applications.

4. *Geographic Information System (GIS):* A system, usually computer based, for the input, storage, retrieval, analysis and display of interpreted geographic data. The database is typically composed of map-like spatial representations, often called coverages or layers. These layers may involve a three dimensional matrix of time, location, and attribute or activity. A GIS may include digital line graph (DLG) data, Digital Elevation Models (DEM), geographic names, land-use characterizations, land ownership, land cover, registered satellite and/or areal photography along with any other associated or derived geographic data.

5. *HEC-HMS*: The Hydrologic Modeling System (HEC-HMS) is designed to simulate the precipitation-runoff processes of dendritic watershed systems. It is designed to be applicable in a wide range of geographic areas for solving the widest possible range of problems. This includes large river basin water supply and flood hydrology, and small urban or natural watershed runoff. Hydrographs produced by the program are used directly or in conjunction with other software for studies of water availability, urban drainage, flow forecasting, future urbanization impact, reservoir spillway design, flood damage reduction, floodplain regulation, and systems operation.

6. *HEC-GeoHMS*: The Geospatial Hydrologic Modeling Extension (HEC-GeoHMS) is a public-domain software package for use with the ArcView Geographic Information System. GeoHMS uses ArcView and Spatial Analyst to develop a number of hydrologic modeling inputs. Analyzing the digital terrain information, HEC-GeoHMS transforms the drainage paths and watershed boundaries into a hydrologic data structure that represents the watershed response to precipitation. In addition to the hydrologic data structure, capabilities include the development of grid-based data for linear quasi-distributed runoff transformation (ModClark), HEC-HMS basin model, physical watershed and stream characteristics, and background map file.

7. *GRASS*: GRASS is an integrated set of programs designed to provide digitizing, image processing, map production, and geographic information system capabilities to its users. GRASS is open software with freely available source code written in C.

***8 Topmodel*:** TOPMODEL predicts catchment water discharge and spatial soil water saturation pattern based on precipitation and evapotranspiration time series and topographic information.

References

- Subramanya, K (2000) Engineering Hydrology, Tata Mc Graw Hill
- Mutreja, K N (1995) Applied Hydrology, Tata Mc Graw Hill

Module 2

The Science of Surface and Ground Water

Lesson
4
Design Flood Estimation

Instructional Objectives

The student, after completion of this lesson, shall know:

1. How is a design flood described for a particular hydraulic structure?
2. How are design floods designated for: a) Storage Dams; b) Barrages and weirs; c) Diversion works and Coffer dams; and d) Cross drainage works, according to the Indian Standard guidelines.
3. What are the methods to calculate design floods, viz, a) The hydro-meteorological approach, and b) The statistical approach
4. The steps followed in finding out design flood by the hydro-meteorological approach, which is generally adopted for large and intermediate sized dams.
5. The steps followed in finding out design flood by the statistical approach, either by probability distributions or by the method of plotting positions.

2.4.0 Introduction

A flood is commonly considered to be an unusually high stage of a river. For a river in its natural state, occurrence of a flood usually fills up the stream up to its banks and often spills over to the adjoining flood plains. For a hydraulic structure planned within the river (like a dam or a barrage) or on an adjoining area (like flood control embankments), due consideration should be given to the design of the structure so as to prevent it from collapsing and causing further damage by the force of water released from behind the structure. Hence an estimate of extreme flood flow is required for the design of hydraulic structures, though the magnitude of such flood may be estimated in accordance with the importance of the structure. For example, the design flood for a large dam like the ***Bhakra*** (Figure 1) or the ***Hirakud*** (Figure 2) would be estimated to be more than a medium sized dam like ***Chamera*** (Figure 3). It must be remembered that proper selection of design flood value is of great importance. While a higher value would result in an increase in the cost of hydraulic structures, an under-estimated value is likely to place the structure and population involved at some risk.

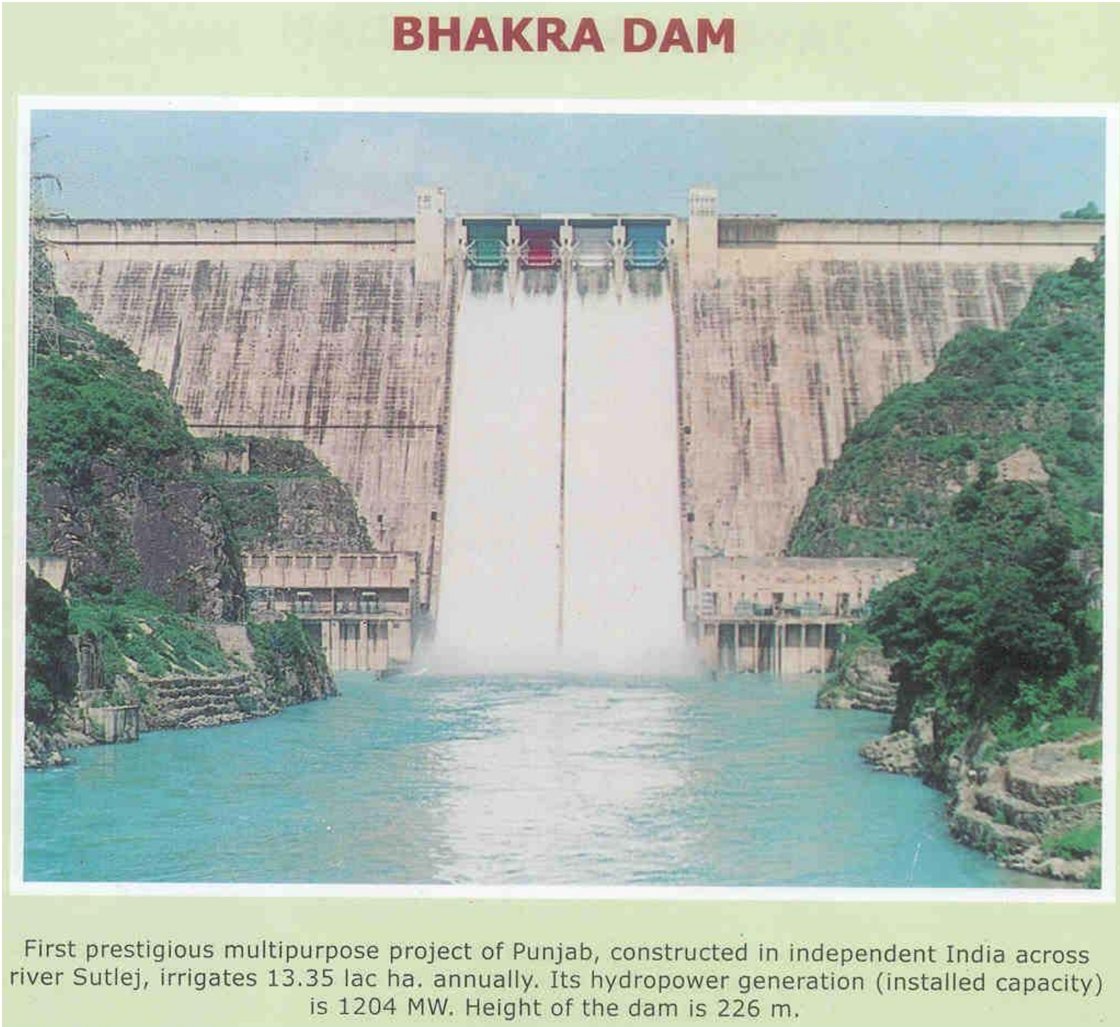
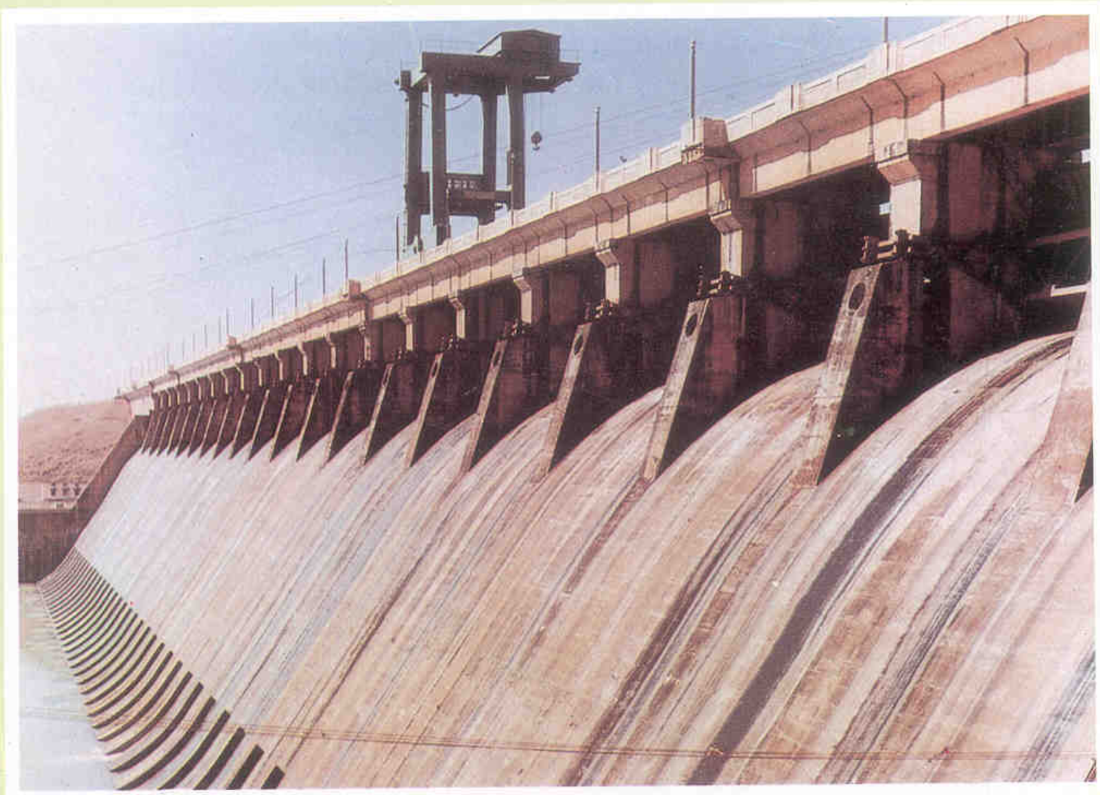


FIGURE 1. Upstream view of Bhakra dam showing spillway gates and chute

Image Cortesy: Ministry of Water Resources, Government of India, Web-site: http://www.wrmin.nic.in/

HIRAKUD DAM



The project across river Mahanadi in the state of Orissa, completed in 1957, provides annual irrigation to 2.51 lac ha. Length of the dam (including Dykes) is 25.5 Km.

CHAMERA PROJECT

(STAGE – I)

The project is located across river Ravi in the state of Himachal Pradesh. Its hydel power generation (installed capacity) is 540 MW.

Images of some other important water resources projects may be obtained from the web-site of the Ministry of Water Resources, Government of India Web-site: http://www.wrmin.nic.in/

2.4.1 Defining the design flood

The Design Flood for a hydraulic structure may also be defined in a number of ways, like:

- The maximum flood that any structure can safely pass.
- The flood considered for the design of a structure corresponding to a maximum tolerable risk.
- The flood which a project (involving a hydraulic structure) can sustain without any substantial damage, either to the objects which it protects or to its own structures.
- The largest flood that may be selected for design as safety evaluation of a structure.

Design Flood is also known as the Inflow Design Flood (IDF). It is the flood adopted for design purpose, and could be:

- The entire flood hydrograph, that is, the possible values of discharge as a function of time.
- The peak discharge of the flood hydrograph.

2.4.2 Choice of design flood

The Bureau of Indian standard guidelines IS: 5477 (Part IV) recommends that the Inflow Design Flood (IDF) of a structure, depending on its importance or risk involved, may be chosen from either one of the following:

- **Probable Maximum Flood (PMF):**
  This is the flood resulting from the most severe combination of critical meteorological and hydrological conditions that rare reasonably possible in the region. The PMF is computed by using the Probable Maximum Storm (PMS) which is an estimate of the physical upper limit to storm rainfall over the catchment. This is obtained from the studies of all the storms that have occurred over the region and maximizing them for the most critical atmospheric conditions.

- **Standard Project Flood (SPF):**
  This is the flood resulting from the most sever combination of meteorological and hydrological conditions considered reasonably characteristic of the region. The SPF is computed from the Standard Project Storm (SPS) over the watershed considered and may be taken as the largest storm observed in the region of the watershed. It is not maximized for the most critical atmospheric conditions but it may be transposed from an adjacent region to the watershed under consideration.

- **Flood of a specific return period:**
  This flood is estimated by frequency analysis of the annual flood values of adequate length. Sometimes when the flood data is inadequate, frequency analysis recorded storm data is made and the storm of a particular frequency applied to the unit hydrograph to derive the design flood. This flood usually has a return period greater than the storm.

The IDF's for different types of structures constructed across rivers are different. Some of the structures which are of importance to water resources engineering are:

- Storage Dams.
- Barrages and Weirs
- Diversion Works and Coffer dams
- Cross drainage works

A brief description of the structures and their corresponding IDF's are discussed subsequently.

2.4.3 Design flood for storage dams

Dams are important hydraulic structures which are constructed to serve a variety of purpose, more of which shall be discussed in detail in lesson 3.2. Most dams have a capacity to store substantial amount of water in the reservoir, and a portion of the inflow flood gets stored and the excess overflows through the spillways. According to Bureau of Indian Standard guidelines IS: 11223-1985, "Guidelines for fixing spillway capacity", the IDF to be considered for different requirements

2.4.3.1 IDF for the safety of the dam

It is the flood for which, when used with standard specifications, the performance of the dam should be safe against overtopping, structural failures, and the spillway and its energy dissipation arrangement, if provided for a lower flood, should function reasonable well.

- For large dams (defined as those with gross storage greater than 60 million m^3or hydraulic head greater than 30 m), IDF should be based on PMF.
- For intermediate dams (gross storage between 10 and 60 million m^3 or hydraulic head between 12 m and 30 m), IDF should be based on SPF.
- For small dams (gross storage between 0.5 to 10 million m^3 or hydraulic head between 7.5 m to 12 m), IDF may be taken as 100 years return period flood.

Floods of larger or smaller magnitude may be used if the hazard involved in the eventuality of a failure is particularly high or low. The relevant parameters to be considered in judging the hazard in addition to the size would be:

- Distance to and location of the human habitations on the downstream after considering the likely future developments; and
- Maximum hydraulic capacity of the downstream channel at a level which catastrophic damage is not expected.

2.4.3.2 IDF for efficient operation of energy dissipation system

It is a flood which may be lower than the IDF for the safety of the dam. When this flood is used with standard specifications or other factors affecting the performance, the energy dissipation arrangements are expected to work most efficiently.

2.4.3.3 IDF for checking extent of upstream submergence

This depends upon local conditions, type of property and effects of the submergence for very important structures upstream like power house, mines, etc. Levels corresponding to SPF or PMF may be used to determine submergence effects. For other structures consideration of smaller design floods and corresponding levels attained may suffice. In general, a 25 – year flood for land acquisition and 50 – year flood for built up property acquisition may be adopted.

2.4.3.4 IDF for checking extent of downstream damage in the valley

This depends on local conditions, the type of property and effects of its submergence. For very important facilities like powerhouse, outflows corresponding to the inflow design flood for safety of the dam, with all spillway gates operative or of that order may be relevant. Normally damage due to physical flooding may not be allowed under this condition, but disruption of operation may be allowed.

2.4.4 Design flood for barrages and weirs

Weirs and barrages, which are diversion structures, have usually small storage capacities, and the risk of loss of life and property would rarely be enhanced by failure of the structure. Apart from damage/loss of structure the failure would cause disruption of irrigation and communications that are dependent on the barrage. According to the bureau of Indian Standard guidelines IS: 6966(Part-I) - 1989, "Hydraulic design of barrages and weirs-guidelines for alluvial reaches", the following are recommended

- SPF or 500 year return period flood for designing ***free board***
- 50 year return period flood for designing of items other than free board.

2.4.5 Design flood for diversion works and coffer dam

Whenever a hydraulic structure like a dam or a barrage is constructed across a river, a temporary structure called a coffer dam is built first for obstructing the river flow and the water diverted though a diversion channel or tunnel. The Bureau of Indian Standards in its guideline IS: 10084 (Part I) -1982, "Criteria for design of diversion works - Part I: Coffer Dams" recommends the following:

> "The coffer dam being a temporary structure is normally designed for a flood with frequency less than that for the design of the main structure. The choice of a particular frequency shall be made on practical judgment keeping in view the construction period and the stage of construction of the main structure and its importance. Accordingly, the design flood is chosen.
>
> For seasonal cofferdams (those which are constructed every year and washed out during the flood season), and the initial construction stages of the main structure, a flood frequency of 20 years or more can be adopted. For coffer dams to be retained for more than one season and for the advanced construction stage of the main structure, a flood of 100 years frequency may be adopted".

2.4.6 Flood for cross drainage works

Cross drainage works are normally encountered in irrigation canal network system. Generally canals flow under gravity and often are required to cross local streams and rivers. This is done by either conveying the canal water over the stream by overhead aqueducts or by passing below the stream though siphon aqueducts. These structures are called cross drainage works and according to the Bureau of Indian Standard guidelines IS:7784 (Part I) – 1993, "Code of practice for design of cross – drainage works" the following is recommended.

"Design flood for drainage channel to be adopted for cross drainage works should depend upon the size of the canal, size of the drainage channel and location of the cross drainage. A very long canal, crossing drainage channels in the initial reach, damage to which is likely to affect the canal supplies over a large area and for a long period, should be given proper importance.

Cross drainage structures are divided into four categories depending upon the canal discharge and drainage discharge. Design flood to be adopted for these four categories of cross drainage structures is given as in the following table:

Category of structure	Canal discharge (m^3/s)	Estimated Drainage discharge Note* (m^3/s)	Frequency of design flood
A	0 – 0.5	All discharges	1 in 25 years
B	0.5 – 15	0 – 150 >150	1 in 50 years 1 in 100 years
C	15 – 30	0 – 100 >100	1 in 50 years 1 in 100 years
D	>30	0 – 150 >150	1 in 100 years Note **

Notes:

* This refers to the discharge estimated on the basis of river parameters corresponding to maximum observed flood level.

** in case of very large cross drainage structures where estimated drainage discharge is above 150 m^3/s and canal discharge greater than 30 m^3/s, the hydrology should be examined in detail and appropriate design flood adopted, which is no case shall be less than 1 in 100 years flood

2.4.7 Methods for design flood computations

The criteria for choosing the design flood for various types of hydraulic structures were discussed. For each one of these, any of the following three methods are suggested:

- ***Probable Maximum Flood (PMF)***
- ***Standard Project flood (SPF)***
- ***Flood of a specific return period***

The methods for evaluating PMF and SPF fall under the hydrometeorological approach, using the unit hydrograph theory. Flood of a given frequency (or return period) is obtained using the statistical approach, commonly known as flood frequency analysis. In every method, adequate data for carrying out the calculations are required. The data which are required include long term and short term rainfall and runoff values, annual flood peaks series, catchment physiographic characteristics, etc.

Within the vast areal extent of our country, it is not always possible to have observations measured on every stream. There are a large number of such ungauged catchments in India which has to rely on synthetically generated flood formulae. The Central Water Commission in association with the India Meteorological Department and Research Design and Standard Organization unit of the Indian Railways have classified the country into 7 zones and 26 ***hydro-meteorologically homogeneous sub-zones***, for each one of which flood estimation guidelines have been published. These reports contain ready to use chart and formulae for computing floods of 25, 50 and 100 year return period of ungauged basins in the respective regions.

In the subsequent section, we look into some detail about the calculations followed for the computation of

- PMF and SPF by the hydrometeorological approach.
- Evaluation of a flood of a given frequency by statistical approach.

2.4.8 The hydro-meteorological approach

The probable maximum flood (PMF) or the standard project flood (SPF) is estimated using the hydro-meteorological approach. For the PMF calculations the worst possible maximum storm (PMS) pattern is estimated. This is then applied to the unit hydrograph of the catchment to obtain the PMF. For the calculation so the SPF, the worst observed rainfall pattern (called the Standard Project Storm or SPF) is applied to the unit hydrograph derived for the catchment.

For the estimation of the PMS or the SPS, which falls under the hydro-meteorological approach, an attempt is made to analyze the causative factors responsible for the production of severe floods. The computations mainly involve estimation of a design storm hyetograph (from past long-term rainfall data within the catchment) and derivation of the catchment response function used which can either be a lumped model or a distributed-lumped model. In the former, a unit hydrograph is assumed to represent the entire catchment area. In the distributed-lumped model, the catchment is divided into smaller sub-regions or sub-catchment and the unit hydrographs of each sub-region are applied together with ***channel routing*** and sometimes ***reservoir routing*** to produce the catchment response.

PMF/SPF calculation method by the hydro-meteorological approach involves the following steps:

- ***Data requirement for PMF/SPF studies***

- ***Steps for evaluating PMF/SPF***

- ***Limitation of PMF/SPF calculations***

These are explained in detail in the additional section 2.4.12 at the end of this lesson.

2.4.9 The statistical approach

The statistical approach for design flood estimation, otherwise also called flood frequency analysis, may be performed on the past recorded data of annual flood peak discharges either directly observed at the site or estimated by a suitable method. Alternatively, frequency analysis may be carried out on the available record of annual rainfall events of the region.

The probability of occurrence of event (say, the maximum flood discharge observed or likely to occur in a year at a location on a river), whose magnitude is equal to or in excess of a specified magnitude X is denoted by P. A related term, the reccurrence interval (also known as the return period) is defined as T = 1/P. This represents the average interval between the occurrence of a flood peak of magnitude equal to or greater than X.

Flood frequency analysis studies interpret past record of events to predict the future probabilities of occurrence and estimate the magnitude of an event corresponding to a specific return period. For the estimation of flood flows of large return periods, it is often necessary to extrapolate the magnitude outside the observed range of data. Though a limited extrapolation to about twice the length of the record (that is, the number of years of data that is available) expected to yield reasonable accuracy, often water resources engineers are required to project much more than that.

2.4.10 Calculations for flood frequency

Basic to all frequency analyses, is the concept that there is a collection of data, called the ‘population’. For flood frequency studies, this population are taken as the annual maximum flood occurring at a location on a river (called the site). Since the river has flooded during the past years and is likely to go on flooding over the coming years (unless something exceptional like drying up of the river happens!), the recorded flood peak values which have been observed for a finite number of years are only a sample of the total population. Here, ‘flood peak’ means the highest recorded discharge value for the river at any year. The following assumptions are generally made for the data:

- The sample is representative of the population. Thus, it is assumed that though only a finite years' data of peak flow has been recorded, the same type of trend was always there and would continue to be so in future.

- The data are independent. That is, the peak flow data which has been collected are independent of each other. Thus, the data set is assumed to be random. In a random process, the value of the variant does not depend on previous or next values.

Flood frequency analysis starts by checking the consistency of the data and finding the presence of features such as trend, jump, etc. Trend is the gradual shift in the sample data, either in the increasing or decreasing directions. This may occurs due to human interference, like afforestation or deforestation of the watershed. Jump means that one or a few of the data have exceptional values – high or low, due to certain factors, like forest fire, earthquake, landslide, etc which may change the river's flow characteristics temporarily.

The next step is to apply a convenient probability distribution curve to fit the data set. Here, it is assumed that yearly observed peak flow values are random numbers and which are also representative of the population, which includes all flood peak values, even these which have not been recorded or such floods which are likely to happen in future. Each data of the set is termed as a variate, usually represented by '*x*' and is a particular value of the entire data range '*X*'.

The probability of a variable is defined as the number of occurrences of a variate divided by the total number of occurrences, and is usually designated by '*P*'. The total probability for all variates should be equal to unity, that is, $\sum P=1$. Distribution of probabilities of all variates is called Probability Distribution, and is usually denoted a *f(x)* as shown in Figure 4.

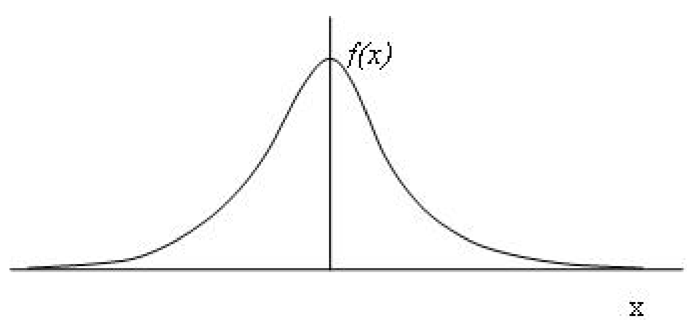


FIGURE 4. A typical probability distribution

The cumulative probability curve, F(*x*) is of the type as shown in Figure 5.

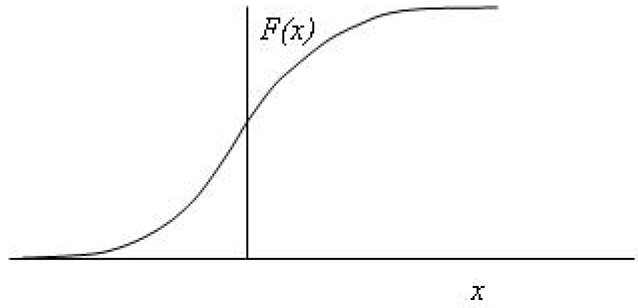


FIGURE 5. Cumulative probability curve

The cumulative probability, designated as P(x ≤ *x*), represents the probability that the random variable has a value equal to or less than certain assigned value *x* is equal to 1 – P(x ≤ *x*), or P(x ≥ *x*).

In the context of flood frequency analysis, we may use the above concepts by assuming the recorded yearly flood peaks as the variate '*x*'. Then, if the functions f(*x*) or F(*x*) becomes known, then it is possible to find out the probability with which certain high flood peak is likely to occur. This idea may be used to recalculate the high flood peak that is likely to be equalled or exceeded corresponding to a given frequency (say, 1 in 100 years).

There are a number of probability distributions f (x), which has been suggested by many statisticians. Of these, the more common are

- ***Normal***
- ***Log – normal***
- ***Pearson Type III***
- ***Gumbel***

Which one of these fits a given data set has to be checked using certain standard statistical tests. Once a particular distribution is found best, it is adopted for calculation of floods likely to occur corresponding to specific return periods.

Details of the above methods may be found in the additional section 2.4.12 at the end of this lesson.

2.4.11 Plotting positions

So far we talked about extrapolation of the sample data. However, if probability is to be assigned to a data point itself, then the 'plotting position' method is used. Here, the sample data (consisting of, say, N values) is arranged in a decreasing order. Each data (say the event X) of the ordered list is then given a rank 'm' starting with 1 for the highest up to N for the lowest of the order. The probability of exceedence of X over a certain value x, that is P(X ≥ x) is given differently by different researchers, the most common of which are as given in the table below:

Sl. No	Name of formula	P(X ≥ x)
1	California	m/N
2	Hazen	(m-0.5)/N
3	Weibull	m/(N+1)

Of these, the Weibull formula is most commonly used to determine the probability that is to be assigned to data sheet.

Example showing application of plotting positions:

The application of the method of plotting may be explained better with an example. Assume that the yearly peak flood flows of a hypothetical river measured at a particular location over the years 1981 to 2000 is given as in the following table. The data is to used to calculate the flood peak flow that is likely to occur once every 10 years, and once every 50 years.

Year	Peak flood (m^3/s)
1981	700
1982	810
1983	470
1984	300
1985	440
1986	600
1987	350
1988	290
1989	330
1990	670
1991	540
1992	430
1993	320
1994	420
1995	690
1996	400
1997	360
1998	510
1999	910
2000	100

Rearranging table according to decreasing magnitude, designate a plotting position and calculate the probability of exceedence by, say, the Weibull formula shown in the following table which also gives the Return Period T (1/P).

m	Peak flood (m^3/s)	Probability $P = \frac{m}{N+1}$	Return period T = 1/P years
1	910	0.048	21.000
2	810	0.095	10.500
3	700	0.143	7.000
4	690	0.190	5.250
5	670	0.238	4.200
6	600	0.286	3.500
7	540	0.333	3.000
8	510	0.381	2.625
9	470	0.429	2.333
10	440	0.476	2.100
11	430	0.524	1.909
12	420	0.571	1.750
13	400	0.619	1.615
14	360	0.667	1.500
15	350	0.714	1.400
16	330	0.762	1.313
17	320	0.810	1.235
18	300	0.857	1.167
19	290	0.905	1.105
20	100	0.952	1.050

2.4.12 Additional information and definition of important terms

Free board

The marginal distance that is providing above the maximum reservoir level to avoid the possibility of water spilling over the dam is known as the free board.

Hydro- meteorologically homogeneous sub-zones

This indicates a partition of the country in terms of similar hydrological and meteorological areas. There are, in all, 26 sub-zones in the country. This has been done together by the Central Water Commission (CWC), Research Designs and Standards Organization (RDSO), and India Meteorological Department (IMD).

Channel routing

The outlet of each sub-catchment is located many km upstream of the outlet of the main catchment. The outflow of a sub-catchment will pass through the channels before finally reaching the catchment outlet. The inflow hydrograph to a channel will get modified by the temporary storage of channel; hence it is necessary to estimate the outflow hydrograph of the channel to in order to find the flow at the outlet of the catchment outlet by a process is known as channel routing.

Reservoir routing

The hydrograph of a flood entering a reservoir will change in shape as it emerging out from the reservoir. This is due to volume of water stored in reservoir temporarily. The peak of the hydrograph will be reduced, time to peak will be delayed and base of the hydrograph will be increased. The extent up to which an inflow hydrograph will be modified in the reservoir will be computed by the process is known as reservoir routing.

Data requirement for PMF/SPF studies

1. Watershed data
    - Total watershed area, snowbound area, minimum and maximum elevations above the mean sea level and length of river up to the project site;
    - ***Lag time***, ***travel times*** of reaches, and time of concentration;
    - Contributing areas, mean overland flow distances and slopes;
    - Design storm water losses, ***evaporation***, ***infiltration***, ***depression*** and ***interception*** losses, infiltration capacities.
    - Land use practices, soil types, surface and subsurface divides

2. Channel data
    - Channel and valley cross sections at different places under consideration to fix the gauge discharge rating curves.
    - Manning's n or the data required to estimate channel roughness coefficient

3. Runoff data
    - ***Base flow*** estimates during design floods.
    - Available historical data on floods along with the precipitation data including that of self-recording rain gauges, if available.

4. Storm data
    - Daily rainfall records of all rain gauge stations in and around the region under study
    - Rainfall data of self-recording rain gauges
    - Data of the storm dew point and maximum dew point temperatures

Steps for evaluating PMF/SPF

1. Estimate duration of design storm

   Duration of design storm equivalent to ***base period*** of ***unit hydrograph*** rounded to the next nearest value which is in multiplier of 24 hours and less than and equal to 72 hours is considered to be adequate. For large catchments, the storm duration for causing the PMF is to be equivalent to 2.5 times the travel time from the farthest point ***(time of concentration)*** to the site of the structure.

2. Selection of design storm
   A design storm is an estimate of the rainfall amount and distribution over a particular drainage areas accepted for use in determining the design flood. This could either be the ***Probable Maximum Storm*** (PMF) or the ***Standard Project Strom*** (SPS)**.**

3. Time adjustment of design storm and its critical sequencing
   The design hyetograph should be arranged in two bells (peak) per day. The combination of the bell arrangement and the arrangement of the rainfall increments within each of the bell shaped spells will be representing the maximum flood producing characteristics.

   The critical arrangement of increment in each bell should minimize the sudden hill or sluggishness and maximizing the flood peak. Hence, the arrangement is to be such that the time lay between peak intensities of two spells may be minimum. The cumulative pattern of all the increments in the order of their positioning should resemble the natural mass curve pattern as observed by a ***Self Recording Rain gauge*** (SRRG) of the project region.

4. Estimate the design ***Unit Hydrograph***
   Depending upon the data availability and characteristics of flood hydrograph etc, the unit hydrograph may be derived using any of the following techniques.
   - Simple method of unit hydrograph derivation from a flood event with isolated peak
   - Collin's method
   - Nash method
   - Clarke model.

   In case of insufficient data, synthetic unit hydrograph may be derived

5. Calculating the probable maximum flood hydrograph
   The critical time sequence of the design storm rainfall is superimposed on the derived design unit hydrograph to give the direct hydrograph, when added to the base flow, gives the probable maximum flood hydrograph. Details of these calculations are given in Lecture 2.4

Limitation of PMF/SPF calculations

- Requirement of long-term hydrometeorological data for estimation of design storm parameters.

- The knowledge of rainfall process as available today has severe limitations and therefore, physical modeling of rainfall to compute PMP is still not attempted.

- Maximization of historical storms for possible maximum favorable conditions is presently done on the basis of surface dew point data. Surface dew point data may not strictly represent moisture availability in the upper atmosphere.

- Availability of self-recording rain gauge (SRRG) data for historical storms (Remember that SRRG data gives the distribution of rain fall with time).

- Many of the assumptions of the unit hydrograph theory are not satisfied in practice.

- Many a times, data of good quality and adequate quantity is not available for the derivation of unit hydrograph.

Normal Distribution

The Normal distribution is one of the most important distributions in statistical hydrology. This is used to fit empirical distributions with skewness coefficient close to zero. The probability density function (PDF) of the distribution is given by

$$f(x) = \frac{1}{\sigma\sqrt{2\pi}} \exp\left[-\frac{1}{2}\left(\frac{x-\mu}{\sigma}\right)^2\right]; \qquad -\infty < x < \infty$$

Where, μ is the location parameter and σ is the scale parameter. The cumulative distribution function (CDF) of the normal distribution is given by:

$$F(x) = \frac{1}{\sigma\sqrt{2\pi}} \int_{-\infty}^{x} \exp\left[-\frac{1}{2}\left(\frac{x-\mu}{\sigma}\right)^2\right]$$

Log – Normal Distribution

If the logarithms, $\ln x$, of a variable *x* are normally distributed, then the variable *x* is said to be log normally distributed so that

$$f(x) = \frac{1}{x\sigma_y\sqrt{2\pi}} \exp\left[-\frac{1}{2}\left(\frac{x-\mu_y}{\sigma_y}\right)^2\right]$$

Where, μ_y and σ_y are the mean and standard deviation of the natural logarithm of x. Log normal distributions can be applied to a wide variety of hydrologic events especially in the cases in which the corresponding variable has a lower bound, the frequency distribution is not symmetrical and the factors causing those are independent and multiplicative.

If the variable x has a lower boundary x_0, different from zero, and the variable $z = x - x_0$ follows a lognormal distribution, then x is lognormally distributed with three parameters. The probability distribution function of the lognormal distribution with parameters is

$$f(x) = \frac{1}{(x - x_0)\sigma_y \sqrt{2\pi}} \exp\left[-\frac{1}{2} \left(\frac{\ln(x - x_0) - \mu_y}{\sigma_y} \right)^2 \right]$$

Where, μ_y, σ_y and x_0 are called the scale, the shape and the location parameters respectively. Parameter x_0 is generally estimated by trial and error.

Pearson Type III Distribution

Pearson type III is a three parameter distribution, also known as Gamma distribution with three parameters. The PDF of the distribution is given as

$$f(x) = \frac{(x - x_0)^{\gamma - 1} \exp\left[-\frac{(x - x_0)}{\beta} \right]}{\beta^{\gamma} \Gamma(\gamma)}$$

The CDF of the Pearson Type III distribution is given by

$$F(x) = \int_{x_0}^{x} \frac{(x - x_0)^{\gamma - 1} \exp\left[-\frac{(x - x_0)}{\beta} \right]}{\beta^{\gamma} \Gamma(\gamma)} dx$$

Where x_0, β, and γ are location, scale and shape parameters respectively.

Gumbel Distribution

Gumbel distribution is a member of family of Extreme Value distributions with the value of parameter k = 0. It is a two parameter distribution and is widely used in hydrology.

The PDF is given as

$$f(x) = \frac{1}{\alpha} \exp\left[-\frac{x - \mu}{\alpha} - \exp\left\{ -\frac{x - \mu}{\alpha} \right\} \right]$$

And CDF is given as

$$F(x) = \exp\left[-\exp\left\{ -\frac{x - \mu}{\alpha} \right\} \right]$$

$$E(X) = \mu + 0.5772\alpha$$

$$Var(X) = \frac{\pi^2\alpha^2}{6}$$

Where, u and α are location and shape parameters respectively.

Lag Time

Lag is the time between the peak flow and the centroid of rainfall.

Travel time

The time taken by the water to reach the basin outlet, from the different points in the basin, is called the travel time.

Evaporation

The process of extracting moisture is known evaporation.

Infiltration

Infiltration is defined as the slow passage of a liquid through a filtering medium.

Interception

Interception is the act of catching the precipitation by the trees or buildings without reaching to the ground surface.

Base flow

Base flow is the portion of the stream discharge that is derived from natural storage (e.g., groundwater outflow and the draining of large lakes and swamps or other source outside the net rainfall that creates surface runoff).

Base period

The time between the first watering of a crop at the time of its sowing to its last watering before harvesting is called base period. Base period is always less than crop period.

Unit hydrograph

A unit hydrograph is defined as the hydrograph of runoff produced by excess rainfall of 1cm occurring uniformly over the entire drainage basin at a uniform rate over the entire specified duration.

Time of concentration

The time of concentration of a drainage basin is the time required by the water to reach the outlet from the most remote point of the drainage area.

Module 2

The Science of Surface and Ground Water

Lesson
5
Subsurface Movement of Water

Instructional Objectives

At the end of this lesson, the student shall be able to learn

1. The ways by which water moves below ground.
2. How water is stored underground in soil and fractured bedrock.
3. What is water table and its relation to saturated and unsaturated ground water.
4. Pressure measurement in unsaturated and saturated soil water.
5. The theory behind movement of water in the unsaturated zone.
6. The theory behind movement of water in the saturated zone, also called as ground water.
7. What are equipotential and flow lines.
8. What are aquifers and confining layers.
9. Qualitatively, the ground water flow movement in confined and unconfined layers.
10. The definitions of water storage, and the portion of that which can be withdra3wn from an aquifer.

2.5.0 Introduction

In the previous lectures of this module, we talked about the hydrologic cycle, which is a continuous process of transformation of water in the form of water vapour as it evaporates from land and ocean, drifts away to clouds and condenses to fall as rain. Of the rain falling over the land surface, a part of it infiltrates into the soil and the balance flows down as surface runoff. From the point of view of water resources engineering, the surface water forms a direct source which is utilized for a variety of purposes. However, most of the water that infiltrates into the soil travels down to recharge the vast ground water stored at a depth within the earth. In fact, the ground water reserve is actually a huge source of fresh water and is many times that of surface water. Such large water reserves remain mostly untapped though locally or regionally, the withdrawal may be high. Actually, as a result of excess withdrawal of ground water in many places of India (and also of the world), a number of problems have arisen. Unless the water resources engineer is aware of the consequent damages, this type of situation would lead to irreversible change in the quality and quantity of subsurface water which likely to affect our future generations.

In this lecture it is proposed to study how the water that infiltrates into the soil and the physics behind the phenomena. We have deliberately separated the study of subsurface movement of water from that of surface flow, as discussed in the earlier lectures, because of the fact that the scale of movement of these two types of flows can vary by an order of magnitude 10 to more than 1000! This would be clear from Figure 1.

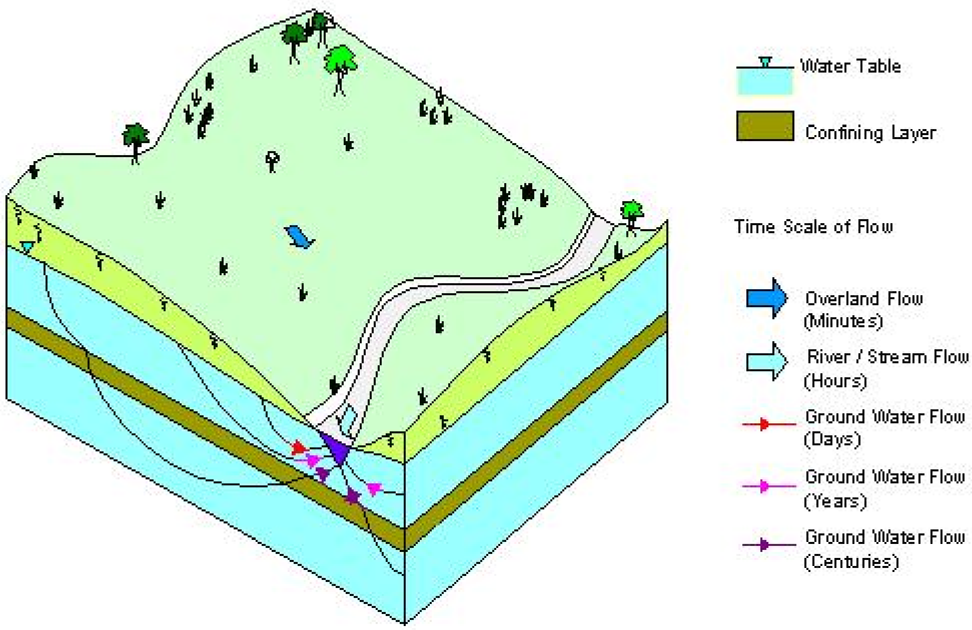


FIGURE 1. Surafce and sub-surface movement of water

2.5.1 Subsurface water and the soil – rock profile

Figures 2 and 3 show two examples of underground soil–rock profiles and their relations with subsurface water that may exist both as confined and unconfined ground water reserves.

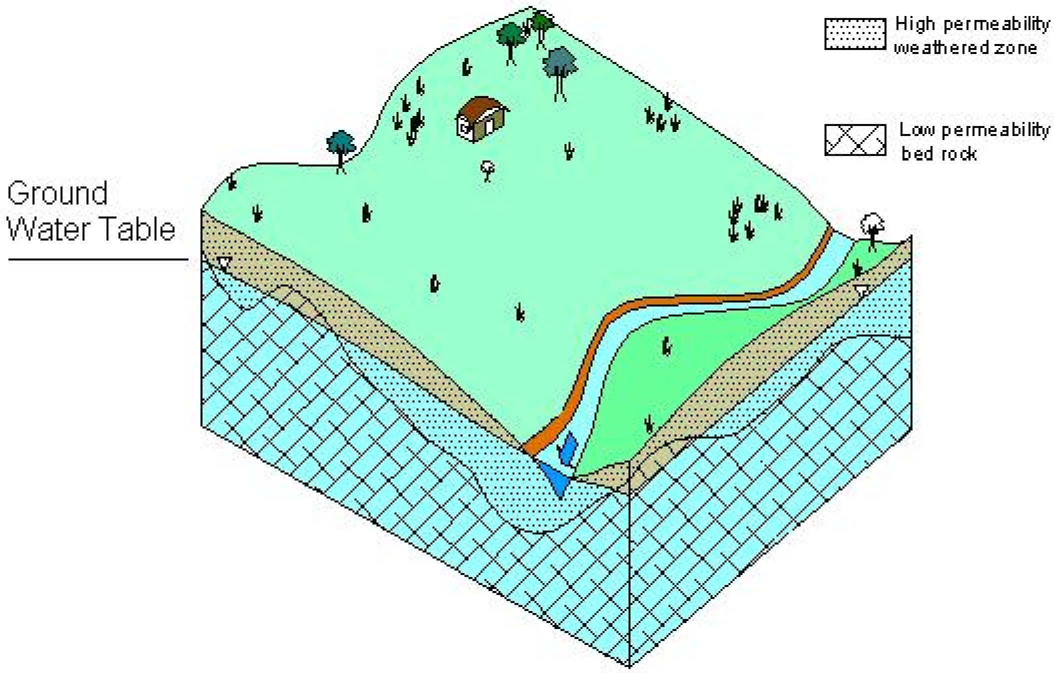


FIGURE 2. Sub-soil water reserve without any confining layer of soil

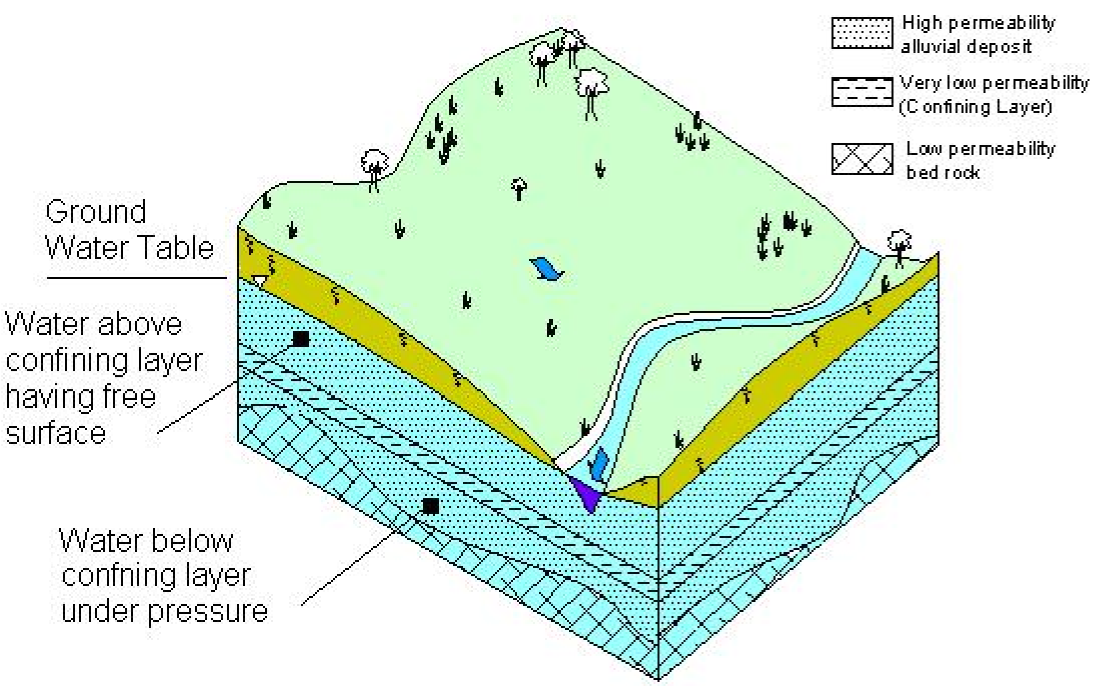


FIGURE 3. Sub-soil water reserve as unconfined as well as confined aquifers

In fact, water is present in the pores of soils and fissures of rock up to a depth beyond which there is solid rock with no gaps which can store water. Although water is present in the pores of the soil and permeable rocks, there is difference between that stored above the ***water table*** and below it. The soil above the water table has only part of the voids filled up with water molecules whereas the soil below is completely saturated.

If we look more closely at the upper layers of the soil rock system, we find that it is only the change in moisture content that separates the unsaturated portion and the saturated portions of the soil. Figure 4 shows a section through a soil – rock profile and corresponding graph showing the degree of saturation. Except the portion of the soil storing groundwater the remaining is unsaturated.

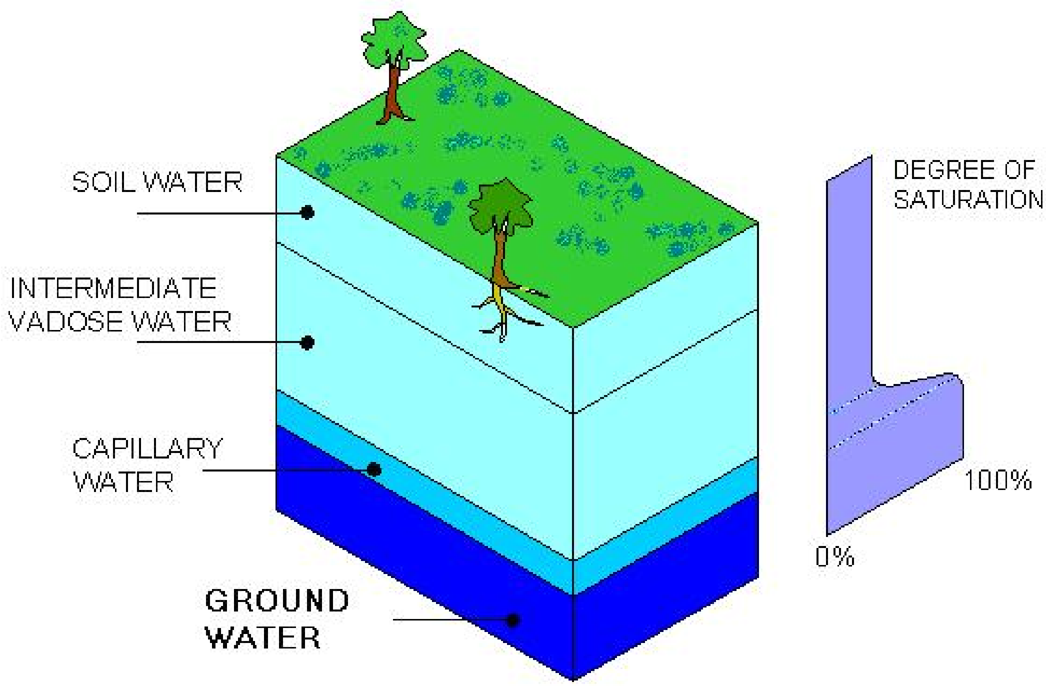


FIGURE 4. Changes in the degree of saturation for different zones of soil

It may be noted that even in the driest climate, the degree of saturation in the unsaturated zone would not be zero as water clings to the soil particles by surface tension.

Some of the definitions related to subsurface water are as follows:

- **Soil water**: The water stored in the upper layers of the soil from the ground surface up to the extent of roots of plants

- **Vadose water**: That stored below in the region between soil water zone and the capillary fringe. It is a link between water infiltrating from the ground surface and moving down to the saturated layer of ground water

- **Capillary water:** That which has risen from the saturated ground water region due to capillary action. Naturally, the pressure here would be less than atmospheric.

- **Ground water:** This is the water in the fully saturated zone. Pressure of water here would be more than atmospheric.

- **Water table:** An imaginary surface within ground below which all the voids of the soil or permeable rock are completely filled with water. Below this imaginary surface, the pore water pressure is atmospheric. As one moves downwards from the water table, the pressure increases according to the hydrostatic law. Above the water table, the voids of soil/porous rock are only partially saturated with water clinging to the surface of the solids by surface tension. Hence, the pressure here is sub-atmosphere.

2.5.2 Water pressure in unsaturated zone

In literature, the term 'ground water flow' is used generally to describe the flow of water in the saturated portion of soil or fractured bedrock. No doubt it is important from the point of extraction of water from the zone using wells, etc. But the unsaturated zone, too, is important because of the following reasons:

- The water in the unsaturated zone is the source of moisture for vegetation (the soil water)
- This zone is the link between the surface and subsurface hydrologic processes as rain water infiltrates through this zone to recharge the ground water.
- Water evaporated or lost by transpiration from the unsaturated zone (mainly from the soil water zone) recharges the atmospheric moisture.

Further, the process of infiltration, quite important in hydrologic modeling catchment, is actually a phenomenon occurring in the unsaturated zone. Hence, knowledge about unsaturated zone water movement helps to understand infiltration better.

At the water table, the pressure head (conventionally denoted by Ψ) is zero (that is atmospheric), that in the unsaturated zone is (here Ψ is also called the moisture potential) and in the saturated zone, it is positive. The hydraulic head at a point would, therefore, be defined as

$$h = z + \psi \qquad (1)$$

Where, z is the elevation head, or the potential head due to gravity. According to the mechanics of flow, water moves from higher hydraulic head towards lower hydraulic head

It may be noted that we may measure the negative pressure head within the unsaturated zone using a tensiometer. It consists of a porous ceramic cup connected by a water column to a manometer. The positive pressure head below water table can be determined using the hydrostatic pressure head formula γD, where γ is the unit weight of water and D is the depth of water below water table.

2.5.3 Movement of water in unsaturated zone

The negative pressure head in the unsaturated zone of the soil can be metaphorically expressed as the soil being "thirsty". All the pores of the soil here are not filled up. Hence, as soon as water is applied to the soil surface, it is "lapped up" by the soil matrix. Only if the water is applied in excess of the amount that it can "drink", would water flow over the land surface as surface runoff. This capacity of the soil in the unsaturated portion to absorb water actually depends on the volumetric water content, θ expressed as:

$$\theta = \frac{V_w}{V}$$

Where V_w is the volume of water and V is the unit volume of soil or rock.

What happens to the water that got absorbed (that is infiltrated) at the surface of the unsaturated soil during application of water from above? It moves downward due to gravity through inter connected pores that are filled with water. With increasing water content, more pores fill, and the rate of downward movement of water increases.

A measure of the average rate of movement of water within soil (or permeable bed rock) is the hydraulic conductivity, indicated as 'K', and has the unit of velocity. Though it is more or less constant for a particular type of soil in the saturated zone, it is actually a function of the moisture content in the unsaturated portion of the soil. As θ increases, so does K, and to be precise, it should correctly be written as K(θ), indicating K to be a function of θ. Figure 5 shows such a typical relation for an unsaturated soil.

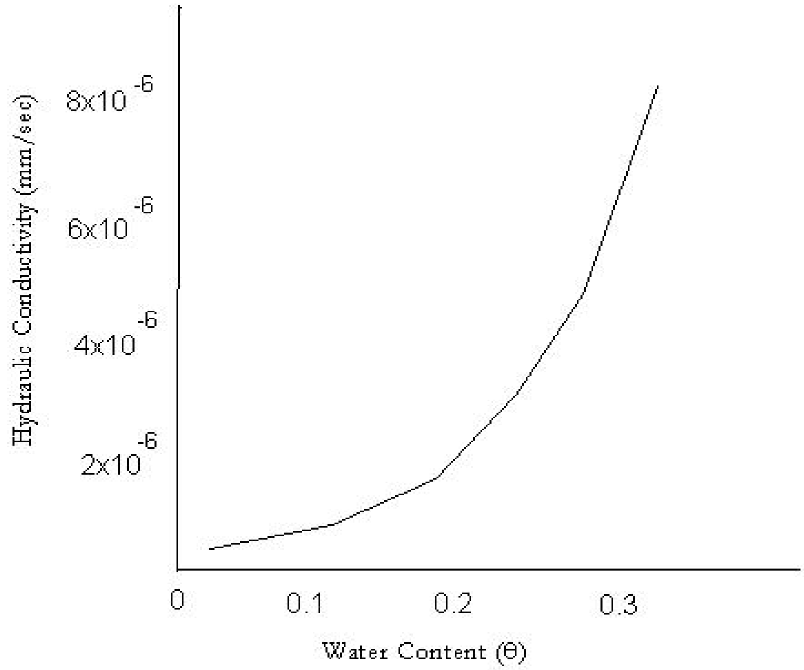


FIGURE 5. Variation of Hydraulic Cionductivity (K) with Water Content (θ)

Actually, the moisture potential (Ψ) is also a function of θ, as shown in Figure 6.

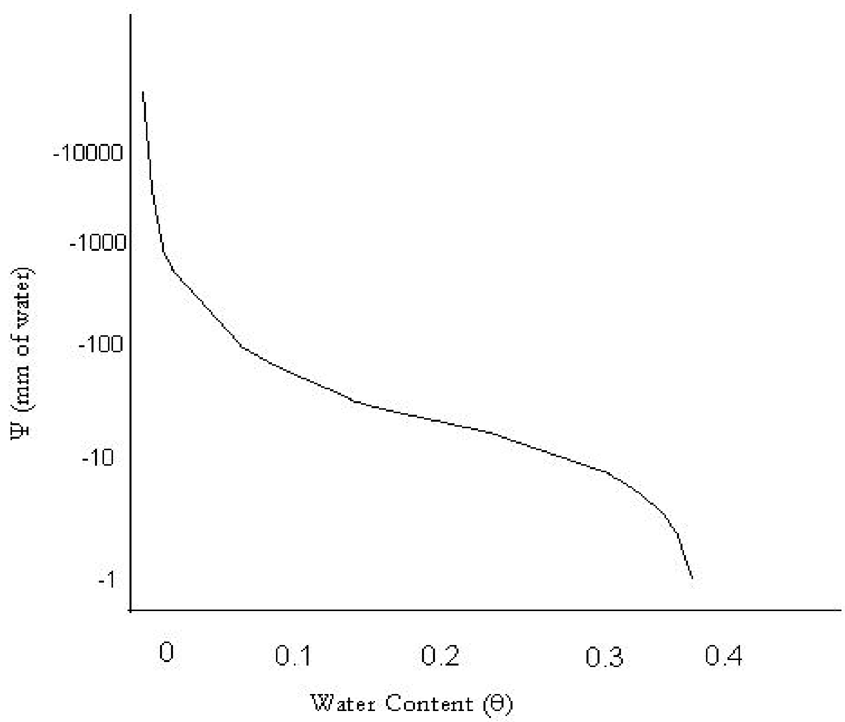


FIGURE 6. Variation of Moisture Potential and Water Content

The relationship of unsaturated hydraulic conductivity and volumetric water content is determined experimentally. A sample of soil placed in a container. The water content is kept constant and the rate at which water moves through the soil is measured. This is repeated for different values of θ (that is different saturation levels). It must be recommended that both K and Ψ very with θ and by its very nature, unsaturated flow involves many changes in volumetric moisture content as waves of infiltration pass.

The movement of a continuous stream of water infiltrating from the ground into the unsaturated soil may be typically seem to be as shown in Figure 7.

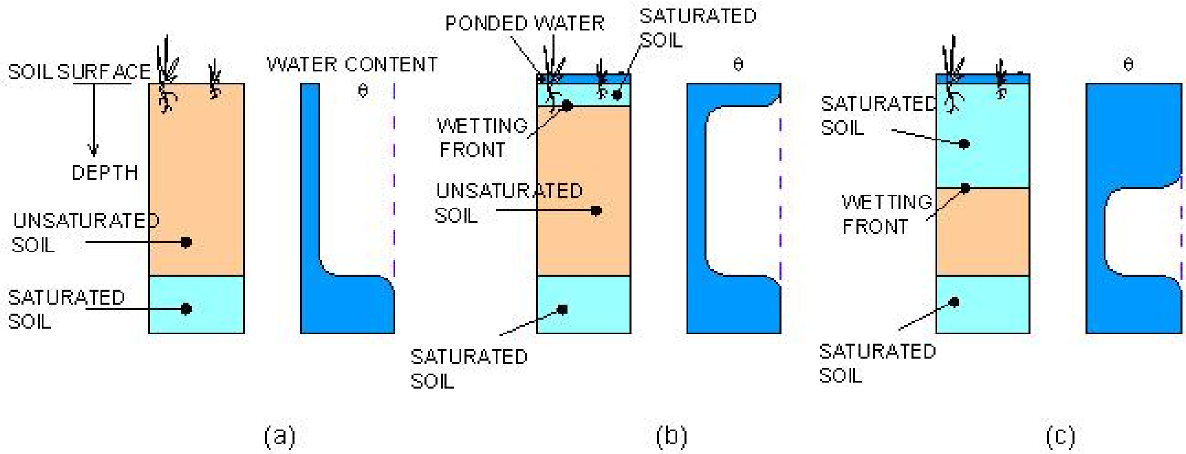


FIGURE 7. Variation of soil moisture for water infiltrating from surface
(a) Intial condition, water is yet to be applied
(b) Soon after water has been applied by ponding
(c) Some time after ponding

If the source of the water is now cut off, then the distribution of water content with depth may look like as shown in Figure 8.

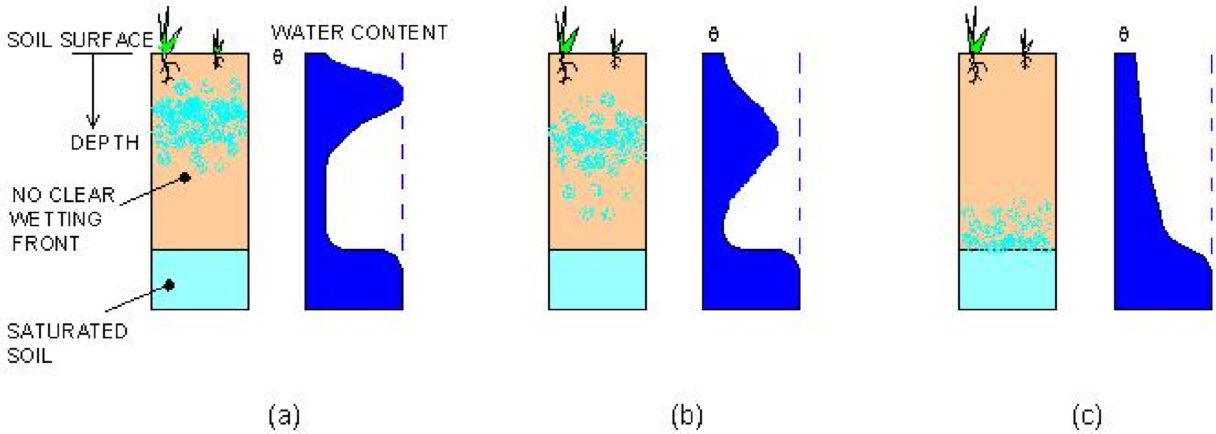


FIGURE 8. Variation in soil water content after the ponded water is exhausted
(a) Soon after
(b) Some time after
(c) A long time after

2.5.4 Movement of water in saturated zone

The water that infiltrates through the unsaturated soil layers and move vertically ultimately reaches the saturated zone and raises the water table. Since it increases the quantity of in the saturated zone, it is also termed as 'recharge' of the ground water.

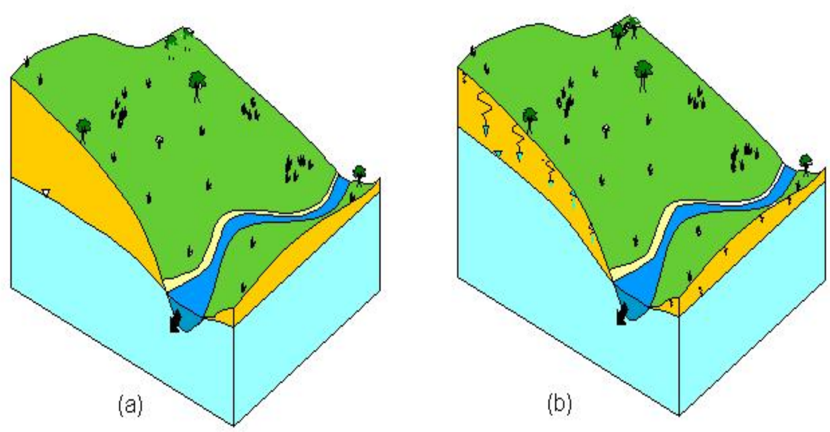


FIGURE 9. Variation of ground water table: (a) Before infiltration;
(b) During / soon after infiltration

It may be observed from Figure 9 that both before any infiltration took place, there existed a gradient of the water table which showed a small gradient towards the river. However, the rise of the water table after the recharge due to infiltrating water is not uniform and thus the gradient of the water table after recharge is more than that before recharge. This has a direct bearing on the amount of ground water flow, which is proportional to the gradient. Based on actual observation or on mathematical analyses, we may draw lines of equal hydraulic head (the equipotentials) within the saturated zone, as shown in Figure 10. We may also draw the flow lines, which would be perpendicular to the equipotential lines. The flow lines, indicating the general direction of flow within the saturated soil zone is also drawn in the figure.

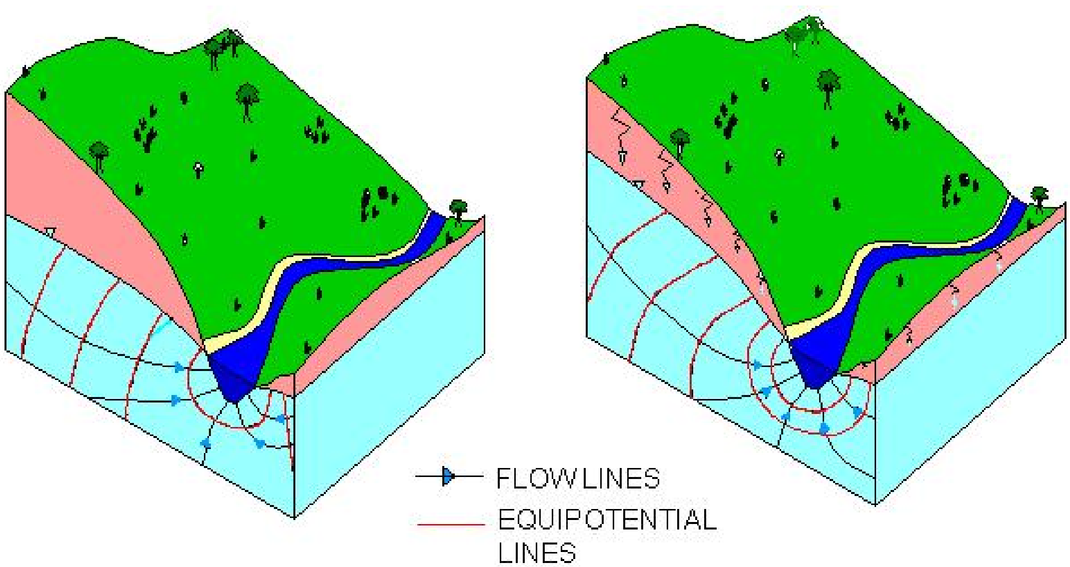


FIGURE 10. Equipotential lines and Flow lines of ground water movement:
(a) Before infiltration, with a low water table
(b) During / soon after infiltration, with a raised water table

The rate of movement of the ground water, of course, varies with the material through which it is flowing since actually the flow is taking place through the voids which is different for different materials. The term ***hydraulic conductivity*** of a porous medium is used to indicate the ease with which water can flow through it. It is defined as the discharge taking place through a flow tube (which may be thought of as a short pipe along a flow line) per unit area of the tube under the influence of unit hydraulic gradient (which is the difference of potential heads in unit distance along the flow line). Hydraulic conductivity is generally denoted by 'K' and if the porous material is homogeneous, then K is also likely to be the same in any direction. However, in nature, the soil layers are often formed in layers resulting in the hydraulic conductivity varying between different directions. Even porous bed rock, which is usually fractured rock, may not be fractured to the same extent in all directions. As a result, in many natural flows the flow is more in some preferential direction. This type of conducting media is referred to as being heterogeneous and the corresponding hydraulic conductivity is said to be anisotropic.

2.5.5 Examples of ground water flow

Although ground water flow is three – dimensional phenomenon, it is easier to analyse flows in two – dimension. Also, as far as interaction between surface water body and ground water is concerned, it is similar for lakes, river and any such body. Here we qualitatively discuss the flow of ground water through a few examples which show the relative interaction between the flow and the geological properties of the porous medium. Here, the two – dimensional plane is assumed to be vertical.

1. Example of a gaining lake and river.

Figure 11 shows an example of a lake perched on a hill that is receiving water from the adjacent hill masses. It also shows a river down in a valley, which is also receiving water.

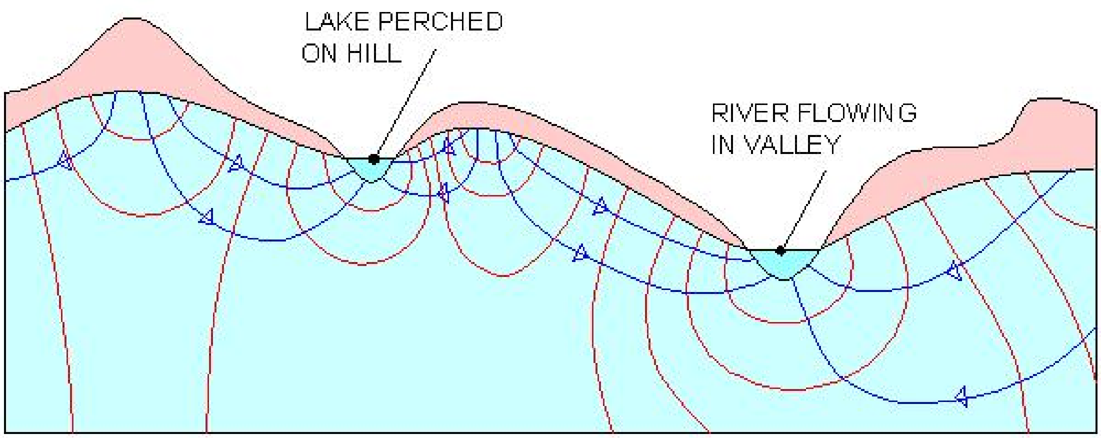


FIGURE 11. Example of a lake and a river, both of which are receiving warer from the adjoining soils.

2. Example of a partially losing lake, a disconnected losing lake, and a gaining river.

Figure 12 illustrates this example modifies the situation of example 1 slightly.

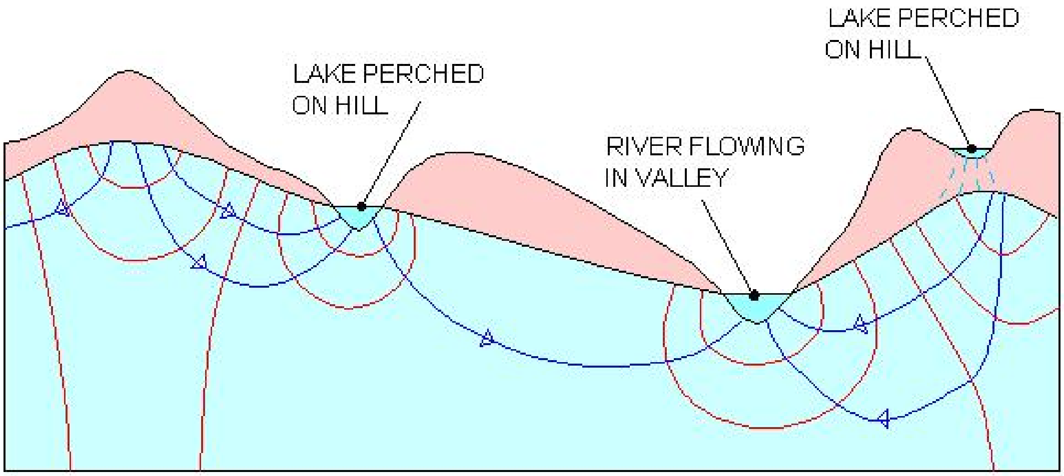


FIGURE 12. An example of two lakes, one of which is gaining water, as well as loosing; one river that is continuously gaining; and another lake perched on a hill, disconnected from the water table, and thus loosing water by infiltration

3. Example of flow through a heterogeneous media, case I.

This case (Figure 13) illustrates the possible flow through a sub-soil material of low hydraulic conductivity sandwiched between materials of relatively higher hydraulic conductivities.

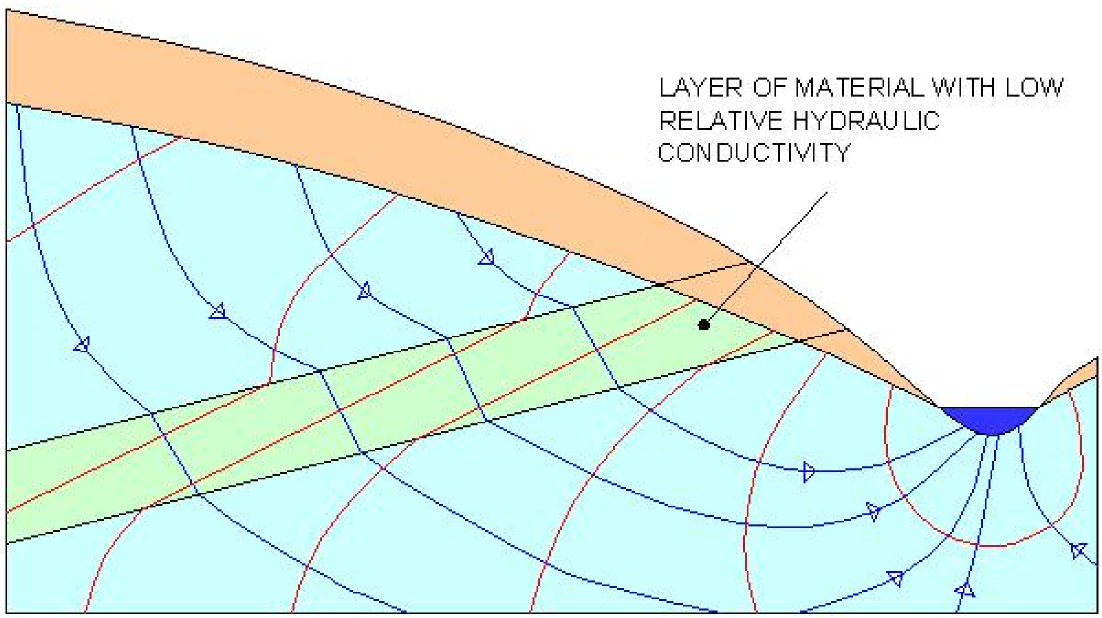


FIGURE 13. Example of sub-soil flow through heterogeneous media - Case I

4. Example of flow through a heterogeneous media, case II.

This case (Figure 14) is just opposite to that shown in example 3. Here, the flow is through a sub-soil material of high hydraulic conductivity sandwiched between materials of relatively low hydraulic conductivities.

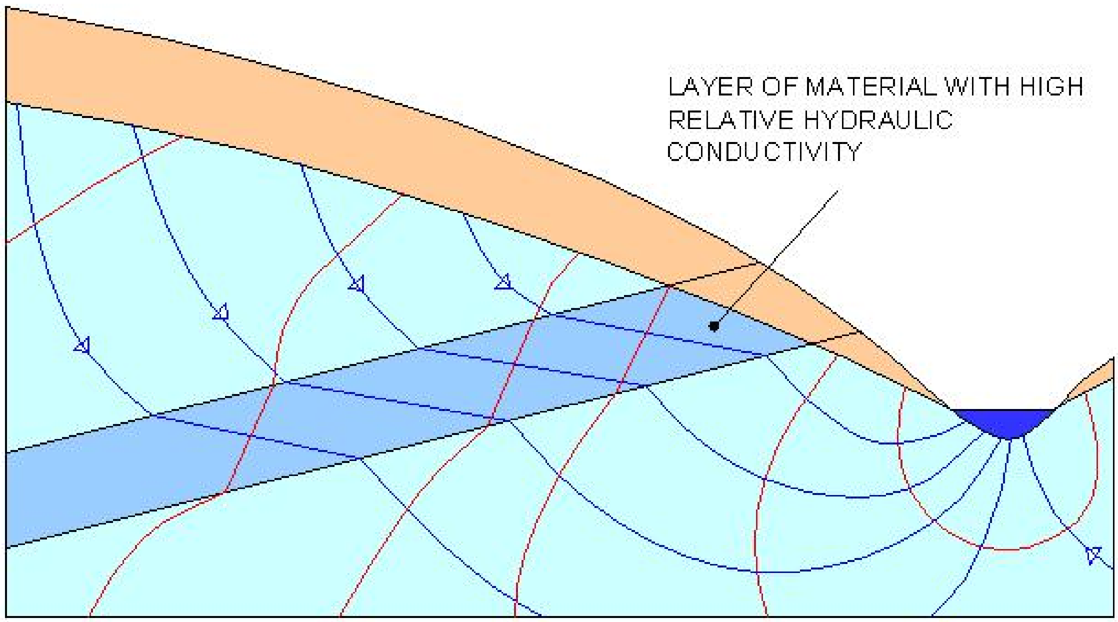


FIGURE 14. Example of sub-soil flow throuhg heterogenous media - Case II

2.5.6 Water table contours and regional flow

For a region, like a watershed, if we plot (in a horizontal plane) contours of equal hydraulic head of the ground water, then we can analyse the movement of ground water in a regional scale. Figure 15 illustrates the concept, assuming homogeneous porous media in the region for varying degrees of hydraulic conductivity (which is but natural for a real setting).

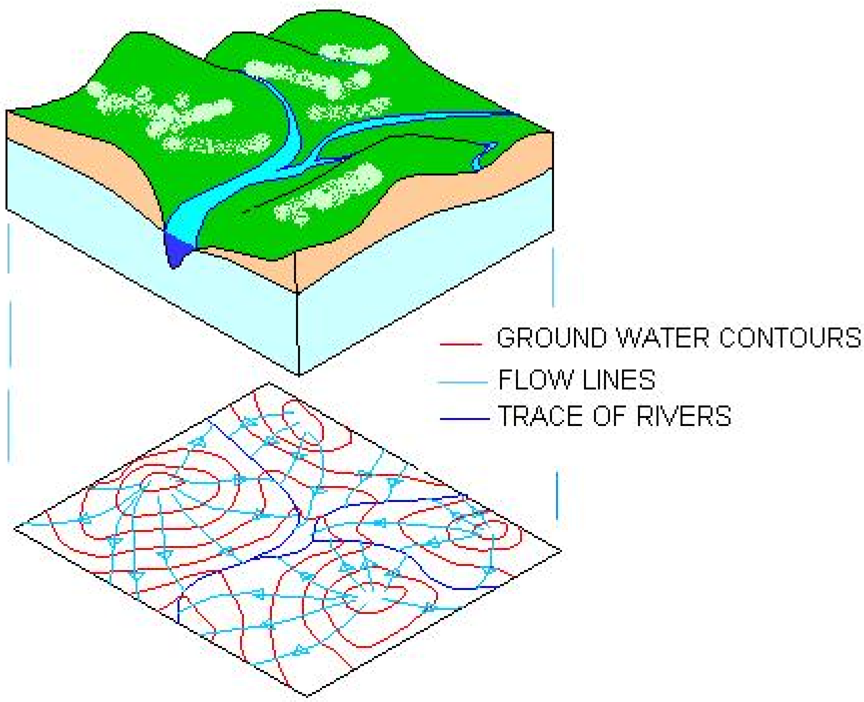


FIGURE 15. Movement of ground water in a regional scale

2.5.7 Aquifer properties and ground water flow

Porosity

Ground water is stored only within the pore spaces of soils or in the joints and fractures of rock which act as a aquifers. The porosity of an earth material is the percentage of the rock or soil that is void of material. It is defined mathematically by the equation

$$n = \frac{100 v_v}{v} \tag{2}$$

Where n is the porosity, expressed as percentage; v_v is the volume of void space in a unit volume of earth material; and v is the unit volume of earth material, including both voids and solid.

Specific Yield

While porosity is a measure of the water bearing capacity of the formation, all this water cannot be drained by gravity or by pumping from wells, as a portion of the water is held in the void spaces by molecular and surface tension forces. If gravity exerts a stress on a film of water surrounding a mineral grain (forming the soil), some of the film will pull away and drip downward. The remaining film will be thinner, with a greater surface tension so that, eventually, the stress of gravity will be exactly balanced by the surface tension (Hygroscopic water is the moisture clinging to the soil particles because of surface tension). Considering the above phenomena, the Specific Yield (S_y) is the ratio of the volume of water that drains from a saturated soil or rock owing to the attraction of gravity to the total volume of the aquifer.

If two samples are equivalent with regard to porosity, but the average grain size of one is much smaller than the other, the surface area of the finer sample will be larger. As a result, more water can be held as hygroscopic moisture by the finer grains.

The volume of water retained by molecular and surface tension forces, against the force of gravity, expressed as a percentage of the volume of the saturated sample of the aquifer, is called Specific Retention S_r, and corresponds to what is called the Field Capacity.

Hence, the following relation holds good:

$$n = S_y + S_r \qquad (3)$$

Specific storage (s_s)

Specific storage (s_s), also sometimes called the Elastic Storage Coefficient, is the amount of water per unit volume of a saturated formation that is stored or expelled from storage owing to compressibility of the mineral skeleton and the pore water per unit change in potentiometric head. Specific Storage is given by the expression

$$S_s = \gamma(\alpha + n\beta) \qquad (4)$$

where γ is the unit weight of water, α is the compressibility of the aquifer skeleton; n is the porosity; β is the compressibility of water.

Specific storage has the dimensions of length^{-1}

The storativity (S) of a confined aquifer is the product of the specific storage (S_s) and the aquifer thickness (b).

$$S = bS_s \qquad (5)$$

All of the water released is accounted for by the compressibility of the mineral skeleton and pore water. The water comes from the entire thickness of the aquifer.

In an unconfined aquifer, the level of saturation rises or falls with changes in the amount of water in storage. As water level falls, water drains out from the pore spaces. This storage or release due to the specific yield (S_y) of the aquifer. For an unconfined aquifer, therefore, the storativity is found by the formula.

$$S = S_y + hS_s \qquad (6)$$

Where h is the thickness of the saturated zone.

Since the value of S_y is several orders of magnitude greater than hS_s for an unconfined aquifer, the storativity is usually taken to be equal to the specific yield.

2.5.8 Aquifers and confining layers

It is natural to find the natural geologic formation of a region with varying degrees of hydraulic conductivities. The permeable materials have resulted usually due to weathering, fracturing and solution effects from the parent bed rock. Hence, the physical size of the soil grains or the pre sizes of fractured rock affect the movement of ground water flow to a great degree. Based on these, certain terms that have been used frequently in studying hydrogeology, are discussed here.

- **Aquifer:** This is a geologic unit that can store and transmit water at rates fast enough to supply reasonable amount to wells.

- **Confining layers:** This is a geologic unit having very little hydraulic conductivity. Confining layers are further subdivided as follows:

    - Aquifuge: an absolutely impermeable layer that will not transmit any water.

    - Aquitard: A layer of low permeability that can store ground water and also transmit slowly from one aquifer to another. Also termed as "leaky aquifer'.

    - Aquiclude: A unit of low permeability, but is located so that it forms an upper or lower boundary to a ground water flow system.

Aquifers which occur below land surface extending up to a depth are known as unconfined. Some aquifers are located much below the land surface, overlain by a confining layer. Such aquifers are called confined or artesian aquifers. In these aquifers, the water is under pressure and there is no free water surface like the water table of unconfined aquifer.

Module 2

The Science of Surface and Ground Water

Lesson
6
Principles of Ground Water Flow

Instructional Objectives

On completion of the lesson, the student shall be learn

1. The description of steady state of ground water flow in the form of Laplace equation derived from continuity equation and Darcy's law for ground water movement.
2. The quantitative description of unsteady state ground water flow.
3. The definition of the terms Specific Yield and Specific Storage and their relationship with Storativity of a confined aquifer.
4. The expressions for ground water flow in unconfined and confined aquifers, written in terms of Transmissivity.
5. Expression for two – dimensional flow in unconfined and confined aquifers; Boussinesq equation.
6. Expression for two – dimensional seepage flow below dams.
7. Analytical solution of steady one dimensional flow in simple cases of confined and unconfined aquifers.

2.6.0 Introduction

In the earlier lesson, qualitative assessment of subsurface water whether in the unsaturated or in the saturated ground was made. Movement of water stored in the saturated soil or fractured bed rock, also called aquifer, was seen to depend upon the hydraulic gradient. Other relationships between the water storage and the portion of that which can be withdrawn from an aquifer were also discussed. In this lesson, we derive the mathematical description of saturated ground water flow and its exact and approximate relations to the hydraulic gradient.

2.6.1 Continuity equation and Darcy's law under steady state conditions

Consider the flow of ground water taking place within a small cube (of lengths Δx, Δy and Δz respectively the direction of the three areas which may also be called the elementary control volume) of a saturated aquifer as shown in Figure 1.

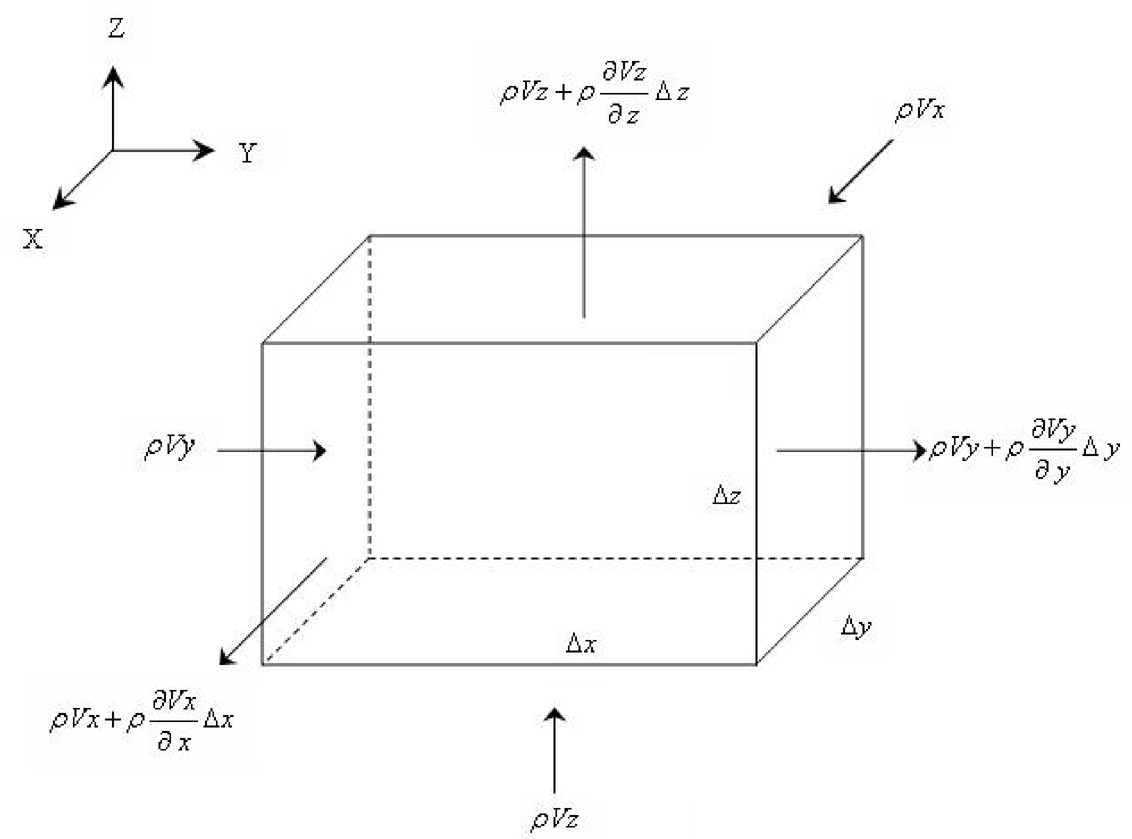


FIGURE 1. Infinitesimal cube for deriving the equation of continuity of flow of ground water

It is assumed that the density of water (ϱ) does not change in space along the three directions which implies that water is considered incompressible. The velocity components in the x, y and z directions have been denoted as v_x, v_y, v_z respectively.

Since water has been considered incompressible, the total incoming water in the cuboidal volume should be equal to that going out. Defining inflows and outflows as:

Inflows:

In x-direction: $\varrho\ v_x\ (\Delta y.\Delta x)$

In y-direction: $\varrho\ v_y\ (\Delta x.\Delta z)$

In z-direction: $\varrho\ v_z\ (\Delta x.\Delta y)$

Outflows:

In X-direction: $\varrho\ (v_x + \frac{\partial v\,x}{\partial x}\Delta x\ \ \Delta x)\ (\Delta y.\Delta z)$

In Y-direction: ϱ ($v_y + \frac{\partial v\, y}{\partial y} \Delta\, y$) (Δx.Δz)

In Z-direction: ϱ ($v_z + \frac{\partial v\, z}{\partial z}$.Δz) (Δy.Δx)

The net mass flow per unit time through the cube works out to:

$$\left[\frac{\partial\, v_x}{\partial\, x} + \frac{\partial\, v_y}{\partial\, y} + \frac{\partial\, v_z}{\partial\, z} \right] (\Delta x.\Delta y.\Delta z)$$

Or

$$\frac{\partial\, v_x}{\partial\, x} + \frac{\partial\, v_y}{\partial\, y} + \frac{\partial\, v_z}{\partial\, z} = 0 \qquad (2)$$

This is continuity equation for flow. But this water flow, as we learnt in the previous lesson, is due to a difference in potentiometric head per unit length in the direction of flow. A relation between the velocity and potentiometric gradient was first suggested by Henry Darcy, a French Engineer, in the mid nineteenth century. He found experimentally (see figure below) that the discharge 'Q' passing through a tube of cross sectional area 'A' filled with a porous material is proportional to the difference of the hydraulic head 'h' between the two end points and inversely proportional to the flow length'L'.

It may be noted that the total energy (also called head, h) at any point in the ground water flow per unit weight is given as

$$h = Z + \frac{p}{\gamma} + \frac{v^2}{2g} \qquad (3)$$

Where

Z is the elevation of the point above a chosen datum;

$\frac{p}{\gamma}$ is the pressure head, and

$\frac{v^2}{2g}$ is the velocity head

Since the ground water flow velocities are usually very small, $\frac{v^2}{2g}$ is neglected and h = Z+$\frac{p}{\gamma}$ is termed as the potentiometric head (or piezometric head in some texts)

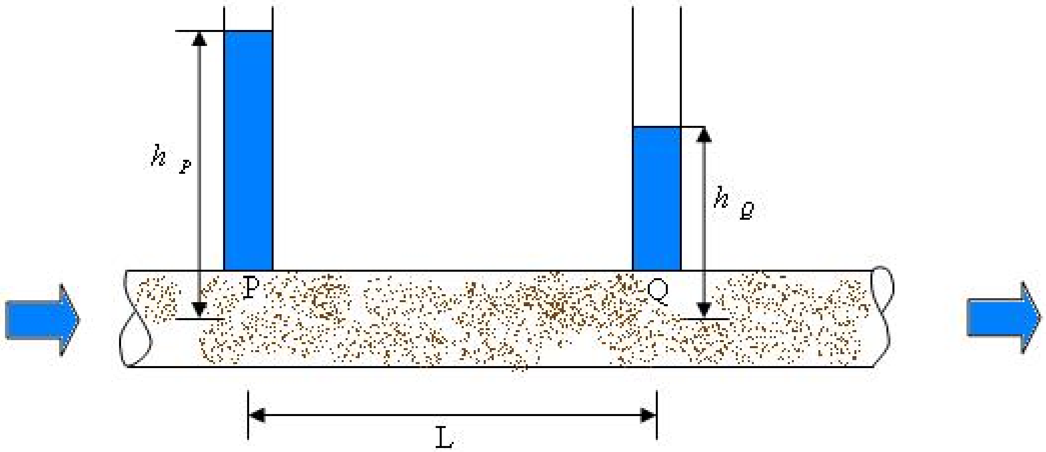


FIGURE 2. Flow through a saturated porous medium

Thus

$$Q \ \alpha \ A \bullet \left(\frac{h_P - h_Q}{L}\right) \quad (4)$$

Introducing proportionality constant K, the expression becomes

$$Q = K.A. \left(\frac{h_P - h_Q}{L}\right) \quad (5)$$

Since the hydraulic head decreases in the direction of flow, a corresponding differential equation would be written as

$$Q = -K.A. \left(\frac{dh}{dl}\right) \quad (6)$$

Where (dh/dl) is known as hydraulic gradient.

The coefficient 'K' has dimensions of L/T, or velocity, and as seen in the last lesson this is termed as the hydraulic conductivity.

Thus the velocity of fluid flow would be:

$$v = \frac{Q}{A} = -K\left(\frac{dh}{dl}\right) \qquad (7)$$

It may be noted that this velocity is not quite the same as the velocity of water flowing through an open pipe. In an open pipe, the entire cross section of the pipe conveys water. On the other hand, if the pipe is filed with a porous material, say sand, then the water can only flow through the pores of the sand particles. Hence, the velocity obtained by the above expression is only an apparent velocity, with the actual velocity of the fluid particles through the voids of the porous material is many time more. But for our analysis of substituting the expression for velocity in the three directions x, y and z in the continuity relation, equation (2) and considering each velocity term to be proportional to the hydraulic gradient in the corresponding direction, one obtains the following relation

$$\frac{\partial}{\partial x}\left(K_x \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial x}\left(K_y \frac{\partial h}{\partial y}\right) + \frac{\partial}{\partial z}\left(K_z \frac{\partial h}{\partial z}\right) = 0 \qquad (8)$$

Here, the hydraulic conductivities in the three directions (K_x, K_y and K_z) have been assumed to be different as for a general anisotropic medium. Considering isotropic medium with a constant hydraulic conductivity in all directions, the continuity equation simplifies to the following expression:

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2} = 0 \qquad (9)$$

In the above equation, it is assumed that the hydraulic head is not changing with time, that is, a steady state is prevailing.

If now it is assumed that the potentiometric head changes with time at the location of the control volume, then there would be a corresponding change in the ***porosity*** of the aquifer even if the fluid density is assumed to be unchanged.

What happens to the continuity relation is discussed in the next section.

Important term:

Porosity: It is ratio of volume of voids to the total volume of the soil and is generally expressed as percentage.

2.6.2 Ground water flow equations under unsteady state

For an unsteady case, the rate of mass flow in the elementary control volume is given by:

$$\rho\left[\frac{\partial v_x}{\partial x}+\frac{\partial v_y}{\partial y}+\frac{\partial v_z}{\partial z}\right]\Delta x\,\Delta y\,\Delta z=\frac{\partial M}{\partial t} \tag{10}$$

This is caused by a change in the hydraulic head with time plus the porosity of the media increasing accommodating more water. Denoting porosity by the term 'n', a change in mass 'M' of water contained with respect to time is given by

$$\frac{\partial M}{\partial t}=\frac{\partial}{\partial t}\left(\rho n\,\Delta x\,\Delta y\,\Delta z\right) \tag{11}$$

Considering no lateral strain, that is, no change in the plan area $\Delta x.\Delta y$ of the control volume, the above expression may be written as:

$$\frac{\partial M}{\partial t}=\frac{\partial \rho}{\partial t}\left(n\Delta x\,\Delta y\,\Delta z\right)+\frac{\partial}{\partial t}\left(n.\Delta z\right).\rho\,\Delta x\,\Delta y \tag{12}$$

Where the density of water (ρ).. is assumed to change with time. Its relation to a change in volume of the water V_w, contained within the void is given as:

$$\frac{d(V_w)}{V_w}=-\frac{d\rho}{\rho} \tag{13}$$

The negative sign indicates that a reduction in volume would mean an increase in the density from the corresponding original values.

The compressibility of water, β, is defined as:

$$\beta=-\frac{\left[\dfrac{d\,(Vw)}{Vw}\right]}{dp} \tag{14}$$

Where 'dp' is the change in the hydraulic head 'p' Thus,

$$\beta=\frac{d\rho}{\rho dp} \tag{15}$$

That is,

$$d\rho = \rho \, dp \, \beta \tag{16}$$

The compressibility of the soil matrix, α, is defined as the inverse of E_S, the elasticity of the soil matrix. Hence

$$\frac{1}{\alpha} = E_S = -\frac{d(\sigma_Z)}{\frac{d(\Delta z)}{\Delta z}} \tag{17}$$

Where σ_Z is the stress in the grains of the soil matrix.

Now, the pressure of the fluid in the voids, p, and the stress on the solid particles, σ_Z, must combine to support the total mass lying vertically above the elementary volume. Thus,

$$p+\sigma_z = \text{constant} \tag{18}$$

Leading to

$$d\sigma_z = -dp \tag{19}$$

Thus,

$$\frac{1}{\alpha} = -\frac{dp}{\frac{d(\Delta z)}{\Delta z}} \tag{20}$$

Also since the potentiometric head 'h' given by

$$h = \frac{p}{\gamma} + Z \tag{21}$$

Where Z is the elevation of the cube considered above a datum. We may therefore rewrite the above as

$$\frac{dh}{dz} = \frac{1}{\gamma}\frac{dp}{dz} + 1 \tag{22}$$

First term for the change in mass 'M' of the water contained in the elementary volume, Equation 12, is

$$\frac{\partial p}{\partial t} . n . \Delta x \, \Delta y \, \Delta z \tag{23}$$

This may be written, based on the derivations shown earlier, as equal to

$$n . \rho . \beta . \frac{\partial p}{\partial t} . \Delta x \, \Delta y \, \Delta z \tag{24}$$

Also the volume of soil grains, V_S, is given as

$$V_S = (1\text{-}n) \ \Delta x \, \Delta y \, \Delta z \tag{25}$$

Thus,

$$dV_S = [\, d\,(\Delta z) - d\,(n\, \Delta z)\,]\, \Delta x\, \Delta y$$
(26)

Considering the compressibility of the soil grains to be nominal compared to that of the water or the change in the porosity, we may assume dV_S to be equal to zero. Hence,

$$[\, d\,(\Delta z) - d\,(n\, \Delta z)\,]\, \Delta x\, \Delta y = 0 \tag{27}$$

Or,

$$d\,(\Delta z) = d\,(n\, \Delta z) \tag{28}$$

Which may substituted in second term of the expression for change in mass, M, of the elementary volume, changing it to

$$\begin{aligned} & \frac{\partial (n\, \Delta z)}{\partial t} \rho \, \Delta x \, \Delta y \\ & = \frac{\partial (\Delta z)}{\partial t} \rho \, \Delta x \, \Delta y \\ & = \rho \frac{\frac{\partial (\Delta z)}{\Delta z}}{\partial t} \Delta x \Delta y \Delta z \\ & = \rho \, \alpha \frac{\partial p}{\partial t} \Delta x \, \Delta y \, \Delta z \end{aligned} \tag{29}$$

Thus, the equation for change of mass, M, of the elementary cubic volume becomes

$$\frac{\partial M}{\partial t} = (\alpha + n\beta).\rho \frac{\partial p}{\partial t} \Delta x \, \Delta y \, \Delta z \qquad (30)$$

Combining Equation (30) with the continuity expression for mass within the volume, equation (10), gives the following relation:

$$-\rho \left[\frac{\partial v_x}{\partial x} + \frac{\partial v_y}{\partial y} + \frac{\partial v_z}{\partial z} \right] = (\alpha + n\beta)\, \rho \frac{\partial p}{\partial t} \qquad (31)$$

Assuming isotropic media, that is, K_X=K=$_Y K_Z$=K and applying Darcy's law for the velocities in the three directions, the above equation simplifies to

$$K \left[\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2} \right] = (\alpha + n\beta) \, \frac{\partial p}{\partial t} \qquad (32)$$

Now, since the potentiometric (or hydraulic) head h is given as

$$h = \frac{p}{\gamma} + z \qquad (33)$$

The flow equation can be expressed as

$$\left[\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2} \right] = (\alpha + n\beta) \, \frac{\gamma}{K} \frac{\partial h}{\partial t} \qquad (34)$$

The above equation is the general expression for the flow in three dimensions for an isotropic homogeneous porous medium. The expression was derived on the basis of an elementary control volume which may be a part of an unconfined or a confined aquifer. The next section looks into the simplifications for each type of aquifer.

2.6.3 Ground water flow expressions for ground water flow unconfined and confined aquifers

Unsteady flow takes place in an unconfined and confined aquifer would be either due to:

- Change in hydraulic head (for unconfined aquifer) or potentiometric head (for confined aquifer) with time.
- And, or compressibility of the mineral grains of the soil matrix forming the aquifer
- And, or compressibility of the water stored in the voids within the soil matrix

We may visually express the above conditions as shown in Figure 3, assuming an increase in the hydraulic (or potentiometric head) and a compression of soil matrix and pore water to accommodate more water

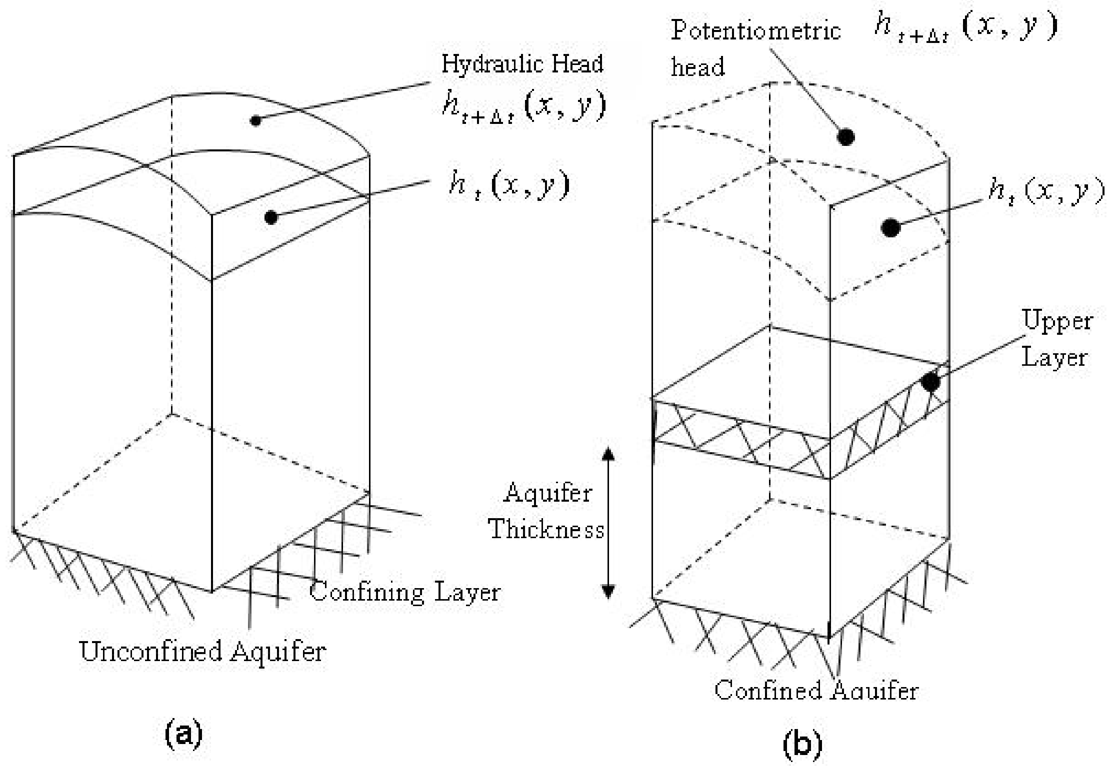


FIGURE 3. (a) Free surface of ground water table in unconfined aquifer
(b) Potentiometric surface in confined aquifer

Since ***storability S*** of a confined aquifer was defined as

$$S = b\gamma(\alpha + n\beta) \tag{35}$$

The flow equation for a confined aquifer would simplify to the following:

$$\left[\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2}\right] = \frac{S}{Kb}\frac{\partial h}{\partial t} \tag{36}$$

Defining the ***transmissivity T*** of a confined aquifer as a product of the hydraulic conductivity K and the saturated thickness of the aquifer, b, which is:

$$T = K \bullet b \tag{37}$$

The flow equation further reduces to the following for a confined aquifer

$$\left[\frac{\partial^2 h}{\partial x^2}+\frac{\partial^2 h}{\partial y^2}+\frac{\partial^2 h}{\partial z^2}\right]=\frac{S}{T}\frac{\partial h}{\partial t} \tag{38}$$

For unconfined aquifers, the ***storability S*** is given by the following expression

$$S = S_y + h\,S_s \tag{39}$$

Where S_y is the specific yield and S_s is the specific storage equal to $\gamma(\alpha + n\beta)$

Usually, S_s is much smaller in magnitude than S_y and may be neglected. Hence S under water table conditions for all practical purposes may be taken equal to S_y.

2.6.4 Two dimensional flow in aquifers

Under many situations, the water table variation (for unconfined flow) in areal extent is not much, which means that there the ground water flow does not have much of a vertical velocity component. Hence, a two – dimensional flow situation may be approximated for these cases. On the other hand, where there is a large variation in the water table under certain situation, a three dimensional velocity field would be the correct representation as there would be significant component of flow in the vertical direction apart from that in the horizontal directions. This difference is shown in the illustrations given in Figure 4.

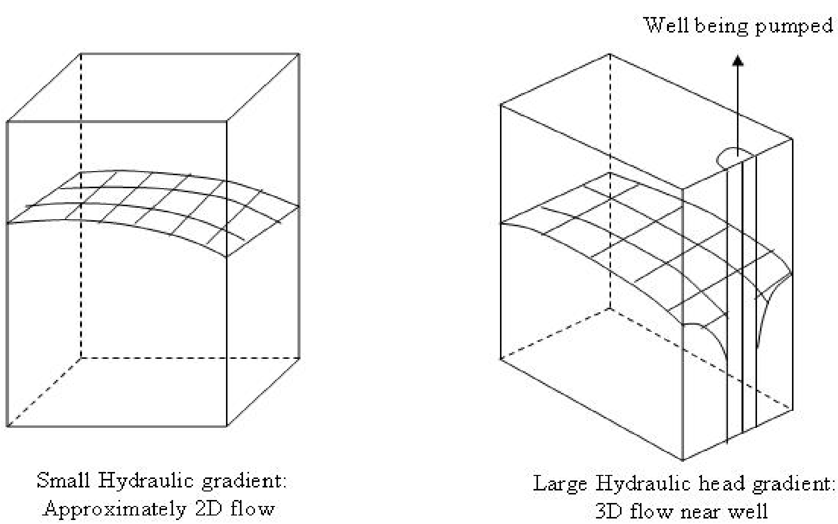


FIGURE 4. Difference between small and large hydraulic gradients of ground water table

In case of two dimensional flow, the equation flow for both unconfined and confined aquifers may be written as,

$$\left[\frac{\partial^2 h}{\partial x^2}+\frac{\partial^2 h}{\partial y^2}\right]=\frac{S}{T}\frac{\partial h}{\partial t} \qquad (40)$$

There is one point to be noted for unconfined aquifers for hydraulic head (or water table) variations with time. It is that the change in the saturated thickness of the aquifer with time also changes the transmissivity, T, which is a product of hydraulic conductivity K and the saturated thickness h. The general form of the flow equation for two dimensional unconfined flow is known as the Boussinesq equation and is given as

$$\frac{\partial}{\partial x}\left(h\frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(h\frac{\partial h}{\partial y}\right)=\frac{S_y}{K}\frac{\partial h}{\partial t} \qquad (41)$$

Where S_y is the specific yield.

If the drawdown in the unconfined aquifer is very small compared to the saturated thickness, the variable thickness of the saturated zone, h, can be replaced by an average thickness, *b*, which is assumed to be constant over the aquifer.

For confined aquifer under an unsteady condition though the potentiometric surface declines, the saturated thickness of the aquifer remains constant with time and is equal to an average value 'b'. Solving the ground water flow equations for flow in aquifers require the help of numerical techniques, except for very simple cases.

2.6.5 Two dimensional seepage flow

In the last section, examples of two dimensional flow were given for aquifers, considering the flow to be occurring, in general, in a horizontal plane. Another example of two dimensional flow would that be when the flow can be approximated. to be taking place in the vertical plane. Such situations might occur as for the seepage taking place below a dam as shown in Figure 5.

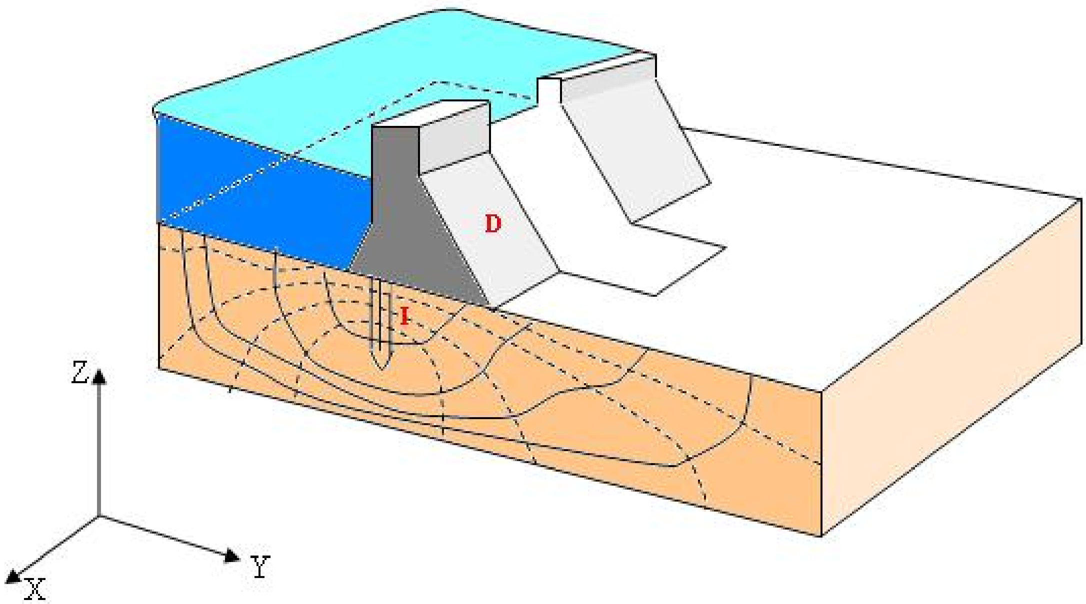


FIGURE 5. Seepage flow below a concrete gravity dam

Under steady state conditions, the general equation of flow, considering an isotropic porous medium would be

$$\frac{\partial^2 h}{\partial y^2}+\frac{\partial^2 h}{\partial z^2}=0 \qquad (42)$$

However, solving the above Equation (42) for would require advanced analytical methods or numerical techniques. More about seepage flow would be discussed in the later session.

2.6.6 Steady one dimensional flow in aquifers

Some simplified cases of ground water flow, usually in the vertical plane, can be approximated by one dimensional equation which can then be solved analytically. We consider the confined and unconfined aquifers separately, in the following sections.

2.6.6.1 Confined aquifers

If there is a steady movement of ground water in a confined aquifer, there will be a gradient or slope to the potentiometric surface of the aquifer. The gradient, again, would be decreasing in the direction of flow. For flow of this type, Darcy's law may be used directly.

Aquifer with constant thickness

This situation may be shown as in Figure 6.

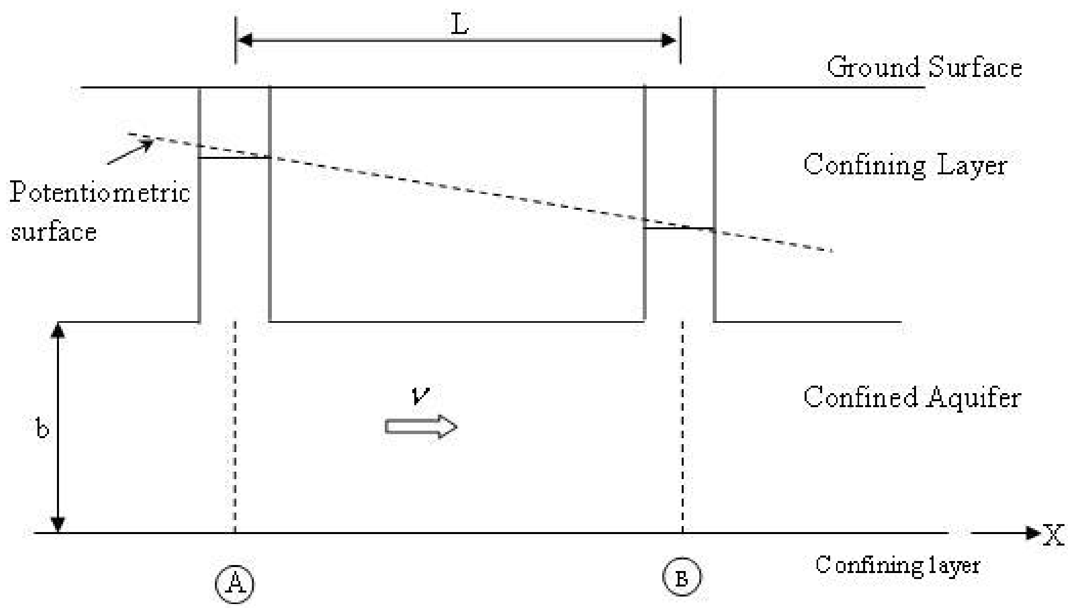


FIGURE 6. Flow through an aquifer of constant thckness

Assuming unit thickness in the direction perpendicular to the plane of the paper, the flow rate 'q' (per unit width) would be expressed for an aquifer of thickness'b'

$$q = b * 1 * v \tag{43}$$

According to Darcy's law, the velocity 'v' is given by

$$v = -K \frac{\partial h}{\partial x} \tag{44}$$

Where h, the potentiometric head, is measured above a convenient datum. Note that the actual value of 'h' is not required, but only its gradient $\frac{\partial h}{\partial x}$ in the direction of flow, x, is what matters. Here is K is the hydraulic conductivity

Hence,

$$q = b\,K \frac{\partial h}{\partial x} \tag{45}$$

The partial derivative of 'h' with respect to 'x' may be written as normal derivative since we are assuming no variation of 'h' in the direction normal to the paper. Thus

$$q = - b K \frac{d h}{d x} \qquad (46)$$

For steady flow, q should not vary with time, t, or spatial coordinate, x. hence,

$$\frac{d q}{d x} = -b K \frac{d^2 h}{d x^2} = 0 \qquad (47)$$

Since the width, b, and hydraulic conductivity, K, of the aquifer are assumed to be constants, the above equation simplifies to:

$$\frac{d^2 h}{d x^2} = 0 \qquad (48)$$

Which may be analytically solved as

$$h = C_1 x + C_2 \qquad (49)$$

Selecting the origin of coordinate x at the location of well A (as shown in Figure 6), and having a hydraulic head,h_A and also assuming a hydraulic head of well B, located at a distance L from well A in the x-direction and having a hydraulic head h_B, we have:

$$h_A = C_1.0+C_2 \text{ and}$$
$$h_B = C_1.L+C_2$$

Giving

$$C_1 = h - h_A /L \text{ and } C_2= h_A \qquad (50)$$

Thus the analytical solution for the hydraulic head 'h' becomes:

$$H = \frac{h_B - h_A}{L} x + h_A \qquad (51)$$

Aquifer with variable thickness

Consider a situation of one- dimensional flow in a confined aquifer whose thickness, b, varies in the direction of flow, x, in a linear fashion as shown in Figure 7.

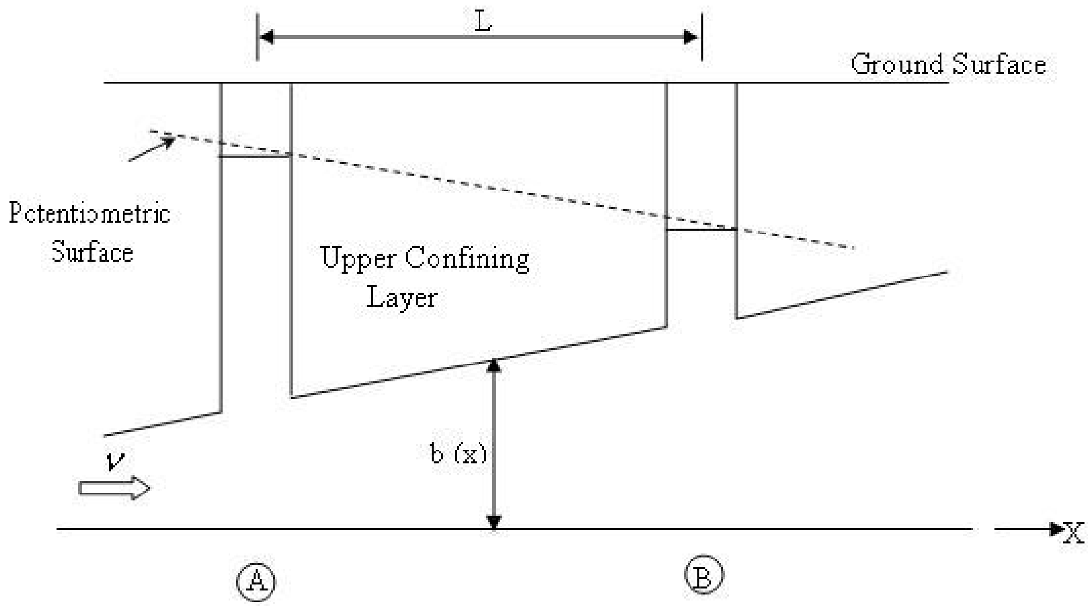


FIGURE 7. Flow through an aquifer with variable thickness

The unit discharge, q, is now given as

$$q = - b(x) K \frac{dh}{dx} \qquad (52)$$

Where K is the hydraulic conductivity and dh/dx is the gradient of the potentiometric surface in the direction of flow,x.

For steady flow, we have,

$$\frac{dq}{dx} = -K\left[\frac{db}{dx}\frac{dh}{dx} + b\frac{d^2h}{dx^2}\right] = 0 \qquad (53)$$

Which may be simplified, denoting $\frac{db}{dx}$ as b'

$$b\frac{d^2h}{dx^2} + b'\frac{dh}{dx} = 0 \qquad (54)$$

A solution of the above differential equation may be found out which may be substituted for known values of the potentiometric heads h_A and h_B in the two observation wells A and B respectively in order to find out the constants of integration.

2.6.6.2 Unconfined aquifers

In an unconfined aquifer, the saturated flow thickness, h is the same as the hydraulic head at any location, as seen from Figure 8:

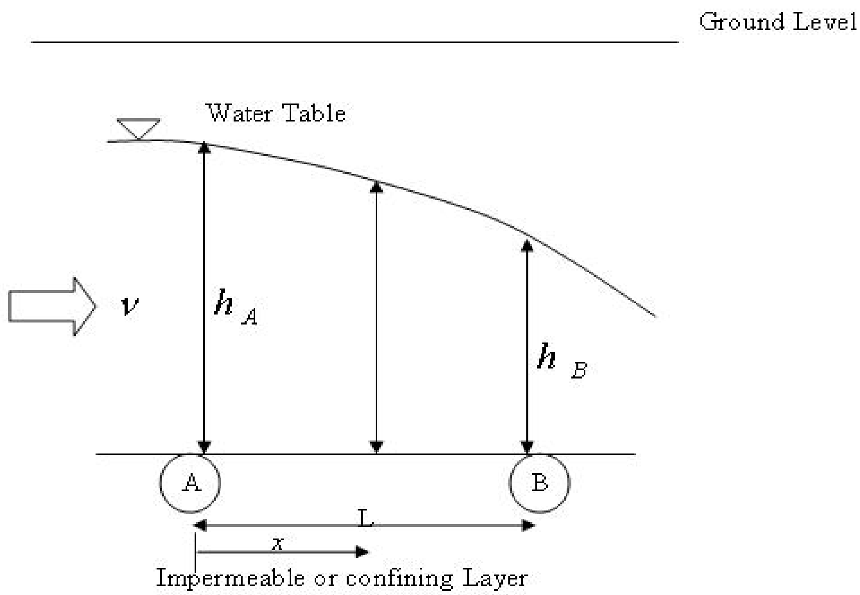


FIGURE 8. Flow through an unconfined aquifer

Considering no recharge of water from top, the flow takes place in the direction of fall of the hydraulic head, h, which is a function of the coordinate, x taken in the flow direction. The flow velocity, v, would be lesser at location A and higher at B since the saturated flow thickness decreases. Hence v is also a function of x and increases in the direction of flow. Since, v, according to Darcy's law is shown to be

$$v = K\frac{dh}{dx} \tag{55}$$

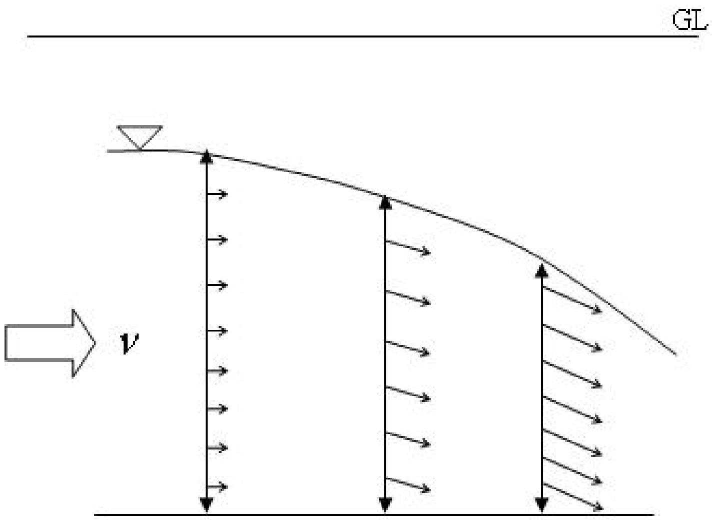
the gradient of potentiometric surface, dh/dx, would (in proportion to the velocities) be smaller at location A and steeper at location B. Hence the gradient of water table in unconfined flow is not constant, it increases in the direction of flow.

This problem was solved by J.Dupuit, a French hydraulician, and published in 1863 and his assumptions for a flow in an unconfined aquifer is used to approximate the flow situation called Dupuit flow. The assumptions made by Dupuit are:

- The hydraulic gradient is equal to the slope of the water table, and

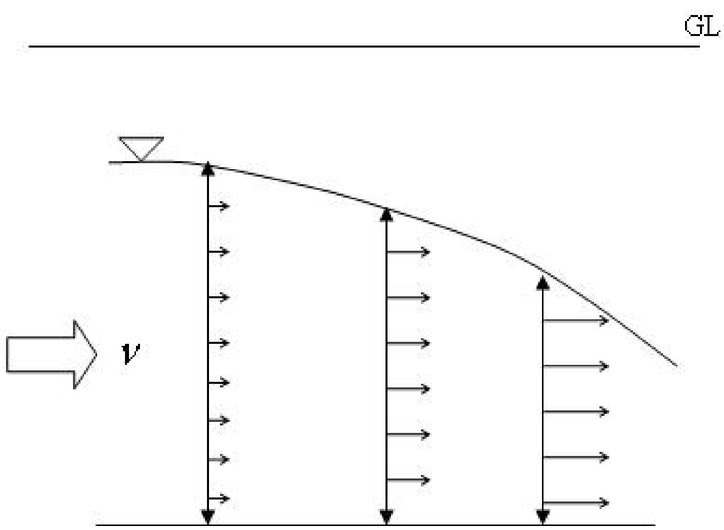
- For small water table gradients, the flow-lines are horizontal and the equipotential lines are vertical.

The second assumption is illustrated in Figure 9.



Actual Velocity pattern

(a)



Velocity pattern after Dupuit's assumptions

(b)

FIGURE 9. (a) Actual velocity pattern in ground water flow and
(b) Assumption of Dupuit regarding ground water flow

Solutions based on the Dupuit's assumptions have proved to be very useful in many practical purposes. However, the Dupuit assumption do not allow for a ***seepage face*** above an outflow side.

An analytical solution to the flow would be obtained by using the Darcy equation to express the velocity, v, at any point, x, with a corresponding hydraulic gradient $\frac{dh}{dx}$, as

$$v = -K \frac{dh}{dx} \tag{56}$$

Thus, the unit discharge, q, is calculated to be

$$q = -K\, h\, \frac{dh}{dx} \tag{57}$$

Considering the origin of the coordinate x at location A where the hydraulic head us h_A and knowing the hydraulic head h_B at a location B, situated at a distance L from A, we may integrate the above differential equation as:

$$\int_0^L q\, dx = -K \int_{h_A}^{h_B} h\, dh \tag{58}$$

Which, on integration, leads to

$$q\, x\Big|_0^L = -\, K\, .\frac{h^2}{2}\Big|_{hA}^{hB} \tag{59}$$

Or,

$$q\, .\, L = K \left[\frac{h_B{}^2}{2} - \frac{h_A{}^2}{2} \right] \tag{60}$$

Rearrangement of above terms leads to, what is known as the Dupuit equation:

$$q = -\frac{1}{2} K \left[\frac{h_B{}^2 - h_A{}^2}{L} \right] \tag{61}$$

An example of the application of the above equation may be for the ground water flow in a strip of land located between two water bodies with different water surface elevations, as in Figure 10.

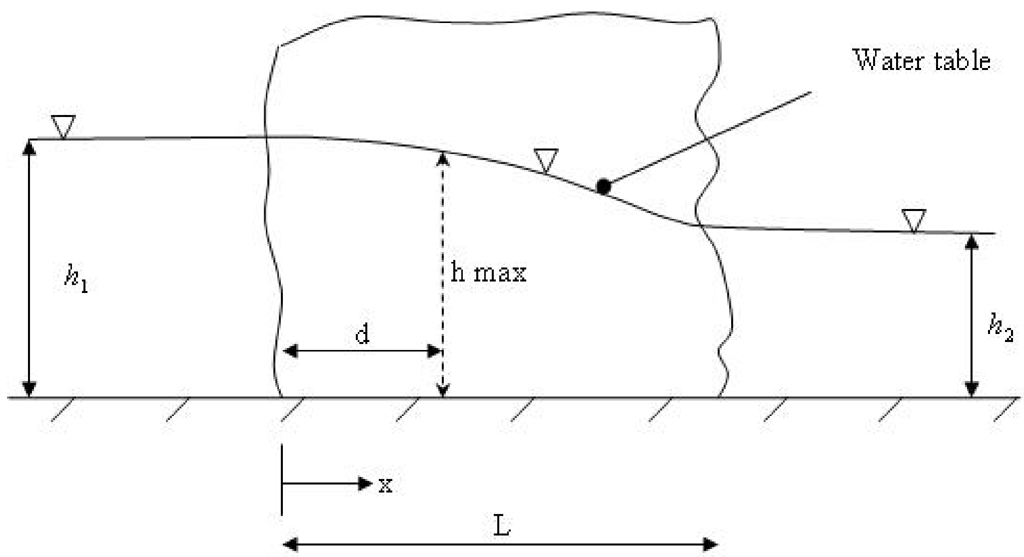


FIGURE 10. Ground water flow through a strip of land with difference in water surface elevation on either side.

The equation for the water table, also called the phreatic surface may be derived from Equation (61) as follows:

$$h = \sqrt{h_1^2 - \left(h_1^2 - h_2^2\right)\frac{x}{L}} \qquad (62)$$

In case of recharge due to a constant infiltration of water from above the water table rises to a many as shown in Figure 11:

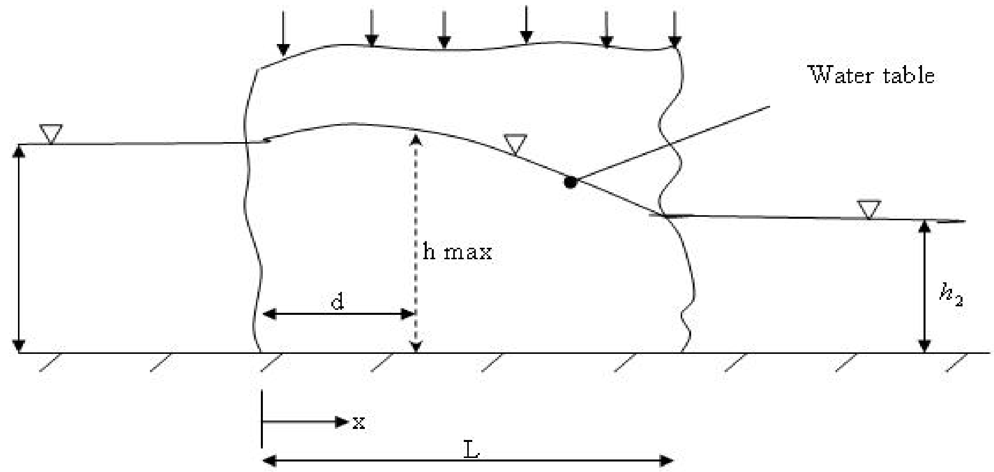


FIGURE 11. Ground water flow through a strip of land with infiltration from above

There is a difference with the earlier cases, as the flow per unit width, q, would be increasing in the direction of flow due to addition of water from above. The flow may be analysed by considering a small portion of flow domain as shown in Figure 12.

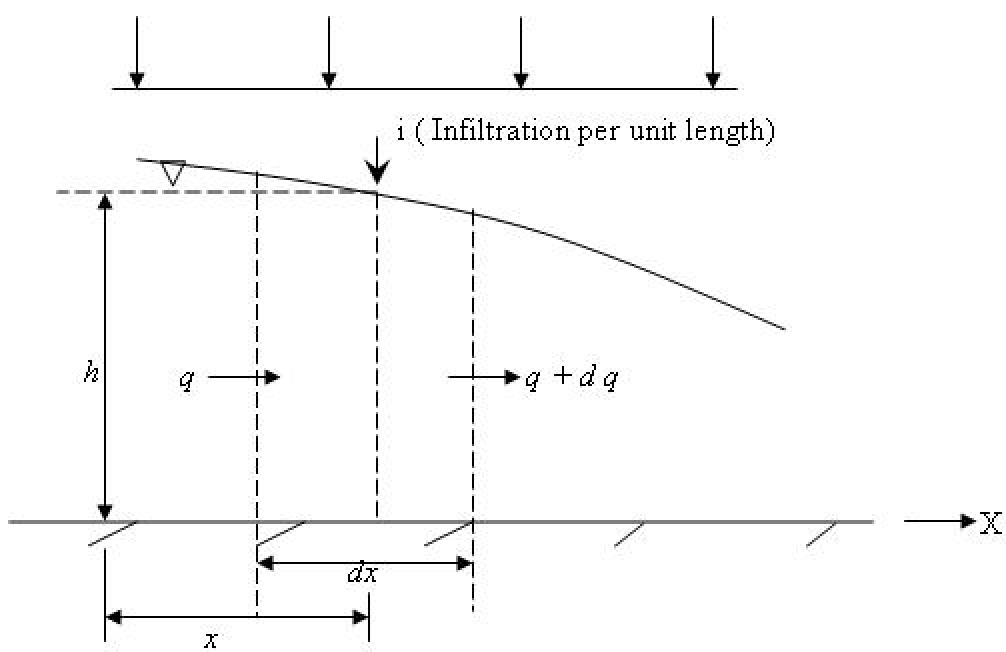


FIGURE 12. Definition of terms for flow analysis for the case shown in Figure 11

Considering the infiltration of water from above at a rate *i* per unit length in the direction of ground water flow, the change in unit discharge d_q is seen to be

$$d_q = i\,.dx \tag{63}$$

Or,

$$\frac{dq}{dx} = i \tag{64}$$

From Darcy's law,

$$q = -K.h.\frac{dh}{dx} = -\frac{1}{2}k\frac{d(h^2)}{dx} \tag{65}$$

$$\frac{dq}{dx} = -\frac{1}{2}K\frac{d^2\left(h^2\right)}{dx^2} \tag{66}$$

Substituting the expression for $\frac{dq}{dx}$, we have,

$$-\frac{1}{2}K\frac{d^2(h^2)}{dx^2} = i \tag{67}$$

Or,

$$\frac{d^2(h^2)}{dx^2} = \frac{2.i}{k} \tag{68}$$

The solution for this equation is of the form

$$Kh^2 + 2x^2 = C_1 x + C_2 \tag{69}$$

If, now, the boundary condition is applied as,

At x = 0, h = h_1, and
At x = L, h = h_2

The equation for the water table would be:

$$h = \sqrt{h_1^2 - \frac{\left(h_1^2 - h_1^2\right)}{L}x + \frac{i}{K}(L-x)x} \tag{70}$$

And,

$$q_x = q_0 + 2x \tag{71}$$

Where q_0 is the unit discharge at the left boundary, *x* = 0, and may be found out to be

$$q_0 = \frac{\left(h_1^2 - h_1^2\right)}{2L} - \frac{iL}{2} \tag{72}$$

Which gives an expression for unit discharge q_x at any point *x* from the origin as

$$q_x = K\frac{\left(h_1^2 - h_1^2\right)}{2L} - i\left(\frac{L}{2} - x\right) \tag{73}$$

For no recharge due to infiltration, *i* = 0 and the expression for q_x is then seen to become independent of x, hence constant, which is expected.

References

Raghunath, H M (2002) Ground Water (Second Edition), New Age International Pvt. Ltd.

Module 2

The Science of Surface and Ground Water

Lesson
7
Well Hydraulics

Instructional Objectives

At the end of this lesson, the student shall be able to learn the following:

1. The concepts of steady and unsteady ground water flow to wells
2. Mathematical equations for water flow to wells in confined aquifers
3. Mathematical equations for water flow to wells in unconfined aquifers
4. Determination of the physical properties of confined aquifers
5. Design and installation of tube wells

2.7.0 Introduction

As established in the earlier lessons, an enormous amount of water is stored within the ground. A small portion of that is in an unsaturated state but that below the water table, also called ground water, can be easily extracted for useful purpose, depending on the type and location within which the water exists. It has been roughly estimated that of the global water resources, about 0.6 percent exists as ground water, out of which about half can be economically extracted with the present drilling technology. In fact, the ground water is the largest source of fresh water on earth excluding the polar icecaps and glaciers. Hence, ground water has been extracted on all regions of the world for different purposes and about nearly one fifth of all the water used in the world is obtained from ground water sources.

Evidence of extraction of water from dug wells has been found in the archeological remnants of Mohenjodaro. In many of the cities established during the medieval ages in India, the main source of water was dug wells, though people were dependent on surface water bodies like rivers or, lakes, or ponds, if that happened to be nearby. It is only during the past century that tube wells became popular as an easily operatable source of extraction of ground water. Gradually with easy access to electricity deep tube wells have become a common source of water. However, establishment of tube well extraction of water involves knowledge about the movement of water through the geological formations, which has been discussed in Lessons 2.5 and 2.6. Water may have to be extracted from formations ranging from sand, silt, clay, fractured rocks of different compositions etc., A well may be dug to extract water from a confined or an unconfined aquifer. Digging of more than one well in close vicinity affects each others' yield as the drawdown of one influences the other. This may be quantitatively estimated by theories of ground water flow applied to the radial flow of water to each well. In this lesson, these theories are discussed, which would be helpful in designing such wells.

2.7.1 Steady flow and unsteady flows

Imagine a farmer using a deep tube or a dug well as a source of water for irrigating his field. The well may be fitted with a ***submergible pump*** or a ***centrifugal pump*** to draw out water and discharge at the head of a channel leading to the fields. As long as the pump is not in operation, the water in the well remains at a steady at a level, at that of the water table (Figure 1).

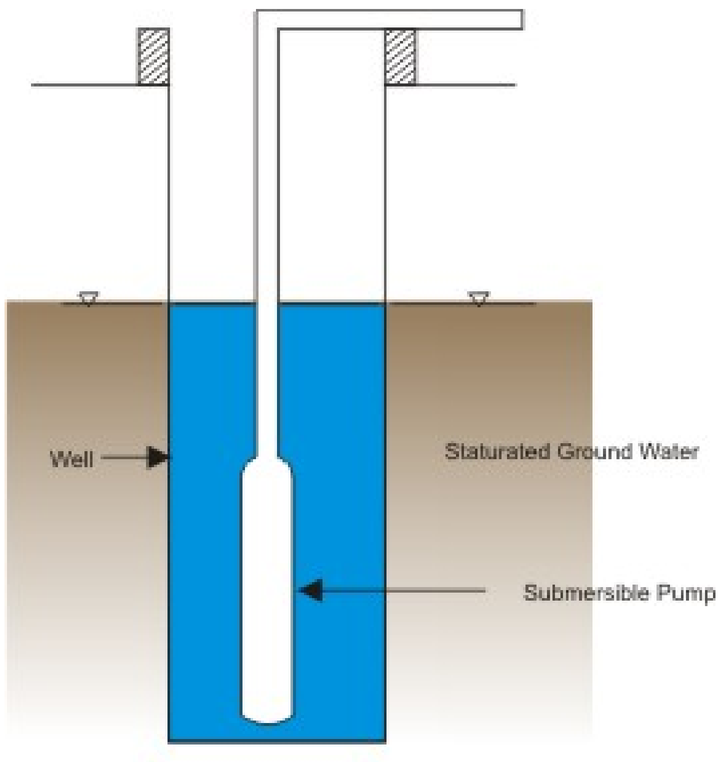


FIGURE 1. PUMP NOT STARTED

When the pump is just started, it starts drawing out water from the well and the level of water in the well decreases. The water table surrounding the well also gets lowered (Figure 2).

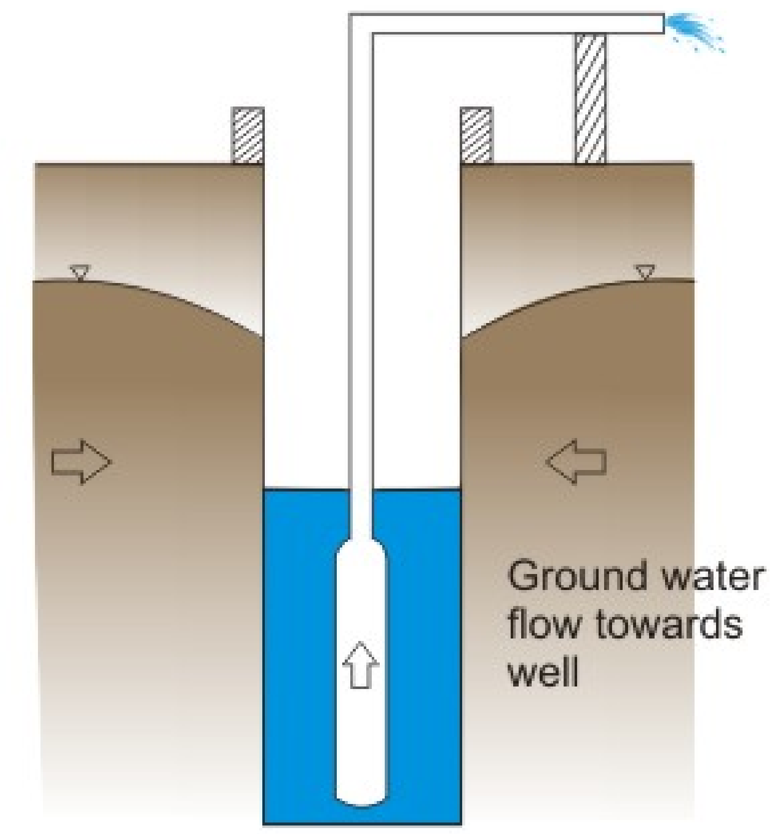


FIGURE 2. PUMP JUST STARTED

The water table gets lowered and forms a conical depression much like that shown in Figure 3.

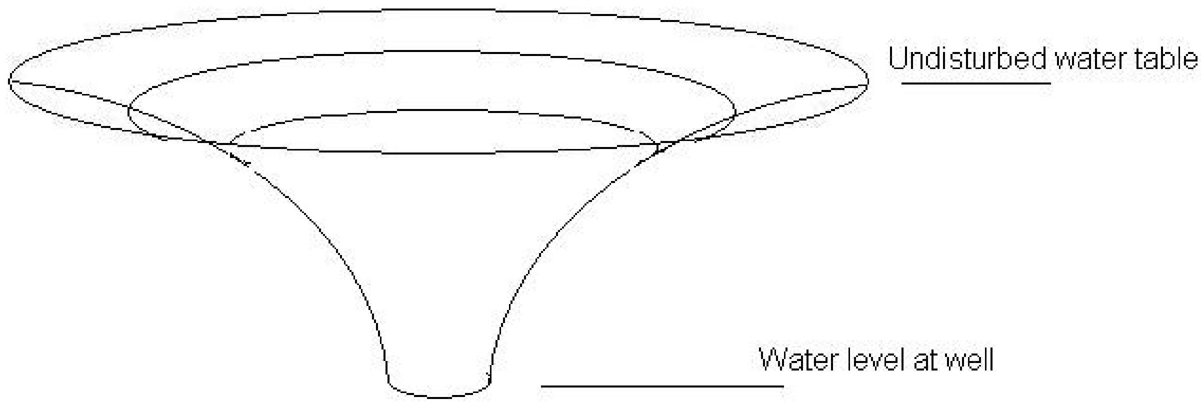


FIGURE 3. Cone of depression

It may be observed that the surface of the water table, shaped now in the form of a cone, is steepest where it meets the well. Farther away from the well, the surface is flatter and beyond a certain distance, called the radius of influence, the surface of the cone is almost as flat as the original water table.

As pumping continues (at the rated capacity of the pump), the water table gets lowered further until it becomes steady. At this position the water surface is called the draw down curve (Figure 4).

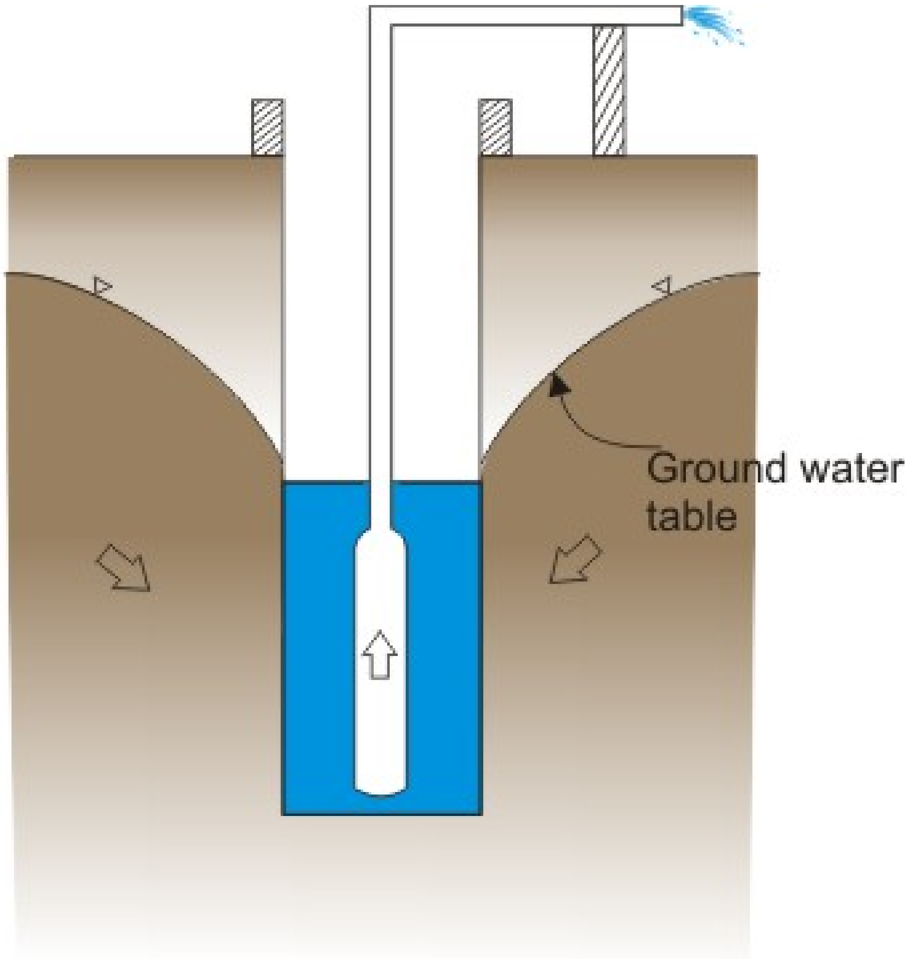


FIGURE 4. STEADY STATE DRAWDOWN

In must be observed that the water that is being pumped up from the well is being replenished by water traveling through the saturated formation towards the well. Further, if the capacity of the pump amount of water being in thrown from aware would be a lowered still.

Figures 1 to 4 depict a well that is drawing water from an unconfined aquifer. Corresponding figure of a steady state draw down curve in a confined aquifer would be as shown in Figure 5.

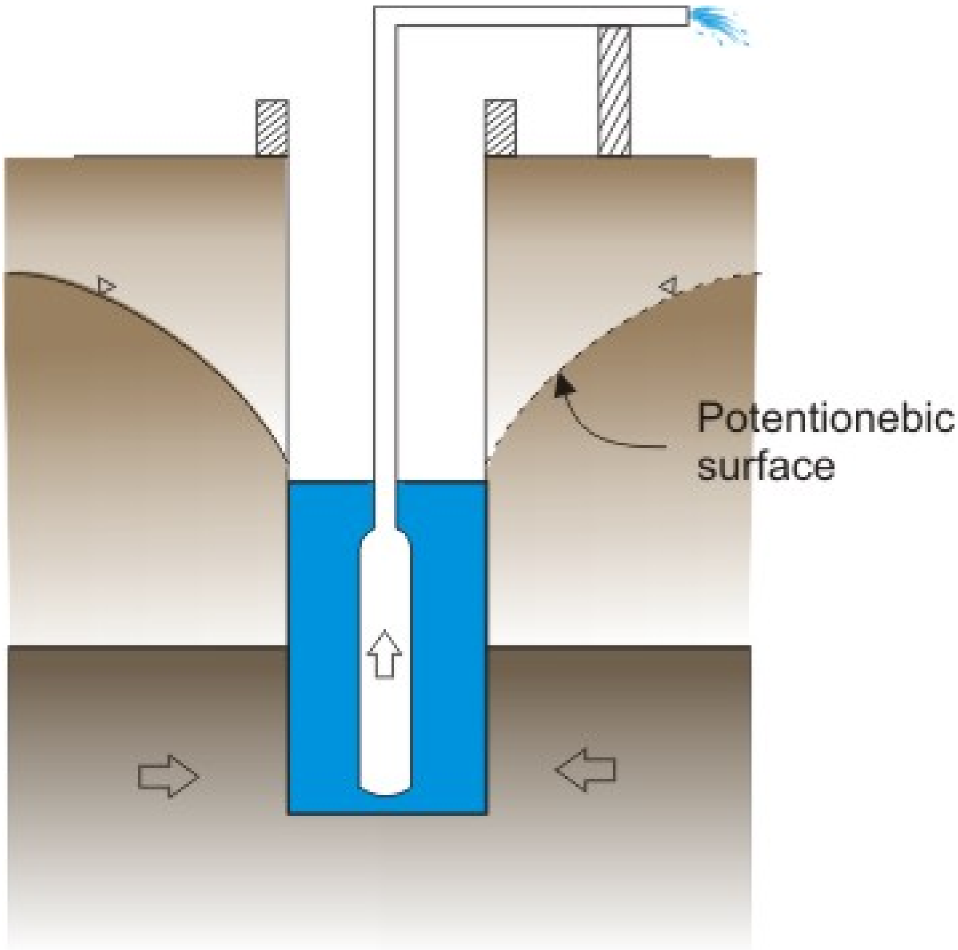


FIGURE 5. STEADY STATE POTENTIONETIC SURFACE FOR CONFINED AGNIFER

The following sections explain the mathematical relation between the water pumped and the location of the draw down. It must be remembered that the flow towards the wells is actually taking place radially. Hence, we shall be predominantly using the ground water flow equations using the cylindrical coordinates (*r*, *θ*, *z*, *w*) in contrast to the ones using Cartesian coordinates (*x, y, z*) as used in the previous lessons.

Steady and unsteady flow situations may further be classified as being confined or unconfined, depending on the relative positions of ground water conveying strata and the water table. The following sections describe each of these conditions individually.

2.7.2 Steady Flow to a Well: Confined Aquifer

Consider the case of a pumped well completely penetrating a confined aquifer (Figure 6). The corresponding steady state piezometric draw down surface is also shown for the assumed constant pumped discharge Q.

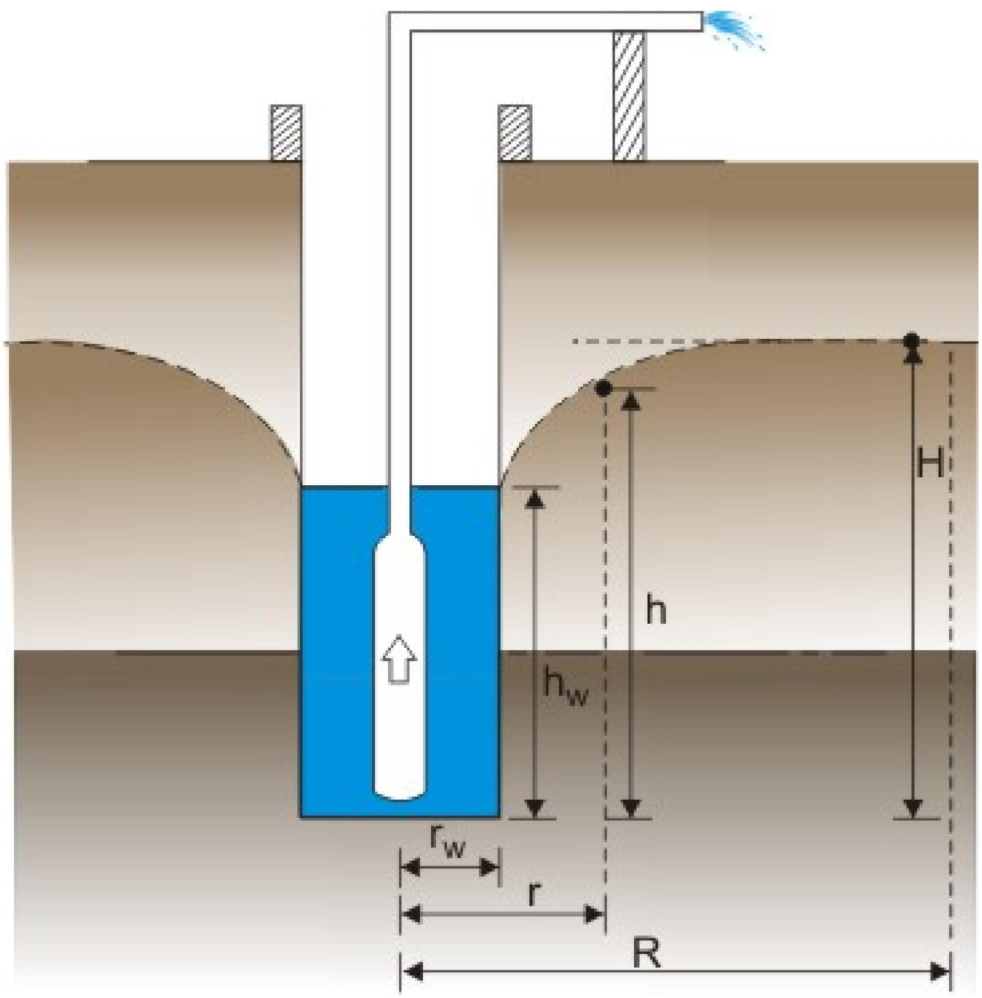


FIG 6. Definition of terms

The well is assumed to have a redius r_w and the radius of influence is thought to be R where the potentiometric surface is nearly equal to the original undisturbed value H, measured from a datum. At the well, the depth of water is designated by $\boldsymbol{h_w}$**,** which is also measured from a common datum. In general, at a certain radius r measured from the center of the well, the potentiometric surface stands at a height 'h' measured above the datum. The yield from the well Q may be expressed in terms of Darcy's law as,

$$Q = K\ i\ \mathrm{A} \qquad (1)$$

Where ***K*** is the coefficient of permeability of the formation, ***i*** is the hydraulic gradient that is, the slope of the potentiometric surface at the well bound and **A** is the surface area of the well through which the flow is converging into the well from the aquifer. Thus,

$$Q = K \left.\frac{dh}{dr}\right|_{r=r_w} \left(2\pi r_w b\right) \qquad (2)$$

In the above equation, ***b*** is the Thickness of the aquifer.

Naturally, the same amount of water also travels through the aquifer at a radial distance *r* from the center of the well. Thus, yield would also be

$$Q = K \frac{dh}{dr}\left(2\pi r\ b\right) \qquad (3)$$

The above expression is true if the aquifer thickness ***b*** is assumed to be constant throughout. The above equations give us a value of the yield, *Q*, of the well but for that a measure of the gradient of the potentiometric surface is essential. This may be done by inserting a piezometer penetrating into the aquifer and noting the water level there (Figure 7).

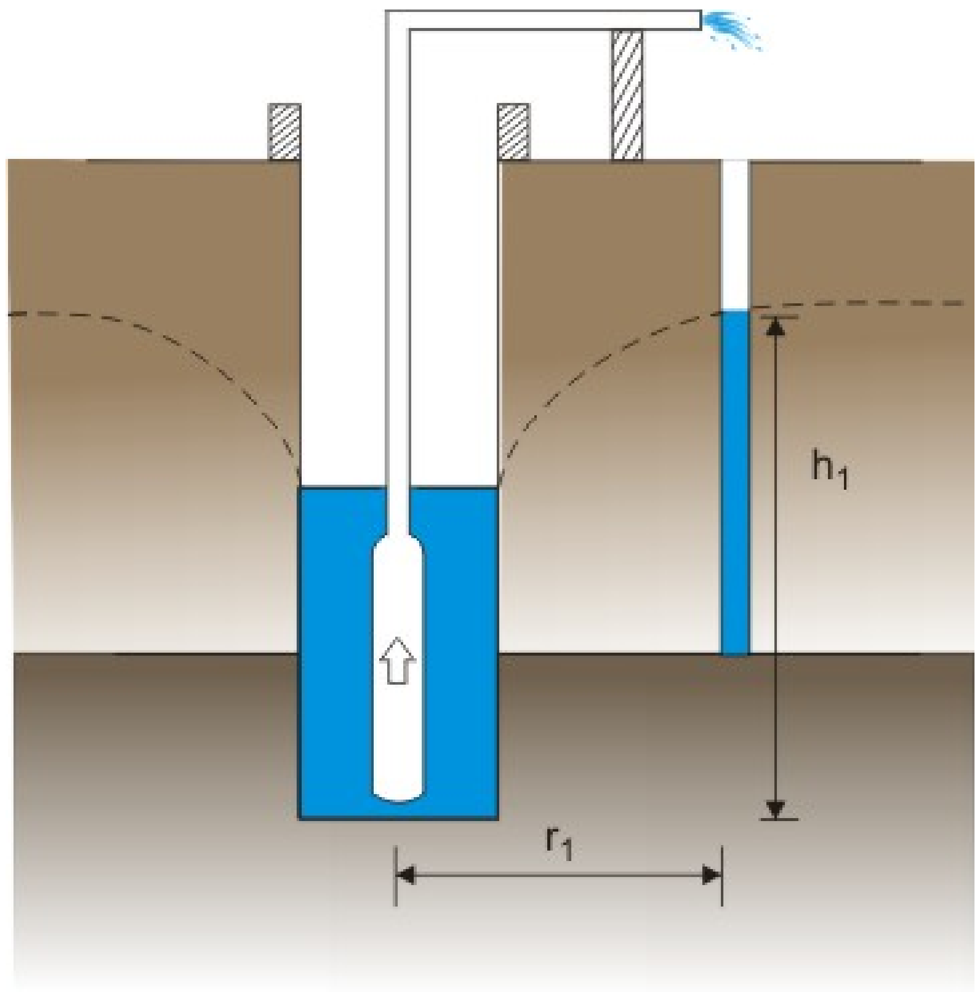


FIGURE .7 WATER VISE IN PIEZOMETER

Integrating (3) between the limits of $\boldsymbol{r_w}$ and $\boldsymbol{r_1}$, one obtains the following expressions:

$$h_1 - h_w = \frac{Q}{2\pi k b} \ln \frac{r_1}{r_w} \tag{4}$$

$$h_1 - h_w = \frac{2.3Q}{2\pi T} \ln \frac{r_1}{r_w} \tag{5}$$

Where ***T = Kb*** denotes the transmissibility of the aquifer.

This equation is known as equilibrium equation and can be used to determine variation of the potentiometric head radially outward from the well. The drawdown ***S*** at a radial distance ***r*** from the well (Figure 8) is given by

$$S = H_1 - h_1 = \frac{2.3Q}{2\pi T} \ln \frac{R}{r_1} \tag{6}$$

Where ***H*** is the undisturbed initial potentimetric surface and *R* is the radius of influence. If the drawdown ***S*** at distance r from the well known it is possible to work out ***T*** or ***K*** as

$$K = \frac{2.3Q}{2\pi S_1} \ln \frac{R}{r_1} \qquad (7)$$

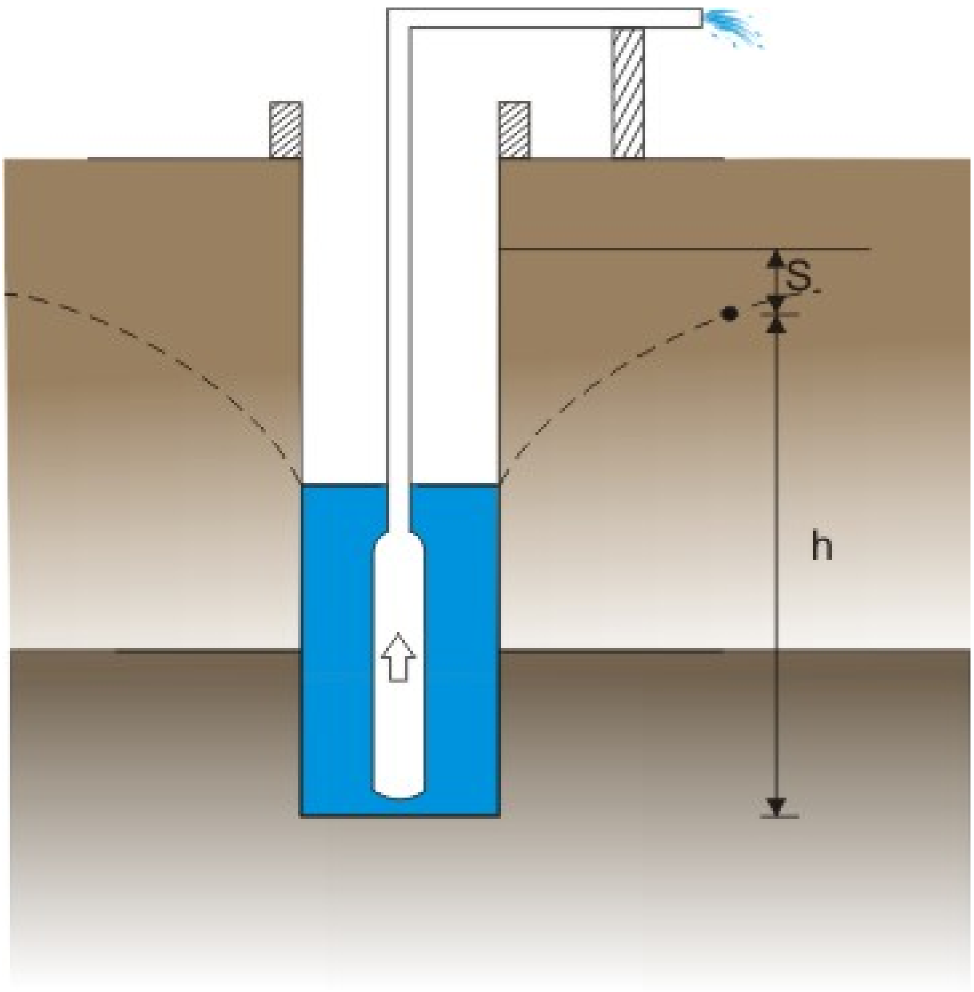


FIGURE .8 DEFINITION OF DRAWDOWN : S

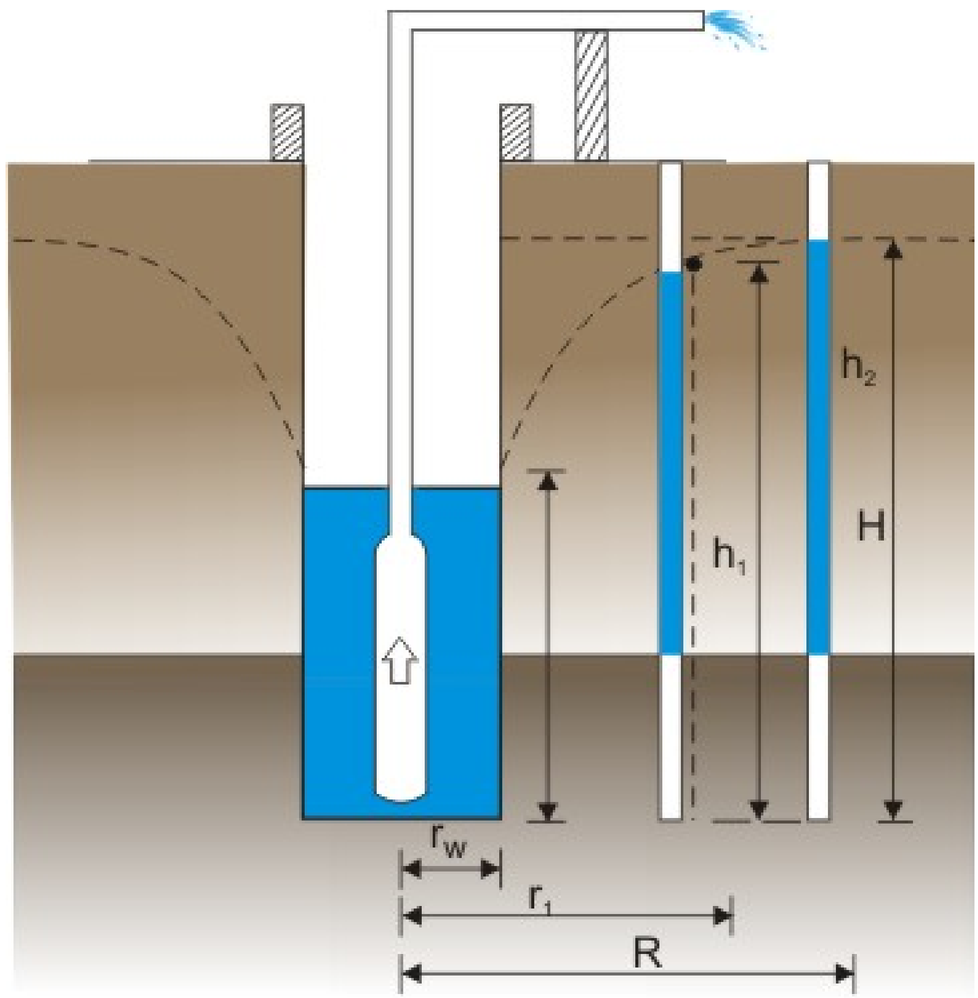


FIGURE . 9 DEFINITION OF TERMS

In case two piezometers are inserted near a well (Figure 9) and the piezometric head at these two places are given as ***h₁*** and ***h₂***, then the following expression is arrived at:

$$K = \frac{2.3Q}{2\pi b(h_2 - h_1)} \ln \frac{r_2}{r_1} \qquad (8)$$

This method of determining the permeability of an aquifer is known as Thiems method. Details about the method may be had from standard text books on ground water as the following:

Raghunath, H M (1998) ***Ground Water*****,** Second Edition, New Age International Publishers.

2.7.3 Steady Flow to a Well: Unconfined Aquifer

For the case of a pumped well located in an unconfined aquifer (Figure 10) the steady state discharge conditions are similar to that of confined aquifer.

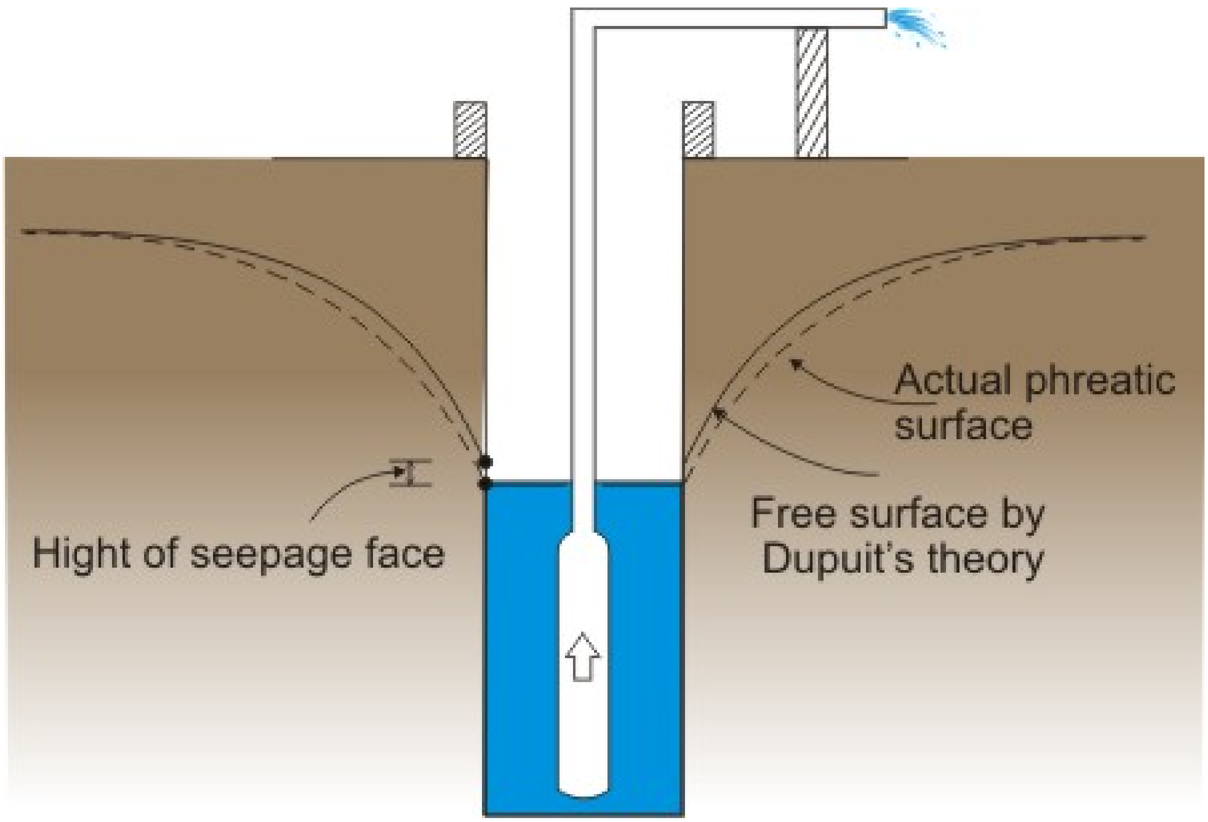


FIGURE .10 ASSUMPTION FOR DUPUIT'S THEORY

The flow at radial distance ***r*** from the well is given by the following equation under the simplifying assumptions made by Dupuit.

$$Q = 2\pi \; r \; K \; h_1 \left. \frac{dh}{dr} \right|_{r = r_1} \qquad (9)$$

Where ***h*** denotes the height of the water take at a distance ***r*** above a datum, which may be the bedrock. Integrating between the limits of r_w and r_1

$$h_1^{\,2} - h_w^{\,2} = \frac{2.3Q}{\pi K} \ln \frac{r_1}{r_w} \qquad (10)$$

By knowing the values of the water table at two places located at distances r_1 and r_2 from the centre of the well with corresponding heads h_1 and h_2 the value of the coefficient of permeability K can be worked out from the equation.

The water table head at any radial distance r can also be expressed in terms of H, the head at undisturbed initial water table before pumping as:

$$H^2 - h^2 = \frac{2.3Q}{\pi K} \ln \frac{R}{r} \tag{11}$$

In the above expression, R is the radius of influence of the radial distances where the water table head is nearly equal to H.

Since (9) was derived with Dupuit's assumption (refer to Lesson 2.6), the actual free surface will be slightly higher than the predicted free surface. This is because the gradient of the cone of depression is larger towards the well where the curvature of streamlines is most marked. The free surface of water table will actually meet the periphery of the well at some height above the water level in the well as shown in Figure 10.

2.7.4 Unsteady flow to a well: Confined aquifer

When a well starts pumping out water at constant rate, the potentimetric surface gradually gets lowered. The unsteady state representation of the potential head in such a case is given by the following expression,

$$\frac{\partial^2 h}{\partial r^2} + \frac{1}{r}\frac{\partial h}{\partial r} = \frac{S}{T}\frac{\partial h}{\partial t} \tag{12}$$

Where h is the potential head at a distance from the well at a time t; S is the Storativity and T is the Transmissivity. The boundary and initial conditions are defined as follows:

- The potential head is equal to H, the undisturbed potential head at $r \geq R$ and for all times, that is for $t \geq 0$
- At the well face, that is at $r = r_w$, the flux (or water getting discharged), Q, is related to the gradient of the potentiometric surface as,

$$r \left. \frac{\partial h}{\partial r} \right|_{r=r_w} = -\frac{Q}{2\pi T} \tag{13}$$

- The initial condition that is at time $t = 0$, the following condition holds:

$$h \big|_{at\ \ any\ \ r} = H \tag{14}$$

A solution of (13), based on the boundary and initial conditions, was given by Theis as in the following equation. Interested readers may refer to text books on ground water for further details.

$$H - h = -\frac{Q}{4\pi T}\left[-0.5772 - \ln u + u - \frac{u^2}{2.2!} + \frac{u^3}{3.3!} - \frac{u^4}{4.4!} + ...\right] \qquad (15)$$

The unsteady state representation of the piezometric head, when the piezometric surface gradually gets lowered during of the well is given as:

$$u = \frac{r^2 s}{4Tt} \qquad (16)$$

In equation (15), ***H–h*** is the drawdown at any radial distance ***r***, measured from the centre of the well. The infinite series term in the equation is generally designated as ***W(u)***, and in textbooks on well hydraulics as Raghunath (1998), these are tabulated for ease of calculation. However, with the help of calculators, it is easy to evaluate the first three terms and for practical calculations, only the first three or four terms may be considered. If only the first two terms are taken, then the expression simplifies to:

$$H\text{-}\,h = \frac{Q}{4\pi T}\left[-0.5772 - \ln u\right] \qquad (17)$$

$$= \frac{Q}{4\pi T}\left[\ln \frac{2.25Tt}{r^2 S}\right] \qquad (18)$$

$$= -\frac{0.183Q}{T}\left[\ln \frac{2.25Tt}{r^2 S}\right] \qquad (19)$$

For values of ***u*** less then 0.05, the value of ***H-h*** evaluated by (19) is practically the same as that obtained by applying (15).

2.7.5 Unsteady flow to a well: Unconfined aquifer

As with confined aquifers, the decline in pressure in the aquifer yields water because of the elastic storage of the aquifer Storativity (***Ss***). The declining water table also yields water as it drains under gravity from the sediments. This is termed as specific yield (s_y). The flow equation has been solved for radial flow in compressible unconfined aquifers under a number of different conditions and by use of a variety of mathematical methods. It is not in the scope of the present text to discuss these methods. The interested reader may refer to textbooks on well hydraulics, for examples "Ground Water" by H.M. Raghunath, New Age International (P) Ltd, Publishers.

2.7.6 Determining properties of confined aquifer

The hydraulic properties of a confined aquifer are often required to be known and the equations discussed in Section 2.7.2 or 2.7.4 can be used if measurements of water levels at known value of the discharge rate.

If the observations correspond to equilibrium conditions, then two observation wells, data may be used along with equation (6), to yield

$$T = \frac{Q \log\left(r_2 / r_1 \right)}{2.73\left(h_2 - h_1\right)} \qquad (20)$$

Where ***T*** is the Transmissivity of the confined aquifer and h_1 and h_2 are the depth of piezometric heads at two observation wells located at radial distance r_1 and r_2 respectively from the pumping well centre.

For non-equilibrium conditions, equation (19) would yield:

$$T = \frac{0.183Q}{\left(S_2 - S_1\right)} \log\left(\frac{t_2}{t_1} \right) \qquad (21)$$

Where S_1 and S_2 are the draw downs in an observation well at a radial distance r from the pumping well – centre at times t_1 and t_2.

Having known ***T***, the Storativity ***S*** of the aquifer may be determined using (13).

2.7.7 Design of wells for water supply

A well is an intake structure dug on the ground to draw water from the reservoirs of water stored within. The water from the well could be used to meet domestic, agricultural, industrial, or other uses. The structure may be an open dug well, or as is common these days, may be tube-wells. The well may be shallow, tapping an unconfined reservoir or could be deep, penetrating further inside the ground to tap a confined aquifer located within aquicludes. In this lesson, we shall discuss the design of tube wells, a typical installation of which is given in Figure 11.

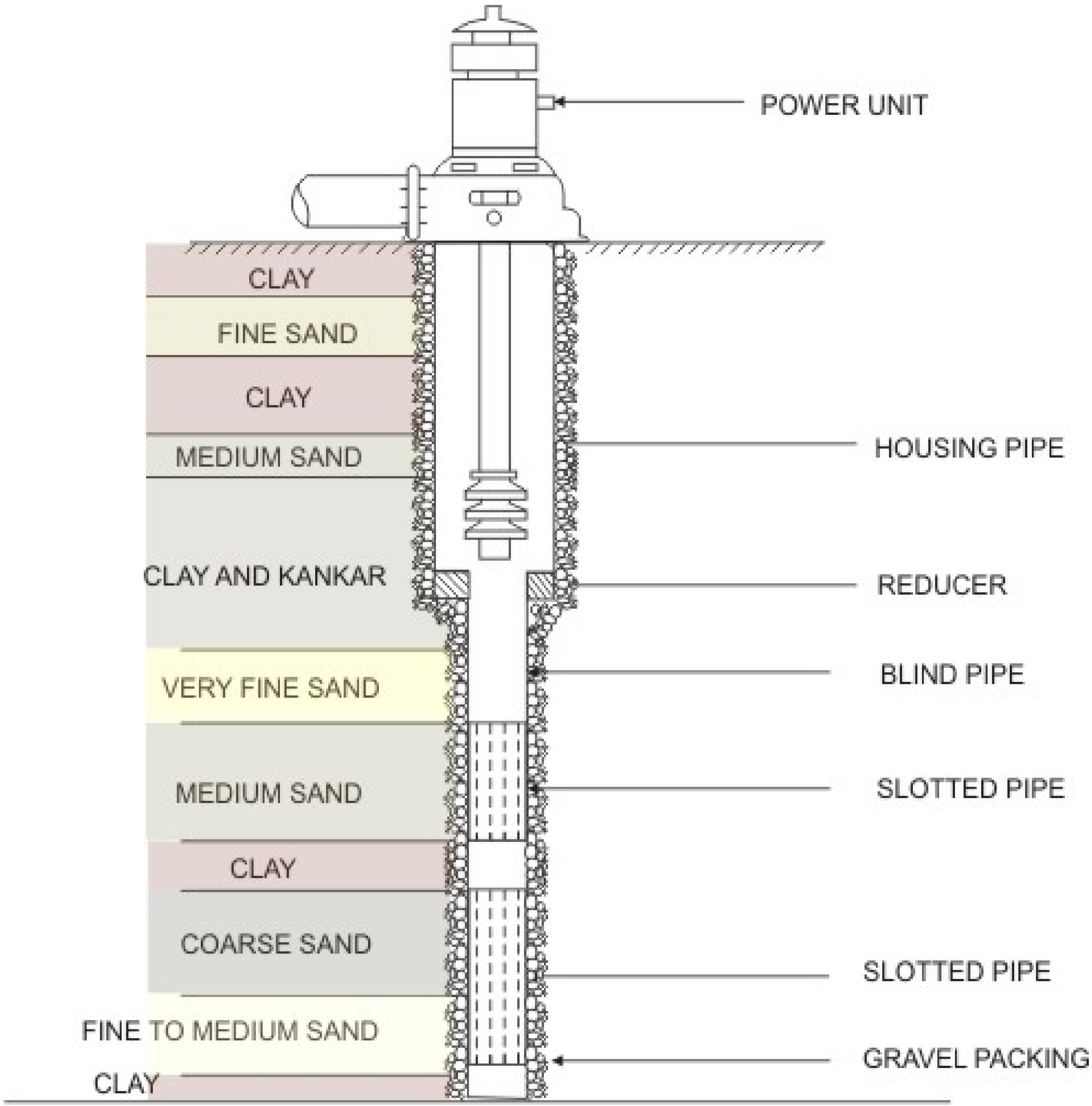


FIGURE . 11 TYPICAL INSTALLATION OF TUBE WELL

Design of a well involves selecting appropriate dimensions of various components and choosing proper materials to be used for its construction. A good design of tube well should aim at efficient utilisation of the aquifer, which it is supposed to tap, have a long and useful life, should have a low initial cost, and low mantenace and operation cost.
The parameters that need to be designed for a well include the following:

- Well diameter
  The diameter of the well must be chosen to give the desired percentage of open area in the screen (15 to 18 percent) so that the entrance velocities near the screen do no exceed 3 to 6 cm/s so as to reduce the well losses and hence, the draw down. The velocity should be reasonably low as indicated, such that the five particles within the sand should not migrate towards the well strainer slots.

- Well depth

- Selection of strata to be tapped
  The samples during drilling are collected from various depths and a bore log is prepared. This log describes the soil material type, size distribution, uniformity coefficient etc. for the material available at different depths.

- Well screen design
  This includes fixing the following parameters for a well:
  - Well screen length
  - Well-screen slot size
  - Well-screen diameter
  - Well-screen material

In case of unconfined aquifers, where too thick and homogeneous aquifer is met, it is desirable to provide screen in the lower one third thickness. In case of confined aquifers where thick and nearly homogeneous aquifer is met, about 80 to 90 percent of the depth at the centre of the aquifer is advised to be screened. Where too thick and homogeneous aquifers are encountered it is common practice to place screen opposite the more permeable beds leaving about 0.3m depth both at the top and bottom of the aquifer, so that finer material in the transition zone does not move into the well.

The size of the well screen slots depends upon the gradation, and size of the formation material, so that there is no migration of fines near the slots. In case of naturally developed wells the slot size is taken as around 40 to 70 percent of the size of the formation material. If the slot size selected on this basis comes to less than 0.75 mm, then an artificial ground pack is used. An artificial gravel pack is required when the aquifer material is homogeneous with a uniformity coefficient less than 3 and effective grain size less than 0.25 mm.

The screen diameter is determined so that the entrance velocity near the well screen does not exceed 3 to 6 cm/sec.

The screen material should be resistant to incrustation and corrosion and should have the strength to withstand the weight of the well pipe. The selection of the screen material also depends on the quality of ground water, diameter and depth of the well and type of strata encountered.

2.7.8 Installation of tube wells

The entire process of installation of tube wells include drilling of a hole, installing the screen and housing pipes, gravel packing and development of the well to insure sand free water. Depending on the size of the tubewell, depth and formation to be drilled, available facility and technical know-how, different methods are used for the construction of tubewells. Two methods that are commonly used are explained below.

- ***Cable-tool percussion drilling***

  A rig consists of a mast, lines of hoist for operating the drilling tool and a sand pump (Figure 12).

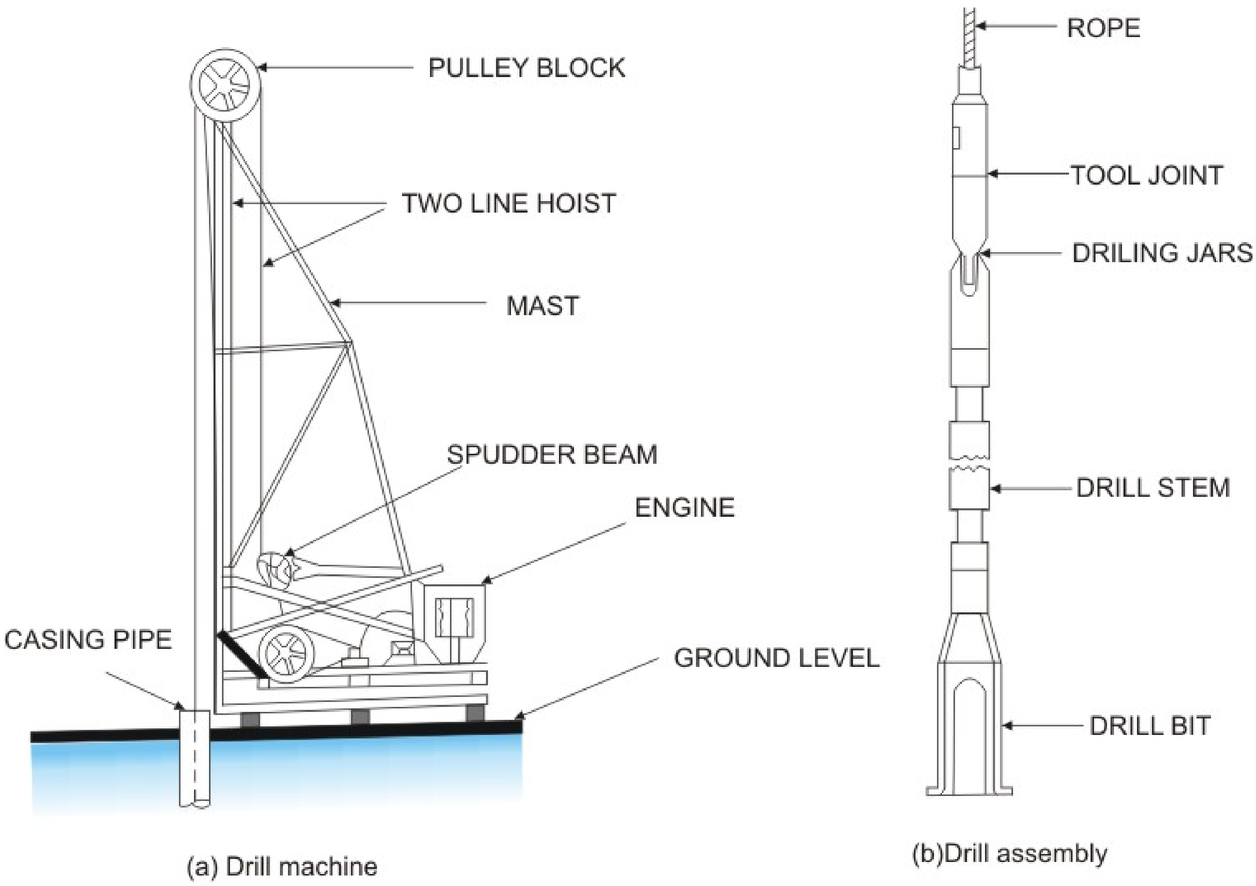


FIGURE .12 CABLE TOOL PERCUSSION DRILLING

The cutting tool is suspended from a cable and the drilling is accomplished by up and down movement (percussion) of the tool. A full string of drilling tool consists of four components:

- Drill bit
- Drill stem
- Drilling jars
- Rope socket

The drill bit is used to loosen the formation material and its reciprocating action breaks it down to smaller particles or muck. Water injected from the top converts the muck into slurry. For this purpose water is added as long as drilling continues in dry formations. The slurry flows up due to the pressure of water. The drill stem fixed just above the bit provides additional tools in order to maintain a straight line. The drilling jars consist of a pair of linked steel bars and can be moved in a vertical direction relative to each other. The rope socket connects the string of tools to the cable.

- ***Rotary Drilling method***

There are two main types of rotary drilling methods:

- Direct rotary methods, and
- Reverse rotary method

In either case, a rotating bit is used as a drilling bit. The major difference is in the direction of the flowing fluid (Figure 13).

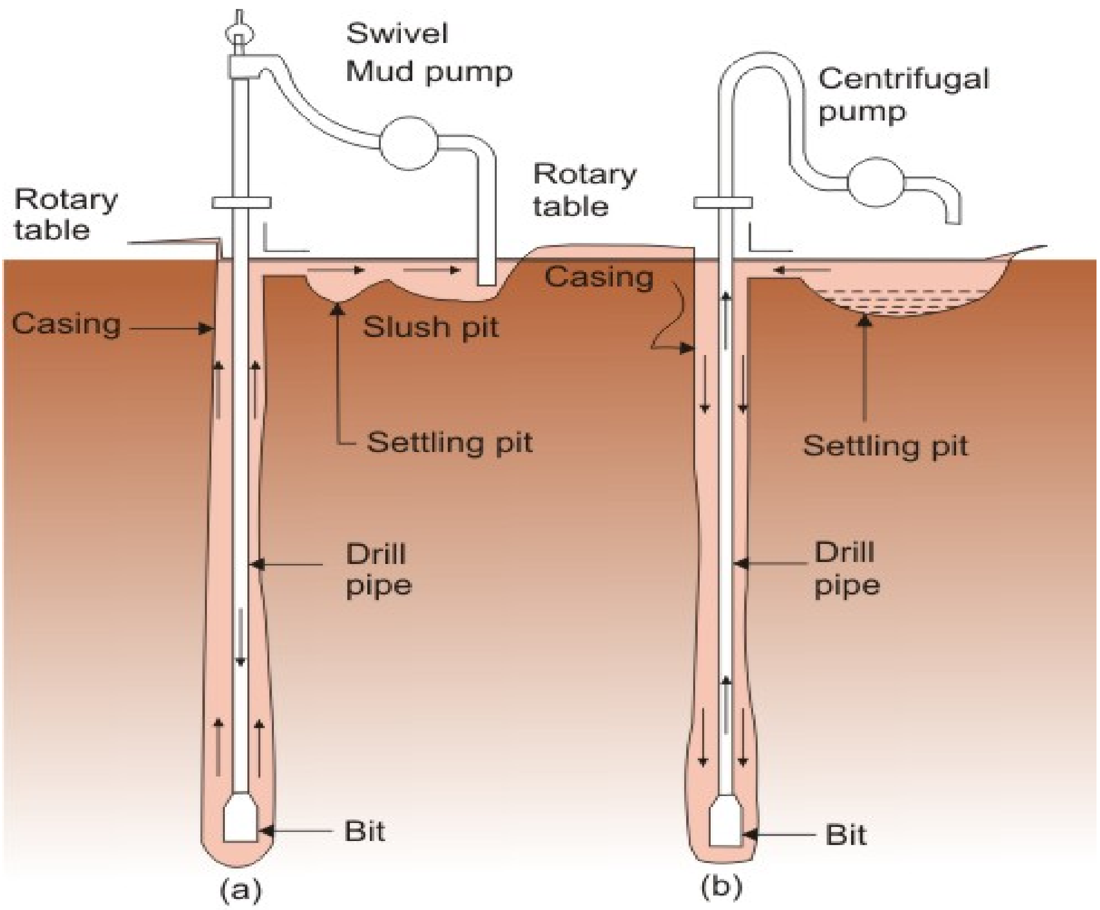


FIGURE . 13 ROTARY DRILLING (a) Hydraulic rotary (b) Reverse rotary

The rotary drilling method, also sometimes called the hydraulic rotary method of drilling, uses continuously circulating pumped fluid. The power to the drill bit is delivered to the bit by a rotating hallow steel pipe or drill pipe. The drilling fluid or bentonite slurry is pumped down through the drill pipe and out through a nozzle in the drill bit. The mud then rises to the surface through the hole and also removes the drilled formation material or muck. At the surface the fluid is led to a setting pit and then to a storage pit from where it is pumped back into the hole. Water and clay are added to the storage in to maintain quantity and consistency.

Well screens

For installation of well screens, different methods are used depending upon the design of the well, the type of well, locally available facility and the type of problems encountered in drilling operation. The Pull-back method is generally used with the cable-tool percussion method of well drilling. After the casing pipe has reached to the depth where the bottom of the screen is to be located, the sand that might have flowed into the pipe is removed. The well assembly consisting of screen and blind pipe lengths is lowered into the well. A heavy plate bail handle is provided at the bottom of the screen. The lowering of the assembly may be accomplished by suspending it by the bail handle using a flat hook attached to the sand line to engage the bail. After lowering the complete well screen assembly inside the casing pipe, the casing rip is pulled back.

For rotary drilled wells generally the Open-Hole method of screen installation used, though the Pull–Back method can also be used in this case too. In the open-hole method, after drilling the hole below the well caring, the drill stem is withdrawn and a telescope–size screen is lowered into the hole by any suitable method. The depth of the hole should be checked such that when the screen rests on the bottom of the hole, the lead packer should remain inside the lower end of the casing.

Gravel packing

Well can either manually ground packed or artificially ground packed. Natural ground packed condition is created by removing the fine sand from the formation either by pumping or by surging. An artificially gravel packed well has a envelop of specially grand sand or gravel placed around the well screen. Ground pack is designed on the basis of sieve analysis of the aquifer materials obtained during drilling. Aquifer consisting of coarse materials of less uniform sizes may not require any gravel pack.

Well Development

This process is used to remove sand, silt and other fine materials from a zone immediately surrounding the well screen. This is done by flow reversal through the screen openings so as to rearrange the formation particles in a naturally developed well and form a graded filter with materials of increasing porosity and permeability towards the well in an artificially gravel packed well, so that ultimately the well will yield clear sand free water.

Module 2

The Science of Surface and Ground Water

Lesson 8

Flow Dynamics in Open Channels and Rivers

Instructional Objectives

On completion of this lesson, the student shall be able to learn:

1. The physical dynamics of water movement in open channels and rivers
2. The mathematical description of flow processes in the above cases
3. Different types of free surface flows: uniform, non uniform, etc.
4. Different channel shapes and cross sections and their representations
5. Computation steps for gradually varied water surface profiles

2.8.0 Introduction

It is common for water resources engineers to design a water system involving flow of water from one place to another, usually passing a variety of structures on the way some of them meant for controlling the flow quantity. Rivers and artificial channels, like canals, convey water with a free surface, that is, the surface of water being exposed to air as opposed to flow of water in pipes. It is easy to visualize that for any such open channel flow, as they are called; the presence or absence of a hydraulic structure controls the position of the free surface of water. Knowing the mathematical description of flowing water, it is possible to compute the water surface profile, which is important for example in designing the height of the channel walls of the water conveying system.

Another example, the case of river flow obstruction by the presence dam may be mentioned. The water level of the river increases on construction of the dam and it is essential to know the maximum possible rise, perhaps during the maximum flood, in order to know the degree of submergence of the land behind the dam. Barrages are low height structures, and hence, the rise of water will not be occurring uniformly across the river, again due to the difference of gate operation.

In this lesson, the behavior and corresponding mathematical description of flow in open channels are reviewed in order to utilize them in designing water resources systems.

2.8.1 Flow in natural rivers

Figure1 shows a river carrying a low discharge.

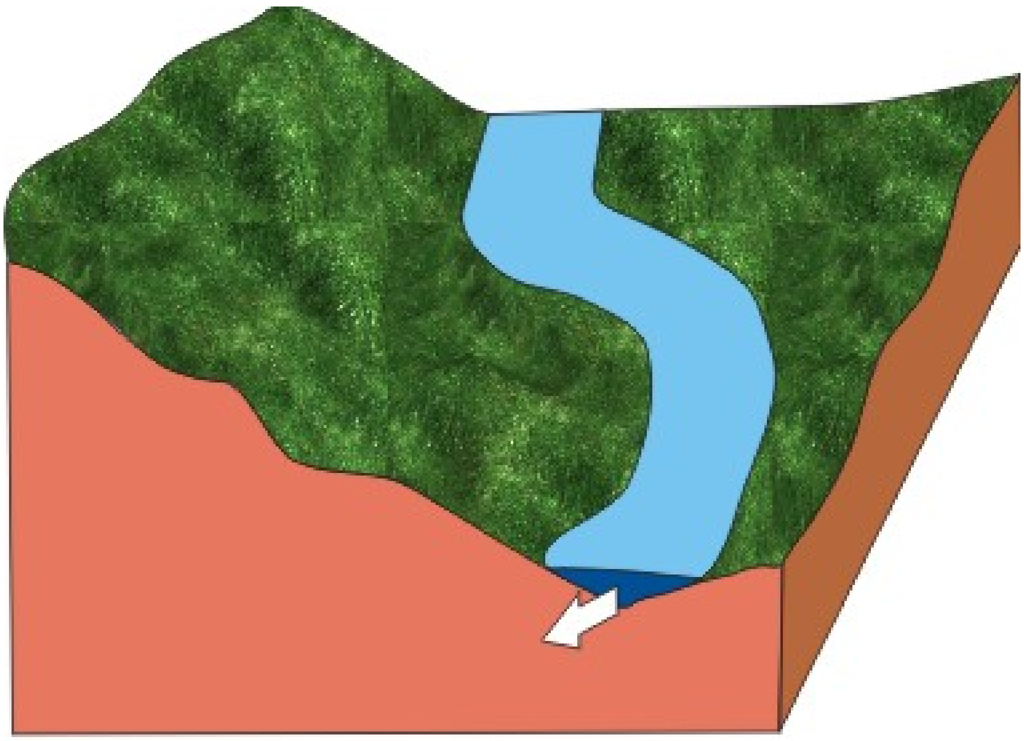


FIGURE. 1 A river flowing within its banks .

When the water surface of the river just touches its banks, the discharge flowing through the river at this stage is called the "***bank full discharge***". It is also sometimes called the "***dominant discharge***". If the discharge in the river increases, the water will overflow the banks and would spill over to the adjacent land, called the ***flood plains*** (Figure 2).

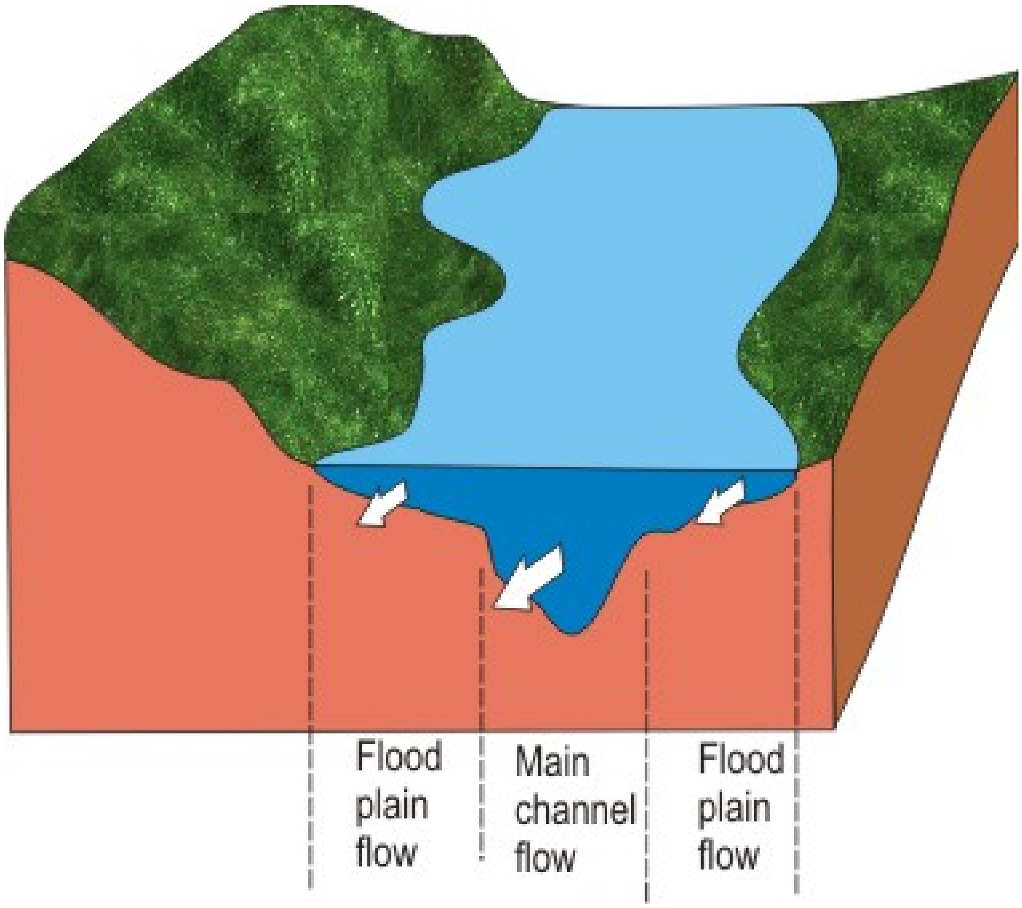


FIGURE .2 A river flowing its banks during flood

Though the amount of discharge flowing through the river is of interest to the water resources engineer it cannot be measured directly by any instruments. Rather, an indirect method is used which requires knowledge of the velocity distribution in a river or an open channel.

If we plot the velocity profile across a river, as shown in Figure 1, it would actually vary in three dimensions. Figure 3 shows the variation of velocity at the water surface.

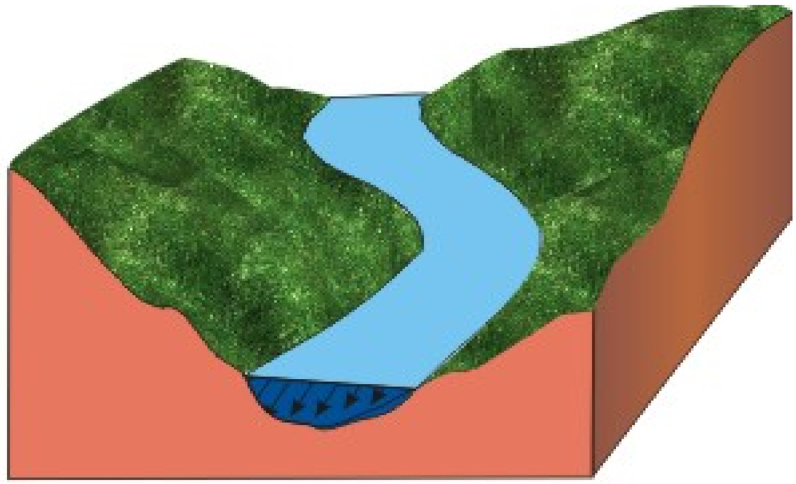


FIGURE 3 . Variation of surface velocity across a river section

It may be observed that velocity is highest at the center of the river but is zero at the banks. If a velocity profile were plotted on another horizontal plane at certain depth of the river, there too the velocity profile would be found to be similar in shape, but smaller in magnitude (Figure 4).

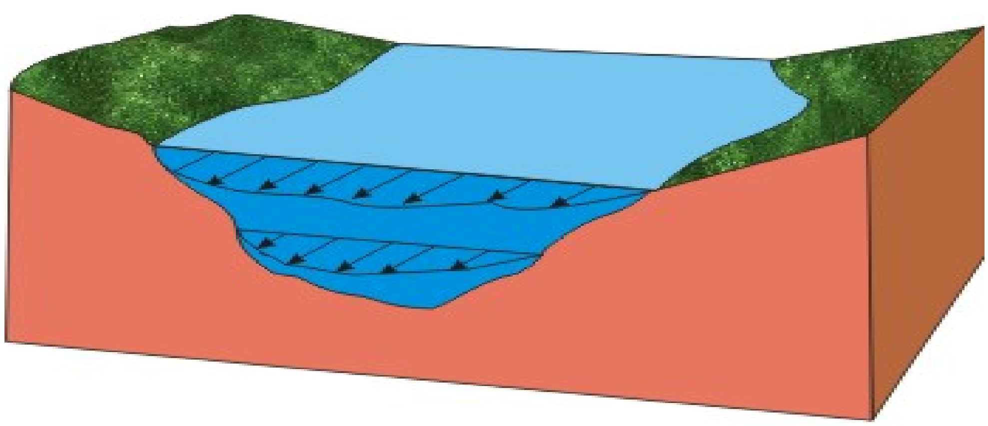


FIGURE 4 . Velocity variation of a river section at two levels.

Similarly the velocity profile of the river flowing in flood would be as shown in Figure 5, showing that the velocities over the flood plains is smaller compared to the main stream flow.

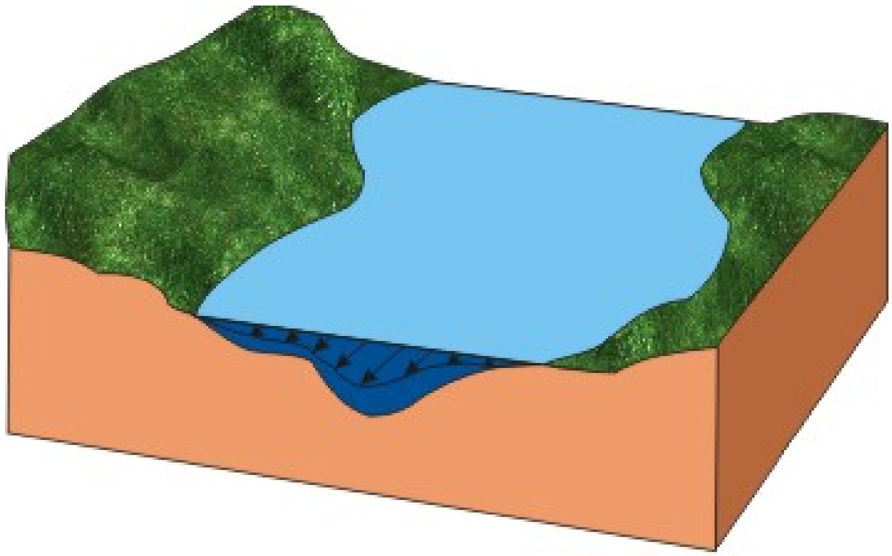


FIGURE 5. Surface velocity profile across a river section for a river flowing in flood

If we now take a look at the variation of velocity in a vertical plane within a river, and we plot them along different vertical lines across the river, then we may find the velocity profiles similar to those shown in Figure 6.

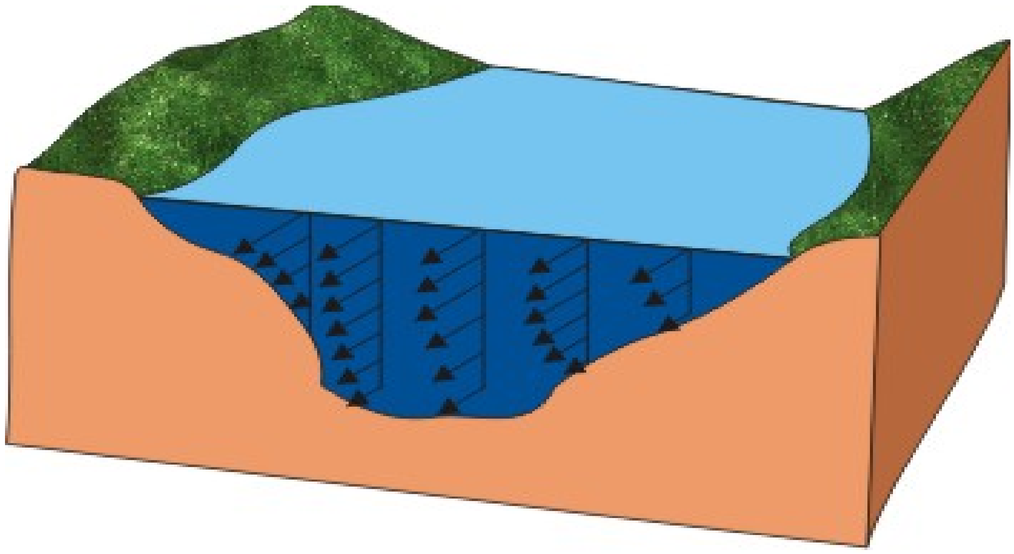


FIGURE 6. Vertical velocity profiles in a river section

In order to measure the discharge being conveyed in a river, the velocity profile or the average velocity at a number of equally spaced sections are measured, as in Figure 6. The total discharge is then approximately taken equal to the sum of the discharges passing through each segment.

Another way of depicting the velocity variation across a river cross-section is to plot "Isovels", which are actually the locus of points having equal velocity (Figure 7).

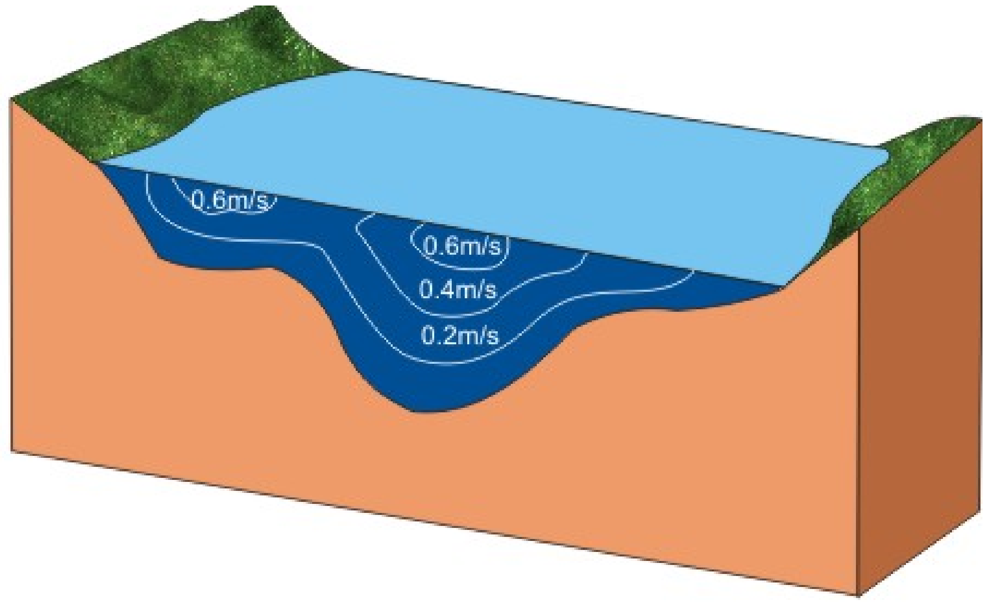


FIGURE 7. Isovels showing contours of typical equal velocities across a typical river section

It has been observed through experiments that a plot of velocity in the vertical plane would show that the maximum velocity occurs slightly below the surface (Figure 8) for a typical river flow.

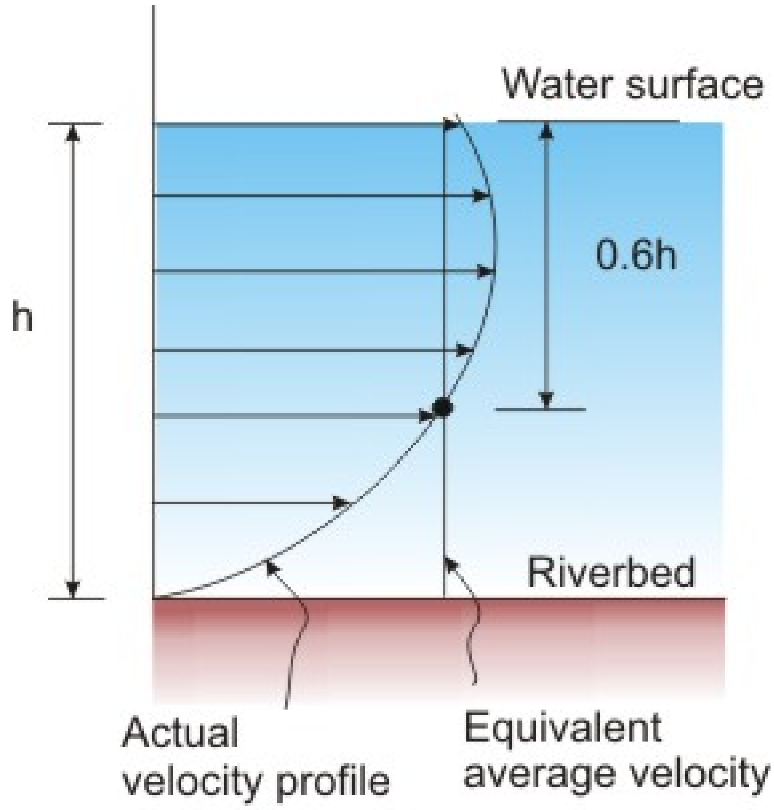


FIGURE 8. Vertical velocity distribution and
average velocity for flow in a river

It has further been observed that an equivalent average velocity is almost equal to the actual velocity measured at 0.6 depth.

2.8.2 Variation of discharge with river stage

The water level in a river is sometimes called the "***stage***" and as this varies, there is a proportional change in the total discharge conveyed. For each point of a river, the relation between stage and discharge is unique but a general form is found to be as shown in Figure 9.

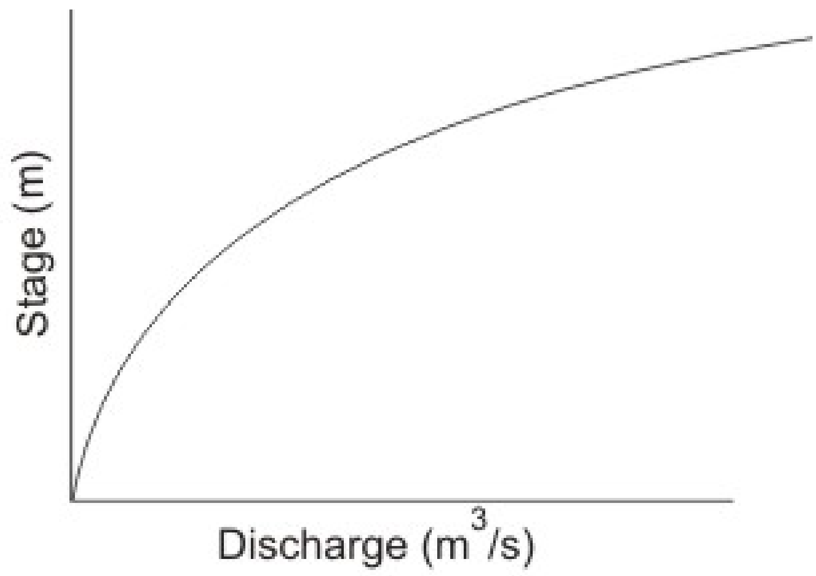


FIGURE 9. Stage-discharge curve for a river

The general mathematical description for the stage-discharge relation is given as:

$$Q = k\,(h - h_0)^m \qquad (1)$$

Where ***h*** is the gauge corresponding to a discharge ***Q*** and ***h₀*** is the corresponding to zero discharge ***k*** and ***m*** are constants. If the variables (***Q*** and ***H***) are plotted on a log-log graph, then it generally plots in a straight line as:

$$\log Q = m \log (h - h_0) + \log k \qquad (2)$$

2.8.3 Flow variation along river length

It may be interpreted from Figures 4 or 6 that the velocity in a river cross section actually varies from bank to bank and from riverbed to free water surface and hence, can be called a two dimensional variation in a vertical plane. However, for engineering purposes it is, sufficient, generally, to use an equivalent velocity in the direction of river motion (perpendicular to river cross section) which may be obtained by dividing the total discharge by the cross sectional area. In a natural river, therefore, these flow velocities may vary from section to section (Figure 10).

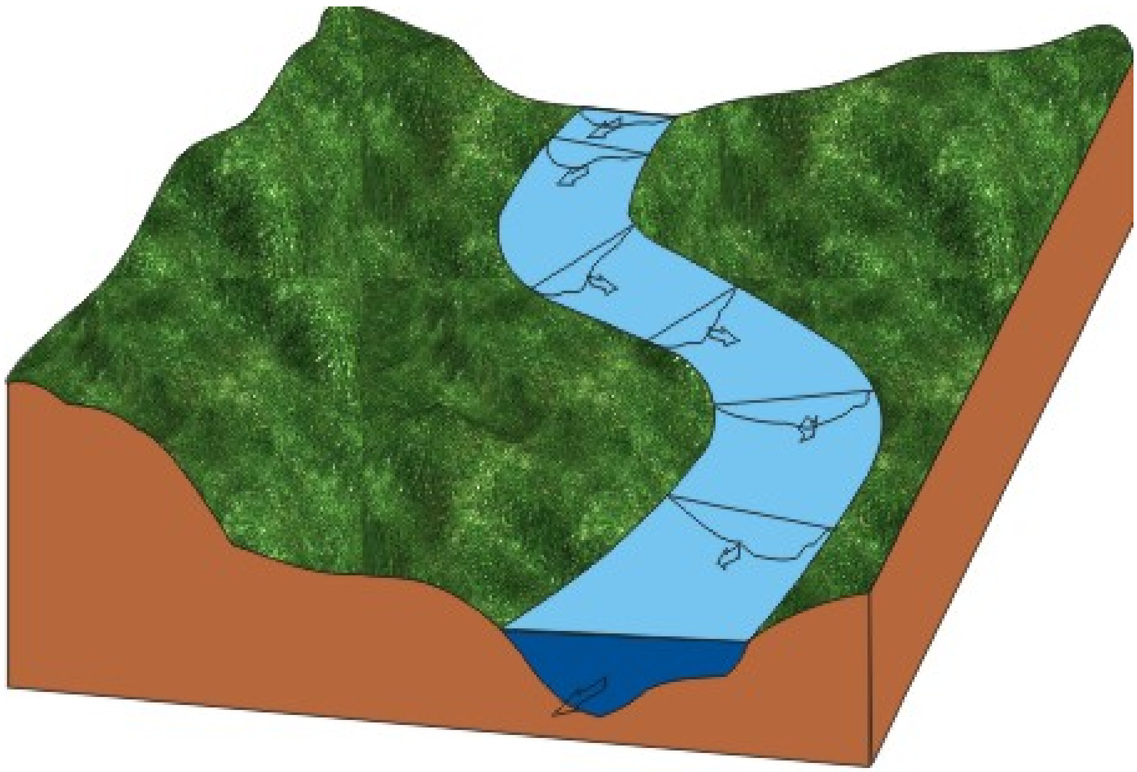


FIGURE. 10:- Variation of cross section and velocity along the length of a river showing vertical section at each point

If we now consider an axis along the length of the river, the total energy (H) is given as:

$$H = Z + h + \frac{V^2}{2g} \tag{3}$$

We may plot the total energy as shown in Figure 11, where the variables are as follows:

- ***Z***: Height of riverbed above a datum
- ***h***: Depth of water
- ***V***: Average velocity at a section

- $\frac{V^2}{2g}$: Kinetic energy head

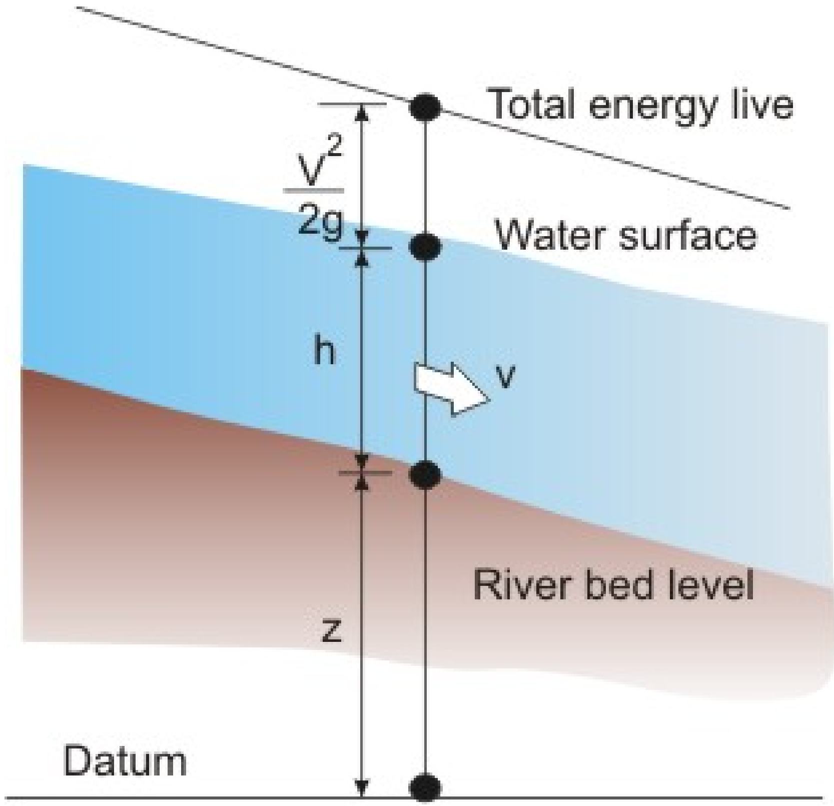


FIGURE 11. Variation of total energy along a river

Since the cross section, bed slope and flow resistance vary along a river length, the depth and velocity would vary correspondingly. However, if a short stretch of a river section is taken, then the variations in riverbed, water surface and the total energy may be considered as linear (Figure 12).

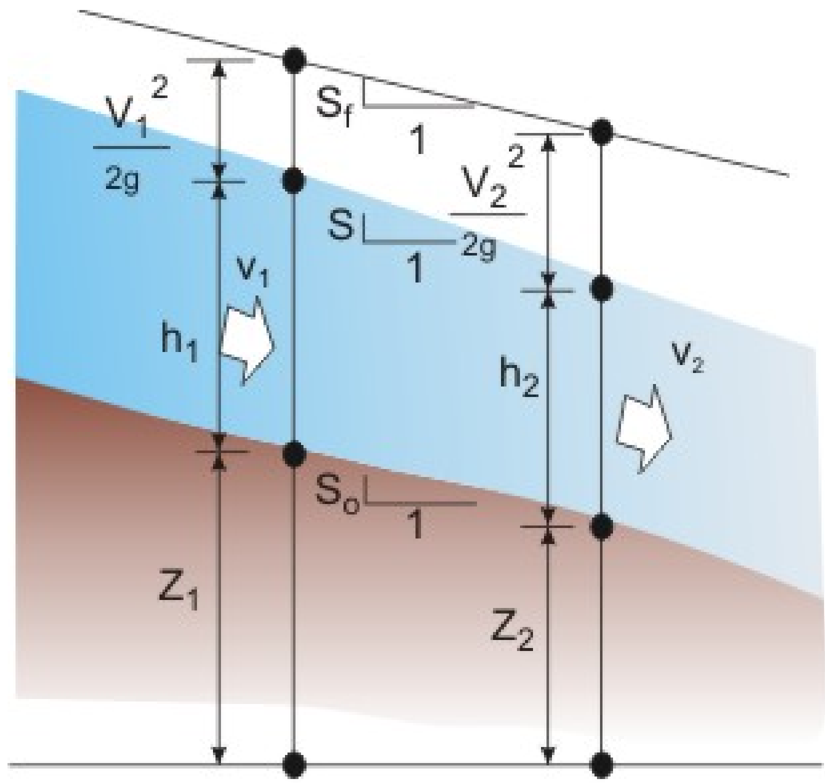


FIGURE 12. Short section between two points along a river

In Figure 12, three slopes have been marked, which are:

- ***S_0:*** Riverbed slope
- ***S:*** Water surface slope
- ***S_f:*** Energy surface slope

Since the total energy of flowing water reduces along the river length due to friction the "energy surface slope" is generally termed as the "friction slope". The energy loss in a river or an open channel occurs mostly due to the resistance at the channel sides, as the turbulent characteristics of the flowing water implies a smaller loss internally within the water body itself.

It has been nearly 200 years when scientists first attempted to mathematically express (or "model") the friction slope in terms of known variables like average velocity, cross section properties and riverbed slope. One of the earliest models for friction slope S_f or, in effect, the channel resistance was derived from the considerations of "uniform flow" (Figure 13) where the flow variables and cross section are supposed to remain constant over a short reach.

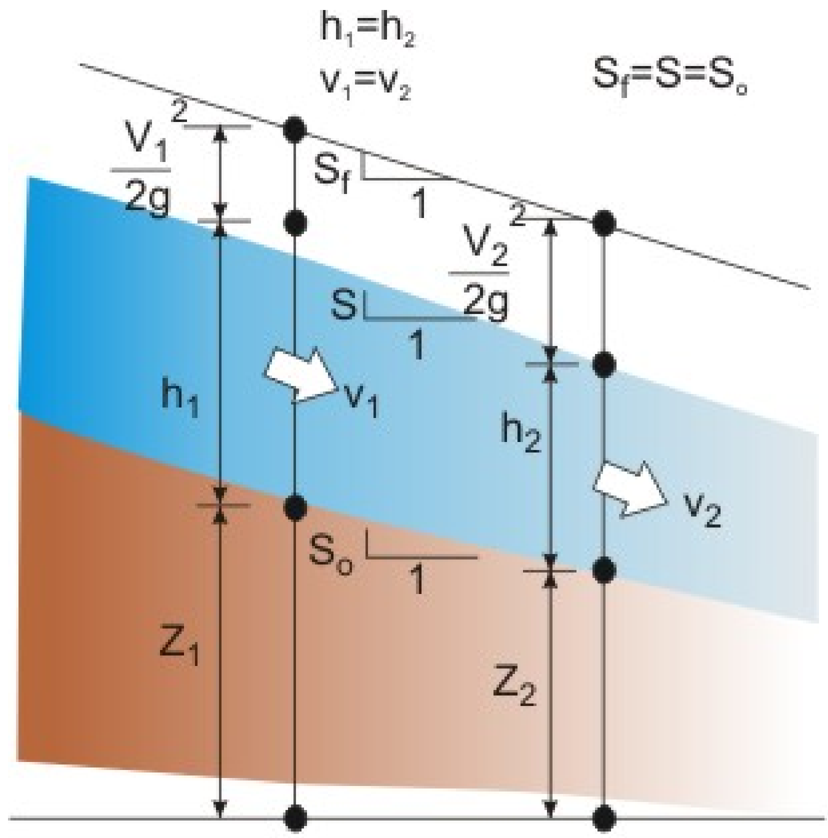


FIGURE 13. Uniform flow along a channel reach

If we take small volume of fluid from these two sections we may make a free body diagram of the forces acting on it (Figure 14).

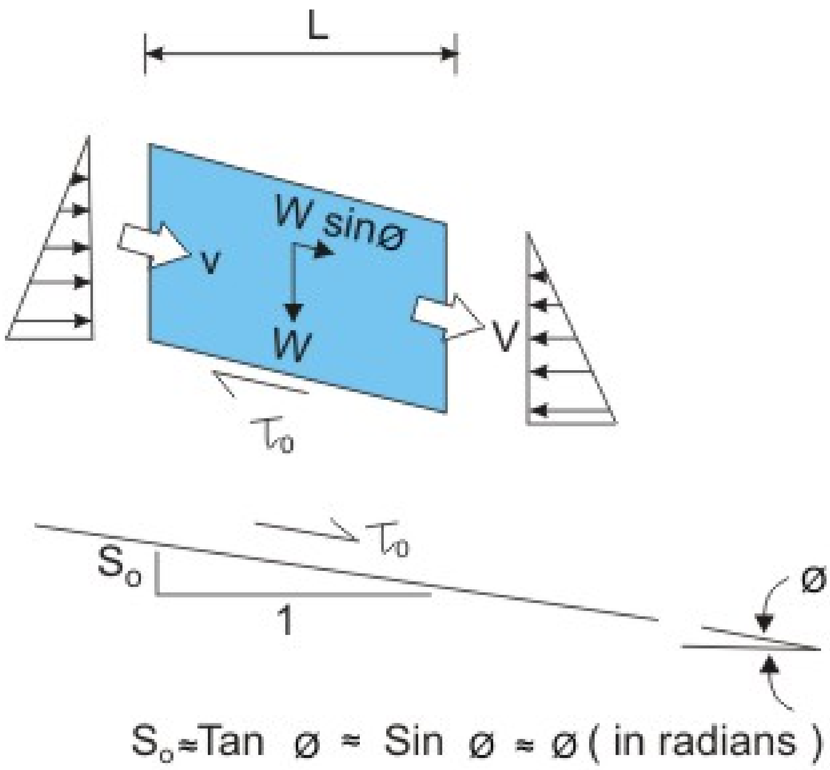


FIGURE 14. Free body diagram of forces on a control volume in uniform flow

The variables represented in the figure are as follows

- ***W:*** Weight of water contained in the control volume
- ***V:*** Inflow velocity, which is the same as the outflow velocities
- ***θ:*** Angle of slope river bed, which is also equal to that water surface and friction slopes
- τ_0**:** Shear stress due to friction acting on the control volume of fluid from the river bed and all along the periphery, though in Figure 14 only the resistance due to the riverbed is shown.

Equating the forces and noting that the inflowing and out flowing momenta are equal as well as the pressure forces at either end of the control volume one obtains:

$$\tau_0 P\ L = W \sin\theta = \rho\ g\ A\ L \sin\theta \qquad (4)$$

Where the remaining variables are:

- ***P:*** wetted perimeter
- ***A:*** Cross section of flow area
- ***L:*** Length of control volume

Assuming θ to be very small and nearly equal to bed slope, we have

$$\tau_0 = \rho g R S_0 \qquad (5)$$

Assuming a state of rough turbulent flow, as is the case for natural rivers and channels, one may write

$$\tau_0 \propto V^2 \qquad \text{or} \qquad \tau_0 = kV^2 \qquad (6)$$

Substituting into (4),

$$V = \sqrt{\frac{\rho g}{k} R S_0} \qquad (7)$$

This may be written as

$$V = C\sqrt{RS} \qquad (8)$$

This is known as Chezy equation after the French hydraulic engineer. Antoine Chezy who first proposed the formula around 1768 while designing a canal for Paris water supply. The constant ***C*** in equation (8) actually varies depending on Reynolds number and boundary roughness.

In 1869, Swiss engineers, Ganguillet and Kutter proposed an elaborate formula for Chezy's ***C*** which they derived from actual discharge data from the river Mississippi and a wide range of natural and artificial channels in Europe. The formula, in metric units, is given as

$$C = 0.552\left(\frac{41.6 + \frac{1.811}{n} + \frac{0.00281}{S_0}}{1 + \left[41.65 + \left(\frac{0.000281}{S_0}\right)\right]\frac{n}{\sqrt{R}}}\right) \qquad (9)$$

Where ***n*** is a coefficient known as Kutter's ***n***, and is dependent solely on the boundary roughness.

In 1889, Robert Manning's, an Irish engineer proposed another formula for the evaluation of the Chezy coefficient, which was later simplified to:

$$C = \frac{R^{1/6}}{n} \tag{10}$$

From Equation (8), the Manning equation may be written as:

$$V = \frac{1}{n} R^{2/3} S_0^{1/2} \tag{11}$$

Where the Manning ***n*** is numerically equivalent to Kutter's ***n***.

Many research workers have experimentally found the value of ***n***, and for natural rivers, the following books may be consulted:

1. Chow, V T (1959) "Open Channel Hydraulics", McGraw Hill.
2. Chaudhry, M H (1994) "Open Channel Flow", Prentice Hall of India.

2.8.4 Uniform flow in channels of simple cross section

For problems concerning the steady uniform flow in rivers and open channels, the Manning's equation is commonly used in India. The depth of water corresponding to a discharge in a channel or river under uniform flow conditions is called "normal depth". By combining the continuity equation with that of Mannings, one obtains

$$Q = \frac{1}{n} A R^{2/3} S^{1/2} \tag{12}$$

Where the variables have been defined in the earlier sections.

One may also write equation (12) as follows

$$Q = K\sqrt{S} \tag{13}$$

Where $K = \frac{AR^{2/3}}{n}$, also called Conveyance, is often necessary to find out the normal depth of flow corresponding to a discharge ***Q***, flowing in a channel for which equation (11) may be rearranged as

$$AR^{2/3} = \frac{nQ}{S^{1/2}} \tag{14}$$

In equation (14), the right hand side terms are known where as those in left hand are unknown and are functions of water depth. For a few commonly encountered sections the parameters A and R are given in the table below.

	Rectangle	Trapezoid	Circle
Flow Area, A	*b.h*	*(b+my).y*	$\frac{1}{8}(\phi - \sin\phi)$
Wetted Perimeter, P	*b +2h*	*b +2h*. $\sqrt{1+m^2}$	$\frac{1}{2}\phi D$
Hydraulic Radius, R	$\frac{bh}{b+2h}$	$\frac{(b+my).y}{b+2h\sqrt{1+m^2}}$	$\frac{1}{4}(1-\frac{\sin\phi}{\phi})D$
Free surface width, B	*b*	*b +2mh*	$(\sin\frac{\phi}{2})D$

In the table, ***m*** stands for the side slope of a trapezoidal channel and □ stands for the angle subtended at the centre by the water surface chord line.

As seen from the above table except for the very simple rectangular section it is not possible directly to evaluate ***h***, corresponding to ***Q*** as the left hand side of equation 13 is nonlinear in terms of ***h***. One way of solving is by Newton's method, where equation (14) is written as

$$f(h) = AR^{2/3} - \frac{nQ}{S^{1/2}} = 0 \tag{15}$$

For using Newton's method the derivative of the function is required

$$f'(h) = \frac{d}{dh}\left(A\frac{A^{2/3}}{P^{2/3}} - \frac{nQ}{S^{1/2}}\right) = 0 \tag{16}$$

$$f'(h) = \frac{5}{3}P^{-2/3}A^{2/3}\frac{dA}{dh} - \frac{2}{3}P^{-5/3}A^{5/3}\frac{dP}{dh} \tag{17}$$

$$f'(h) = \frac{5}{3}BR^{2/3} - \frac{2}{3}R^{2/3}\frac{dP}{dh} \tag{18}$$

Where we have used $\frac{dA}{dh} = B$ .Similarly the expression $\frac{dP}{dh}$ may be evaluated for any section.

Starting with a realistic value h_i the iteration may be carried out as given below:

$$h^{i+1} = h^{i} - \frac{f(h^{i})}{f^{'}(h^{i})} \quad (19)$$

Where $\boldsymbol{h}^{i+1}$ is the value of $\boldsymbol{h}$ at next iteration, which is an improvement of initial guess $\boldsymbol{h}^{i}$. The iteration may be continued till a desired accuracy is achieved.

2.8.5 Uniform flow in channels of compound cross section

A compound section may be defined as a section in which various portions of the cross-section have different flow properties, like surface roughness or channel depth. (Figure 15)

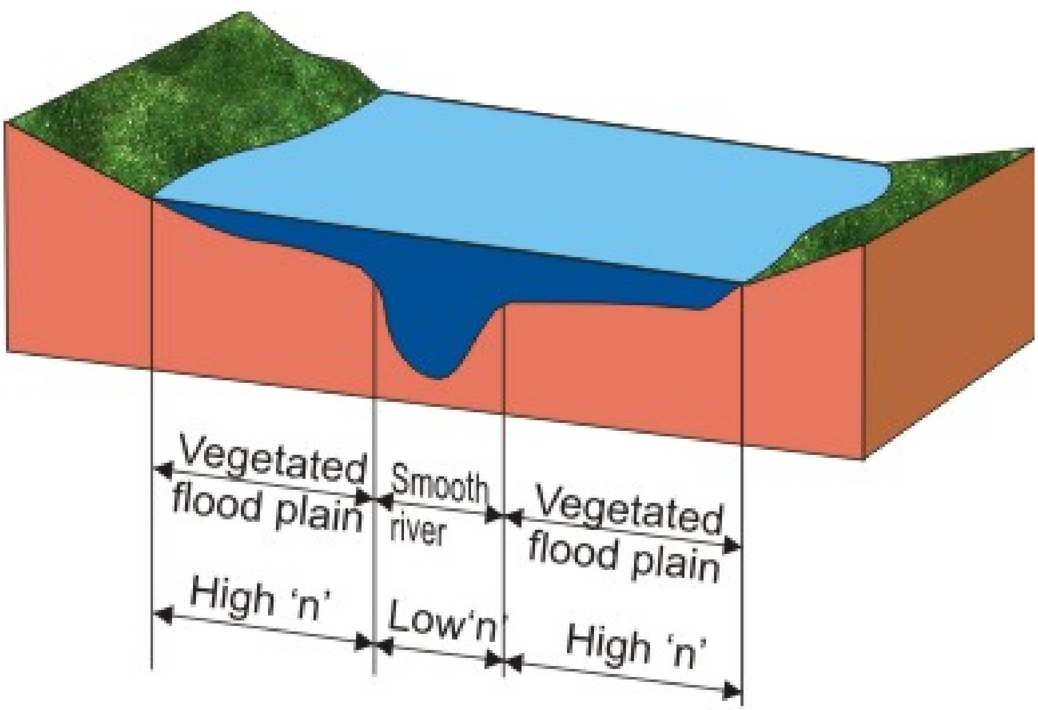


FIGURE 15. A river in flood example of a compound flow section

In order to use the uniform flow formula in compound channels one way may be to divide the flow section into sub areas (Figure 16) and treat the flow in each area separately.

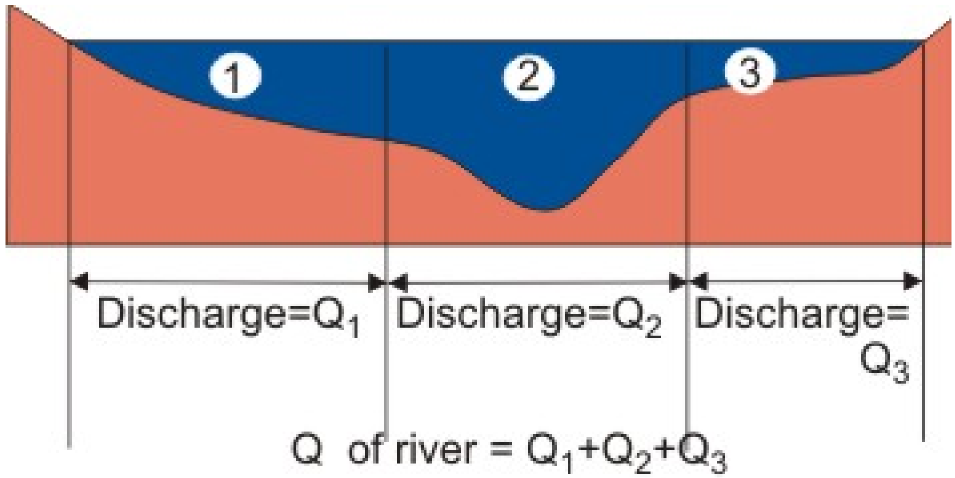


FIGURE 16. Simple flow analysis method for compound channels

However, it has been found that this method may lead to errors by as much as $\pm 20\%$ or even more (Chadwick et al 2004). The error is largely due to the neglecting of mass and momentum interchange between adjacent sub-areas. The current solution would however be more complex by using a two or even three-dimensional model.

In another method, the ***energy coefficient*** (α) and ***friction slope S_f*** are evaluated in terms of conveyance ***K*** of the sub areas. With these expressions, the flow in compound section may be computed without knowing the individual flows in each sub area. For a compound channel divided into N sections. (For example N = 3 in Figure 15). The energy coefficient, α, is found out as:

$$\alpha = \frac{\sum_{i=1}^{N} V_i^{\,3} A_i}{V_m^{\,3} \sum_{i=1}^{N} A_i} \tag{20}$$

Where V_m is the mean flow velocity in the entire section and is given as follows

$$V_m = \frac{\sum V_i \, A_i}{\sum A_i} \tag{21}$$

Where $V_i = Q_i / A_i$ and A_i is the area of its i^{th} sub-area. Equation (18) now can be written as

$$\alpha = \frac{\left(\sum Q_i^3 / A^2{}_i\right)\left(\Sigma A_i\right)^2}{\left(\Sigma Q_i\right)^3} \tag{22}$$

Now, the flow in sub-areas ***i*** may be written as

$$Q_i = K_i \, S_{f\,i}^{\,1/2} \tag{23}$$

$$S_{f\,i}^{\,1/2} = \frac{Q_i}{K_i} \text{ ------------- } (21)$$

Here, an assumption has been made that $\boldsymbol{S_f}$ has the same value for all sub-areas, which is not quite correct since the velocities of each of these areas being different, would not give equal velocity heads. Where as, the water surface is almost level over the entire cross section.

$$\frac{Q_1}{K_1} = \frac{Q_2}{K_2} = \text{- - - - - - - -} = \frac{Q_n}{K_n} = \text{Constant} = S_f^{\,1/2} \tag{24}$$

It follows from equation (23) that

$$Q_1 = K_1 \frac{Q_n}{K_n} \tag{25}$$

$$Q_2 = K_2 \frac{Q_n}{K_n}$$

-

-

-

$$Q_n = K_n \frac{Q_n}{K_n} \tag{26}$$

Adding all the above equation yields

$$\boldsymbol{Q} = \sum_{i=1}^{n} Q_i = \frac{Q_n}{K_n} \sum_{i=1}^{n} K_i \tag{27}$$

By substituting this expression for $Q_i = K_i\left(\frac{Q_n}{K_n}\right)$ into equation (27) and simplifying the equation, one obtains

$$\alpha = \frac{\left(\sum_{i=1}^{n} A_i\right)^2}{\left(\sum_{i=1}^{n} K_i\right)^3}, \sum_{i=1}^{n}\left(\frac{K_i^3}{A_i^2}\right) \qquad (28)$$

Elimination of $\frac{Q_n}{K_n}$ from equations (24) and (26) and squaring both sides give

$$S_f = \left(\frac{\sum Q_i}{\sum K_i}\right)^2 \qquad (29)$$

$$S_f = \frac{Q^2}{\sum K_i^2} \qquad (30)$$

Thus, expressions for $\boldsymbol{\alpha}$ and $\boldsymbol{S_f}$ have been evaluated for any given stage without explicitly determining the flow in each sub areas, $\boldsymbol{Q_i}$. In addition, equation (30) may be used in the procedure for determining varied flow profiles as discussed in Section 2.8.6.

2.8.6 Non uniform in channels

There are quite a few examples of non-uniform flow in rivers or open channels that may be encountered by a water resources engineer. Some of these have been illustrated in Figure 17.

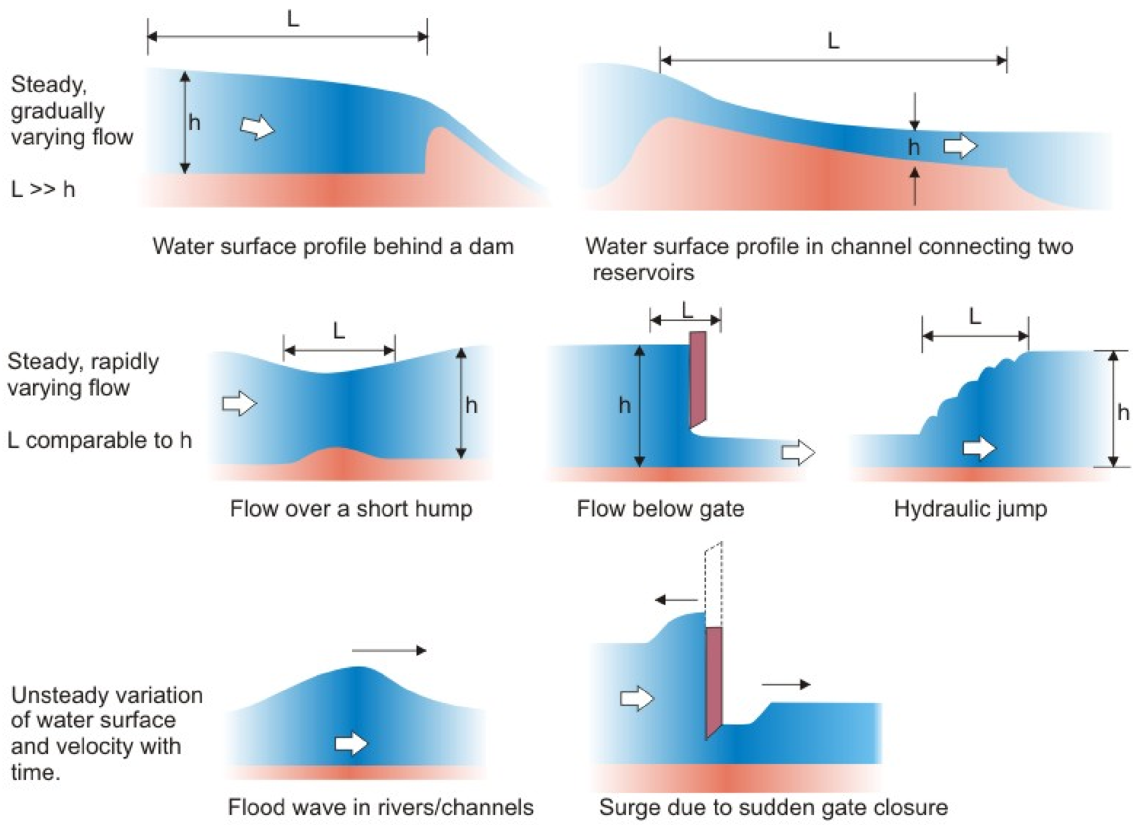


FIGURE 17. Some example of flow situation encountered in practice.

In this lesson we shall discuss the procedure to evaluate water surface profiles for steady, gradually varying flow situations. For steady, rapidly varying and unsteady flow situations, reference may be made to following or similar texts on hydraulics of open channel flow, like Ranga Raju (2003) or Subramanya (2002).

2.8.7 Non-uniform gradually varied flow calculation

A representative non-uniform gradually varied flow is shown in Figure 18.

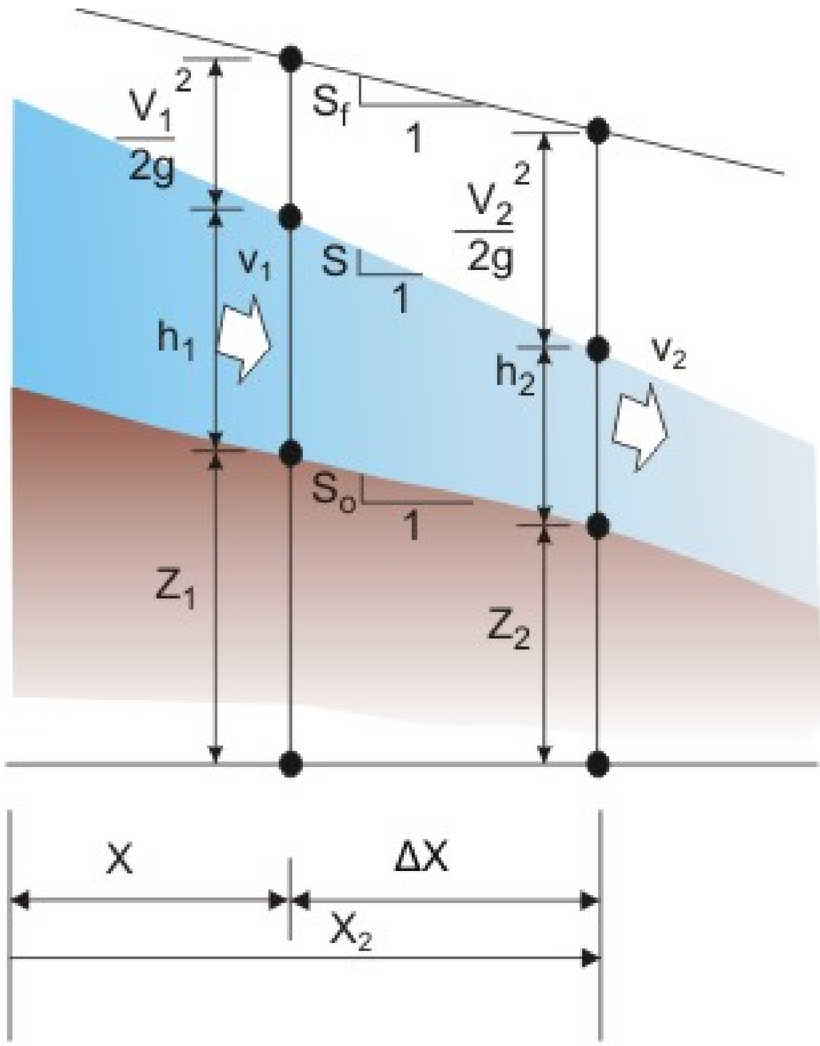


FIGURE 18. Non-uniform gradually varied flow

Over the incremental distance Δx, the depth and velocity are known to change slowly. The slope of the energy grade line is designated as α in contrast to uniform flow, the slopes of the energy grade line, water surface, and channel bottom are no longer parallel. Since the changes in the water depth ***h*** and velocity ***V*** are gradual, the energy lost over the incremental Δx can be represented by manning equation. This means that equation 11, which is valid for uniform flow can also be used to evaluate ***S*** for a gradual varied flow situation, and that the roughness coefficients discussed in Section 2.8.3 are applicable.

Additional assumption includes a regular cross section, small channel slope, hydrostatic pressure distribution and one-dimensional flow.

Applying the equivalence of energy between locations 1 and 2, and assuming the loss term as ***h_L*** given by $S_f \cdot \Delta x$ one obtains

$$Z_1 + h_1 + \alpha \frac{V_1^{\,2}}{2g} = Z_2 + h_2 + \alpha \frac{V_2^{\,2}}{2g} + S_f \, \Delta x \tag{31}$$

In the above equation, Δx is the distance between two consecutive sections ***x_1*** and ***x_2*** such that ***$\Delta x = x_2 - x_1$***.

The energy coefficient α has been used along with the $\frac{V^2}{2g}$ term, as it may be much different from 1.0 for natural sections. The term S_f in equation (31) may be evaluated by the expression for uniform flow, equation (11), where ***S_0*** may be replaced by S_f . Since equation (31) relates the energy between the sections, S_f may be taken either of the following:

Arithmetic mean: $\overline{S_f} = \frac{1}{2}\left(S_{f1} + S_{f2}\right)$

(32)

Geometric mean: $\overline{S_f} = \sqrt{S_{f1}\; S_{f2}}$ (33)

Harmonic mean: $\overline{S_f} = \frac{2 S_{f1}\; S_{f2}}{S_{f1} + S_{f2}}$ (34)

Where S_{f1} and S_{f2} are the friction slopes evaluated at section 1 and 2 by using the Mannings formula equation (12).

Equation (29) may be used by starting from one end of the channel where the flow depth and velocity are known and working backward or forward in steps. Here, two, methods are used of which we shall discuss one, called the standard step method. Avery popular computer program called HEC-2 developed by hydrologic engineering center of the US Army Corps of Engineers is based on this method. It may be freely downloaded from the website: www.hec.usace.army.mil/software/ legacysoftware/hec2/hec2-download.htm.

In the standard step method, for any given discharge the depth of flow would be known at the control section. It is then required to calculate the depth of flow at the section immediately next to the control section. Two examples are illustrated in Figure 19.

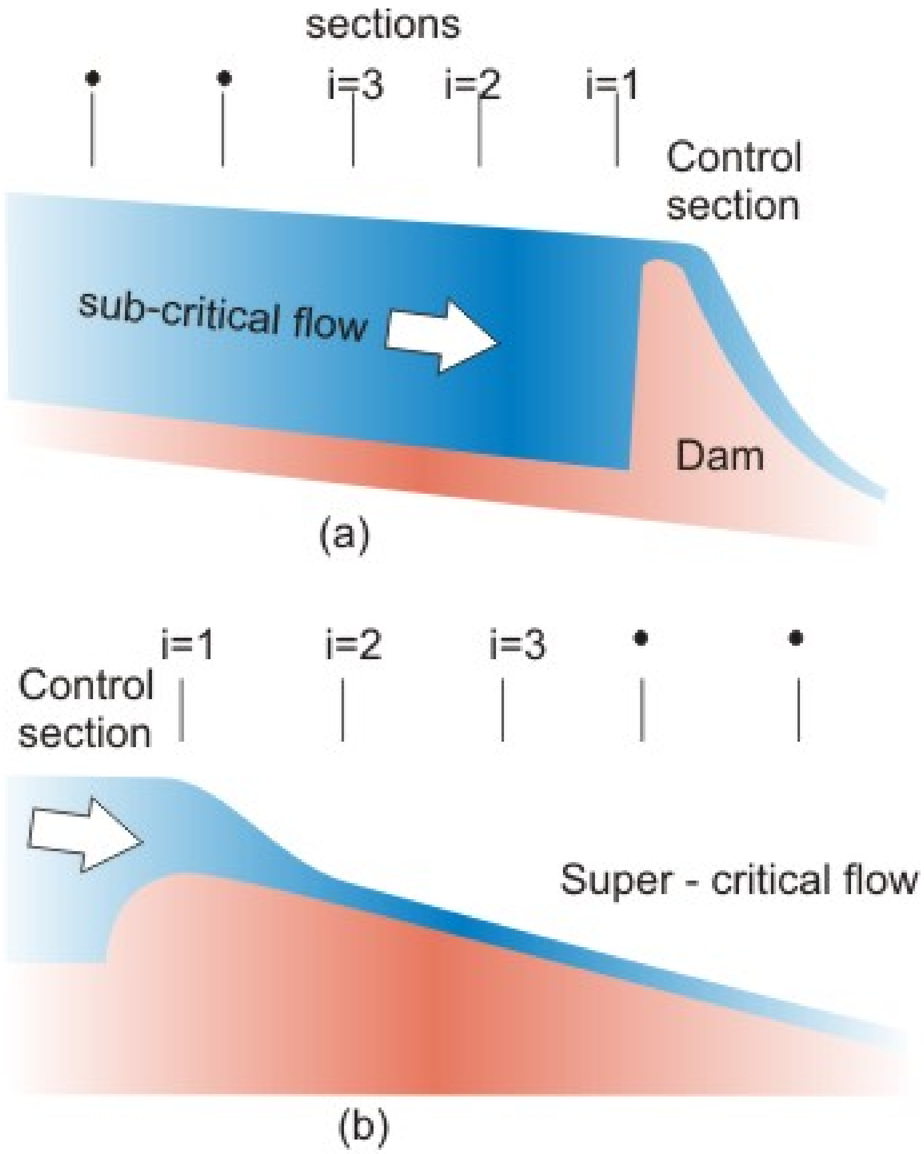


FIGURE 19. Computation grid by the standard step method.
(a) profile behind a dam: calculation proceeds upstream from control section
(b) profile in a steep channel : calculation proceeds downstream from control section.

The distance between the two successive sections (***i*** and ***i*+1**) is taken as constant, say Δx. It may be observed from the Figure 19a since the water is flowing above the dam the water depth above the dam crest can be found out for the given discharge. Hence the water level at the control section just upstream of the dam is known. Similarly, in Figure 19b, since the water is flowing down from the reservoir into the steep channel critical depth corresponding to the given discharge would exist at the control section. Here two, the water level at the control section is then known.

Starting at the control section (i =1), the total energy of water is found out to be

$$H_1 = Z_1 + h_1 + \alpha \frac{V_1^2}{2g} \tag{35}$$

Next, consider the first reach, that is, between sections i =1 and i =2. A depth of flow is assumed at section 2 and the energy there, that is,

$$H_2 = Z_2 + h_2 + \alpha \frac{V_2^2}{2g} \tag{36}$$

is evaluated. Now, one of the equations for finding $\overline{S_f}$ (the average friction slope) in the reach is found out by, say equation 31.

As may be observed from Figure 18 the numerical value of $\boldsymbol{H_2}$ found from equation (33) should be equal to that of $\boldsymbol{h_1}$ found from equation (33) + S_f. If the depth at the section 2 has been correctly assumed if the two don't match, a new depth $\boldsymbol{h_2}$ is assumed and the calculations are repeated till the two values match.

Once a correct depth is found at section 2, a similar procedure is used to find the depth at section 3, and so on.

These are the two other methods to find out water surface profiles of gradually varied flow situations, namely; method of direct integration and method of graphical integration. Interested reader may refer to standard textbooks on Hydraulics of open channel flow, like the following for details about these methods.

1. Ranga Raju (2003)
2. Subramanya (2002)

2.8.8 Gradually varied flow profiles

In many flow problems it is enough to make a qualitative sketch of water surface profile for a given flow that is taking place between two locations. It is not necessary therefore to find out the exact level of water at different points but the general shape of the free surface has to be drawn as accurately as possible. An analysis of water surface profile may be done by studying the governing equation, which can be derived from the sketch in Figure 20.

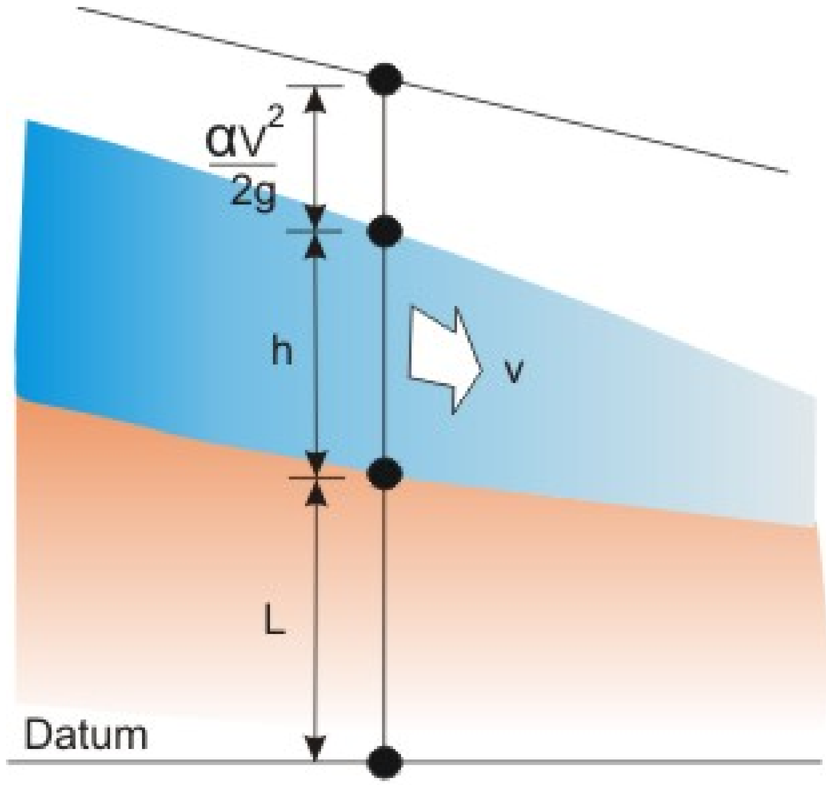


FIGURE 20. Definition sketch

The total energy ***H*** at a channel section is given as

$$\text{H} = Z + h + \alpha \frac{V^2}{2g} \tag{37}$$

Where

- ***H:*** Elevation of energy line above the datum
- ***Z:*** Elevation of channel bottom above datum
- ***h:*** Flow depth
- ***V:*** Mean flow velocity
- α: Velocity head coefficient

Considering ***x*** as the space coordinate, taken positive in the direction of flow one obtains by differentiating both sides of the equation (36) with respect to ***x*** and expressing ***V*** in terms of discharge ***Q***.

$$\frac{dH}{dx}=\frac{dZ}{dx}+\frac{dh}{dx}+\alpha\frac{Q^2}{2g}\frac{d}{dx}\left(\frac{1}{A^2}\right) \tag{38}$$

Again, we know by definition:

$$\frac{dH}{dx} = - S_f \tag{39}$$

And $$\frac{dZ}{dx} = - S_o \tag{40}$$

In which

- ***S_f:*** Slope of the energy grade line
- ***S_o:*** Slope of the channel bottom.

The negative sign of S_f and S_o indicates that both ***H*** and ***Z*** decrease as ***x*** increases. In equation (37) an expression for the derivative of A^{-2} may be found out as follows:

$$\frac{d}{dx}\left(\frac{1}{A^2}\right)=\frac{d}{dA}\left(\frac{1}{A^2}\right)\frac{dA}{dx} \tag{41}$$

$$= \frac{d}{dA}\left(\frac{1}{A^2}\right)\frac{dA}{dh}\frac{dh}{dx} \tag{42}$$

$$= -\frac{2B}{A^3}\frac{dh}{dx} \tag{43}$$

Since $\frac{dA}{dh}=B$

By substituting equations (39), (40) and (43) into equation (38), and rearranging the resulting equation one obtains

$$\frac{dh}{dx}=\frac{S_0-S_f}{1-(\alpha BQ^2)/gA^3} \tag{44}$$

If the channel is not prismatic, then the cross sectional area A changes with distance, and may be expressed as:

$$\frac{dA}{dx} = \frac{\partial A}{\partial x} + \frac{\partial A}{\partial y}\frac{dh}{dx} \tag{45}$$

The above change would modify equations (40) and (43) accordingly.

We may express equation (43), which describes the variation of ***h*** with ***x,*** in terms of the Froude Number (***Fr***) if we note the following:

$$\frac{\alpha BQ^2}{gA^3} = \frac{(Q/A)^2}{(gA)/(\alpha B)} = Fr^2 \tag{46}$$

Hence, equation (39) may be written as

$$\frac{dh}{dx} = \frac{S_0 - S_f}{1 - Fr^2} \tag{47}$$

Equation (47) can give a general idea about the nature of the curve if one knows the relative inclinations of the channel bed slope and friction slope (S_f) and the Froude Number (***Fr***). This may be done by observing the water flow depth (***h***) with respect to ***normal depth*** (h_n) and ***critical depth*** (h_c) for a given discharge, the following figures show the relative changes of h_n and h_c as channel bed slope is increased gradually from horizontal. It may be observed that the for a given discharge h_c does not change but h_n goes on decreasing starting from an infinite value for a flat slope.

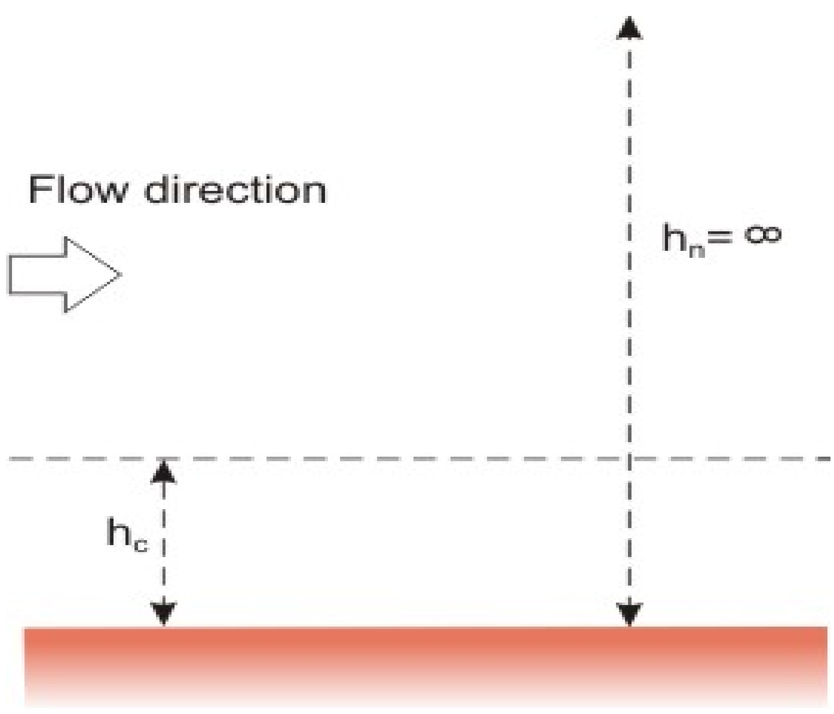


FIGURE 21. Normal depth (h_n) and critical depth (h_c) on horizontal sloped bed

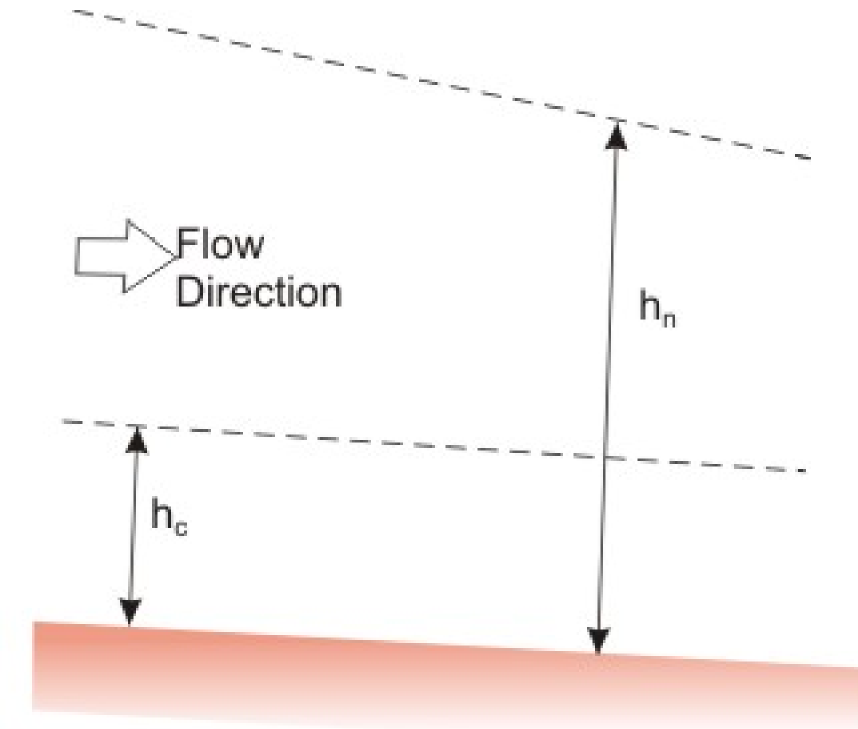


FIGURE 22. Slope increased slightly
$h_n > h_c$; mild slope

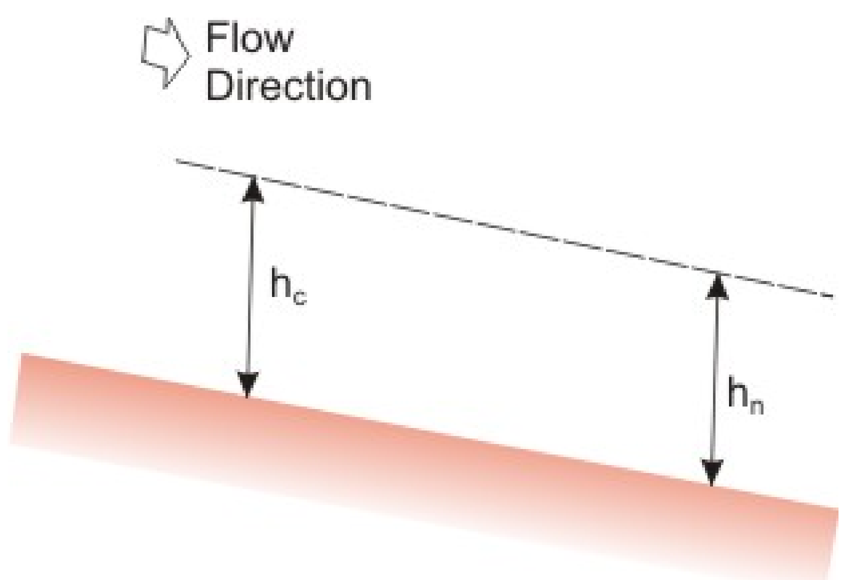


FIGURE 23. Slope increased further
$h_n \approx h_c$; critical slope

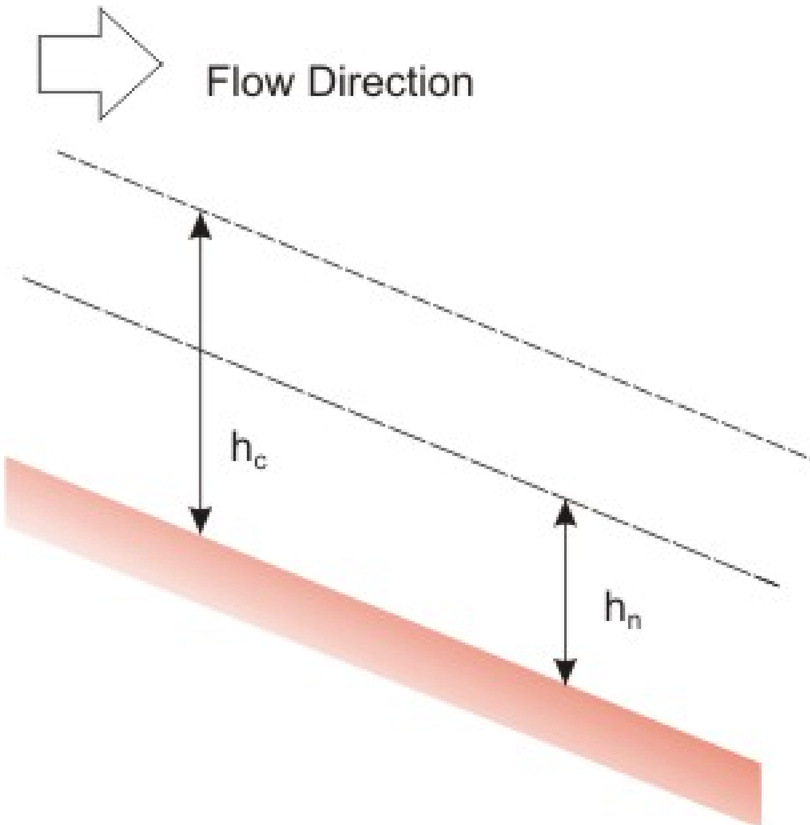


FIGURE 24. Slope increased substantially
$h_n < h_c$; Steep slope

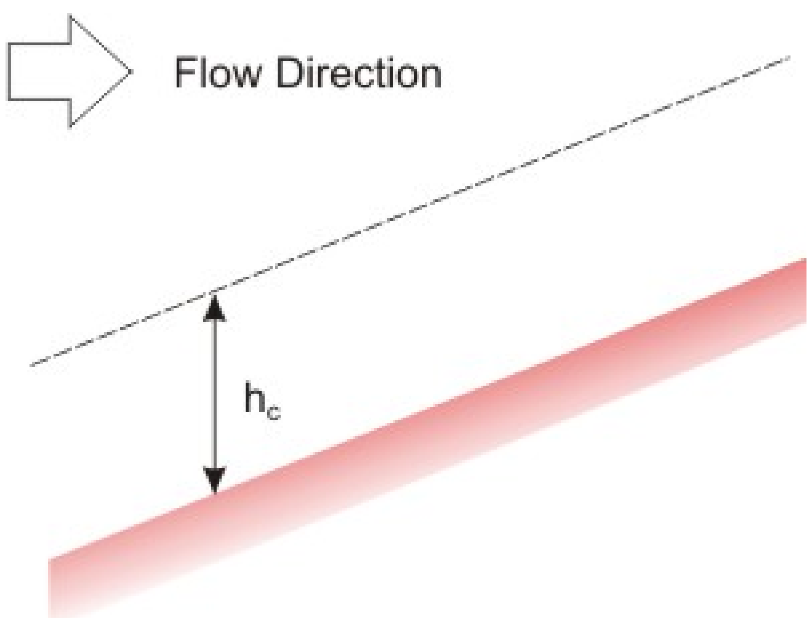


FIGURE 25 . Adverse slope .No defined h_n

In water resources projects, one generally encounters slopes of channels that are either of the following:

- Mild, where $h_n > h_c$ (Figure 22)
- Steep, where $h_n < h_c$ (Figure 24)
- Critical, where $h_n = h_c$ (Figure 23)
- Flat, where $h_n = \infty$ (Figure 21)
- Adverse, where the slope is reversed (Figure 25)

For each of these slopes, the actual water surface would vary depending upon a control that exist either at the upstream or downstream end of the channel. Some examples of controls are given below

- Weir or spillway (Figures 26 and 27)
- Gate (Figure 28)
- Free overfall (Figure 29)

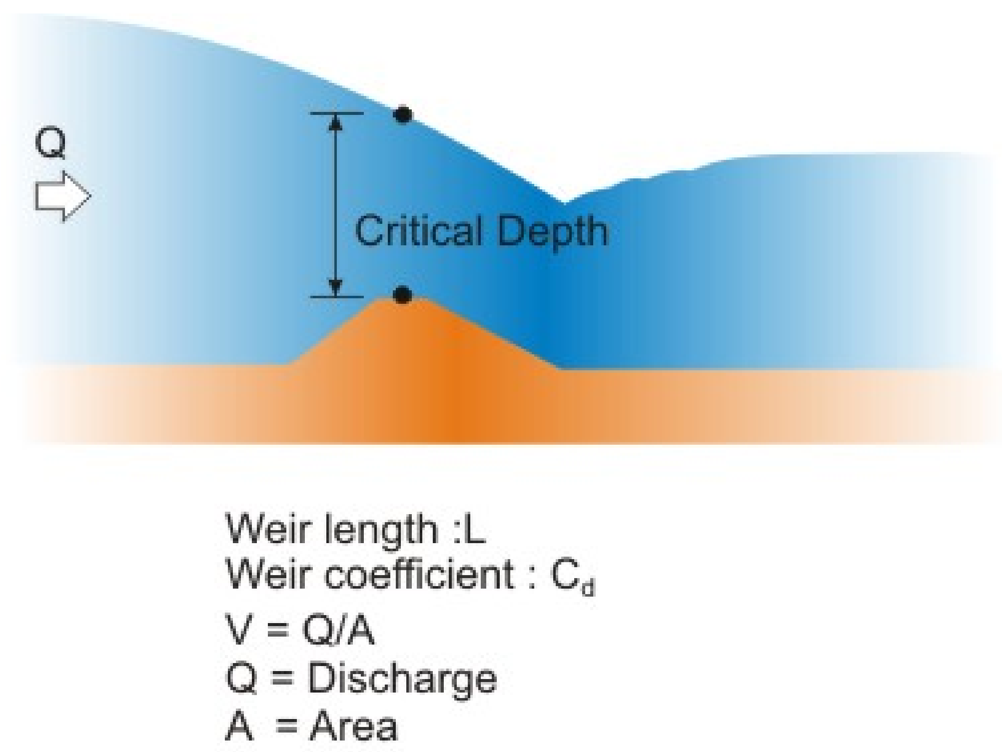
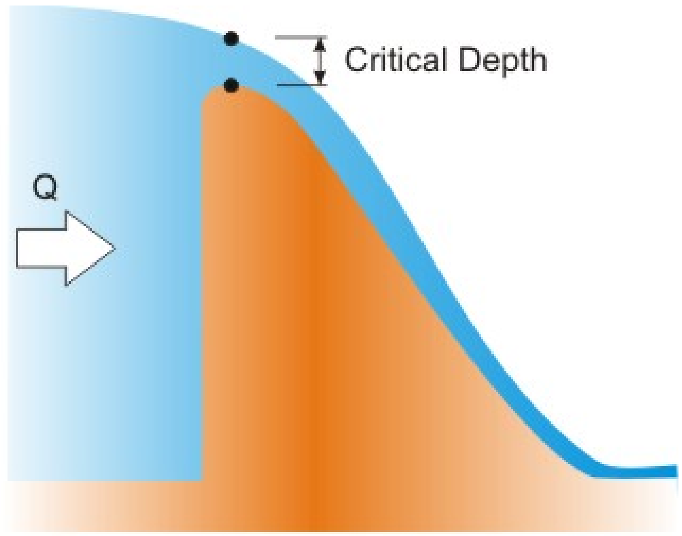


FIGURE 26. Flow over a weir



Spillway Length : L
V = Q/A
Q = Discharge
A = Area

FIGURE 27. Flow over a dam spillway

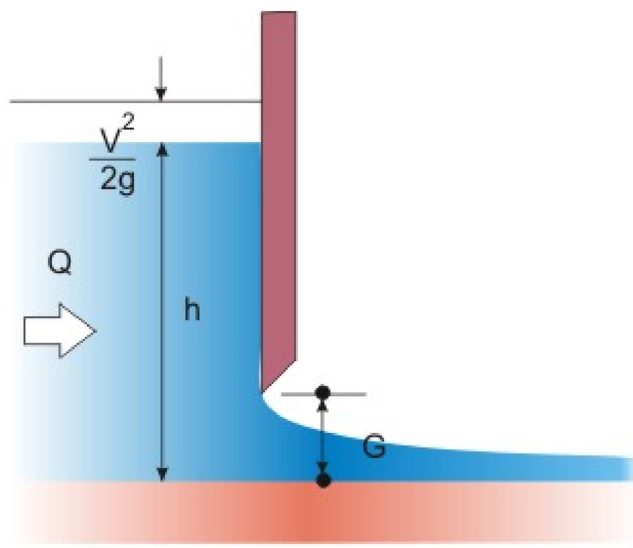


FIGURE 28. Flow below a gate .The depth of water just upstream of the gate (h) may be determined from the formula $Q=C.L.G\sqrt{H}$ where $H=h+v^2/2g$. The formula is valid for small gate openings

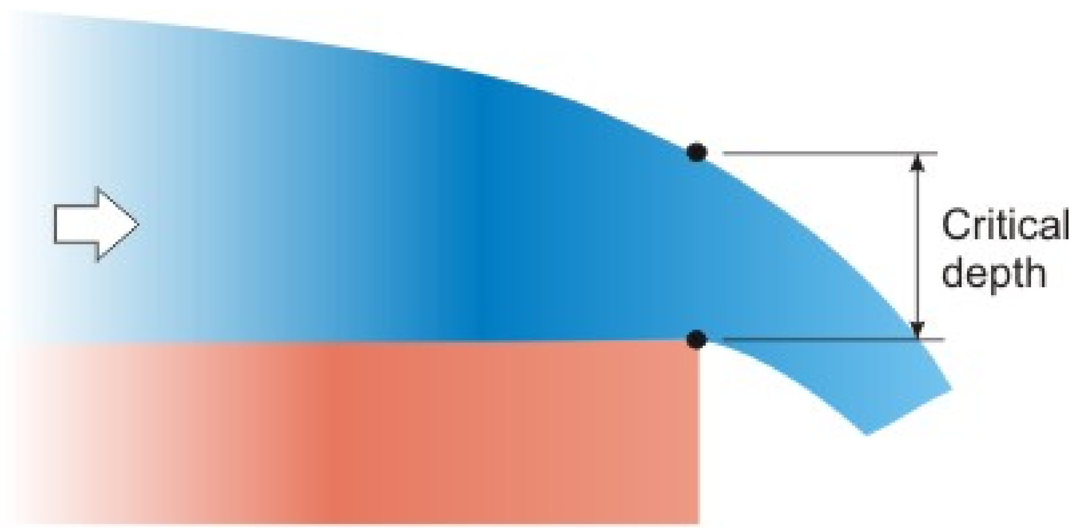


FIGURE 29. Free over fall

Apart from the above a normal depth may be assumed to exist within a very long channel, for which the conditions at the far end may be neglected (Figure 30).

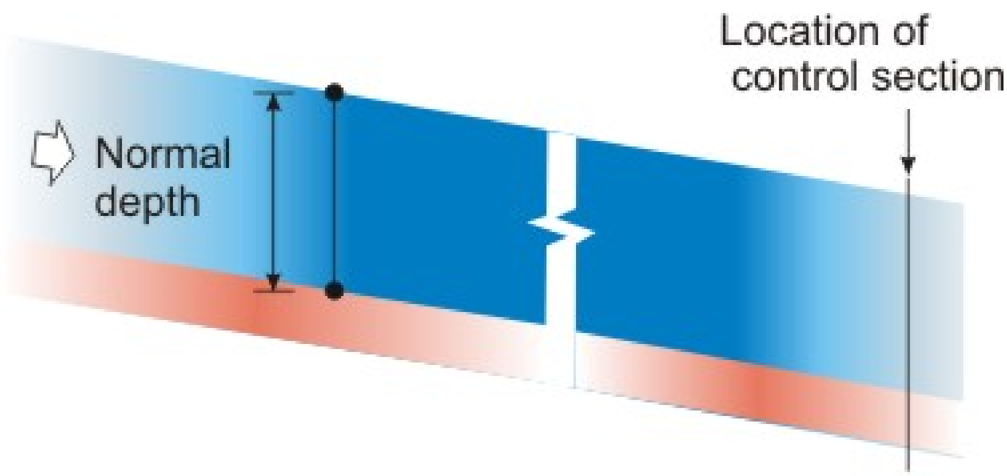


FIGURE 30. Flow in a long channel

The situation shown in Figure 30 is used often while analyzing flow in, say, at the tail end of long irrigation channels or in a long river. Examples illustrating the use of equation (42) and a known control section in determining flow profiles where for a mildly slope channel. Similar profiles may be qualitatively sketched for other channels too.

2.8.9 Downstream control raising the water level above normal depth

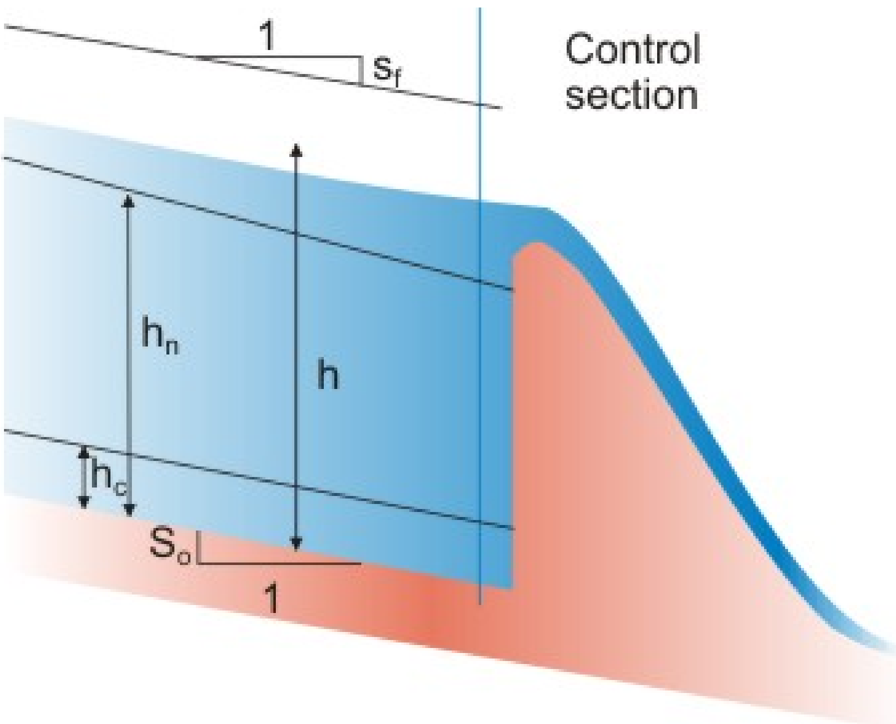


FIGURE 31. M1 type of water surface profile behind dam spillway

This situation is common for spillways of large dams. The flow profile in a mildly sloped channels where $h > h_n > h_c$ as shown in Figure 31 is known as the M_1 curve. Now, for uniform flow, $S_f = S = S_o$ when $h = h_n$. Hence it is clear from Mannings formula (equation 11), that for a given discharge, Q,

$$S_f < S_o \text{ if } h > h_n$$

Thus, in equation (47) i.e.,

$$\frac{dh}{dx} = \frac{S_0 - S_f}{1 - Fr^2} \qquad (47)$$

The numerator is positive ***Fr<1*** since ***h>h_c***. Therefore, the denominator of equation (47) is positive as well. Hence, it follows from this eqn that

$$\frac{dh}{dx} = \frac{S_0 - S_f}{1 - Fr^2} = \frac{+}{+} = +$$

This means that ***h*** increases with distance ***x***.

Comparing with Figure 29 it may be inferred that quite some distance upstream of the spillway the flow depth nearly equals normal depth. And, since ***dh/dx*** for this profile is positive which means that the water depth goes on increasing towards the spillway, the flow depth becomes nearly horizontal. However very close to spillway the flow profile again changes which is due to the fact that the flow here is not really one-dimensional (Figure 32).

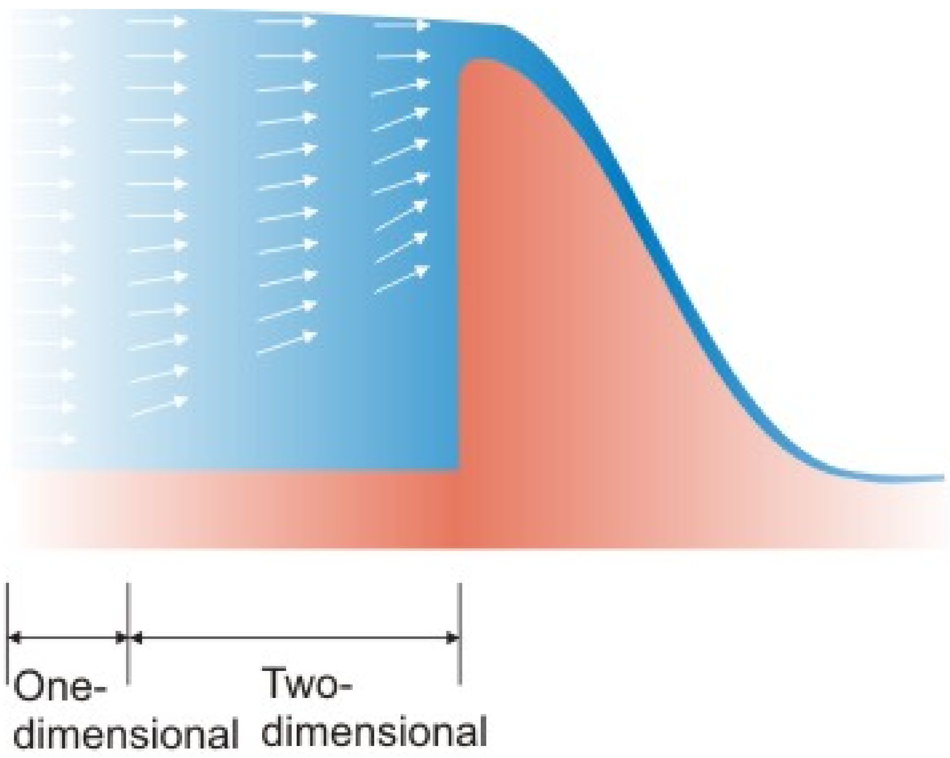


FIGURE 32. Flow situation very close to a spillway

2.8.10 Downstream controlled raising water level above critical depth but below normal depth

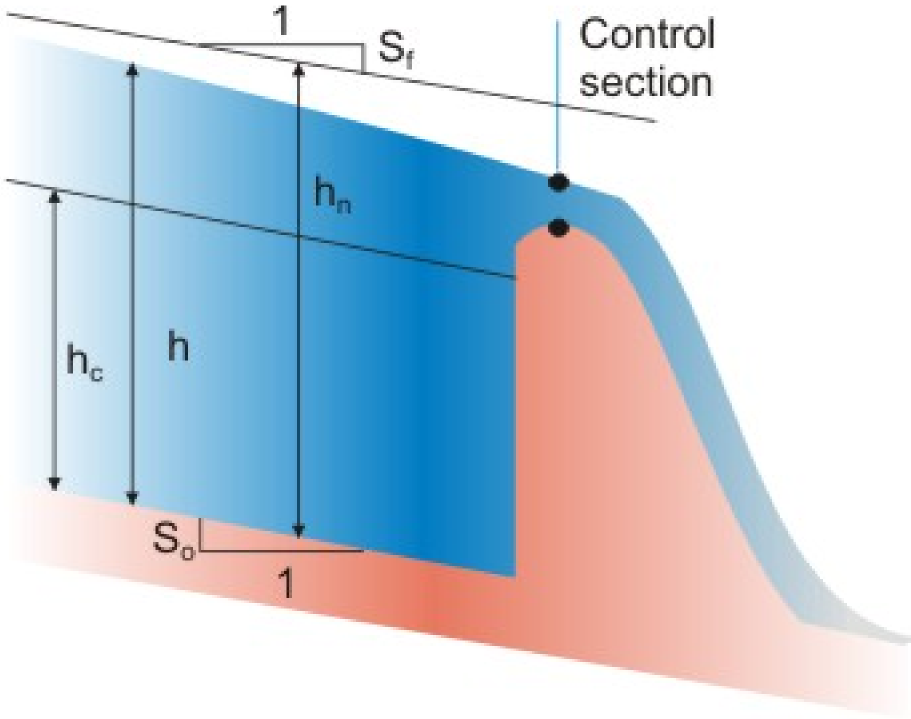


FIGURE 33. M2 type of water surface profile behind a short-height spillway

The flow profile in a mildly sloping channel where $h_n > h > h_c$, has been shown in Figure 33 is known as the M_2 curve. In this case $S_f > S_o$ since $h < h_n$ (from Mannings formula). Thus the numerator in equation (46) is negative. However, the denominator is positive, since $Fr<1$ because $h>h_c$ hence it follows from equation 46 that

$$\frac{dh}{dx} = \frac{S_0 - S_f}{1 - Fr^2} = \frac{-}{-} = -$$

Thus ***h*** decreases as ***x*** increases for upstream of this spillway control section the flow depth would be asymptotic to normal depth h_n.

2.8.11 Upstream control causing water depth to be less than both normal and critical depths

This situation is shown in Figure 34 for flow taking place below a sluice gate. The reader is advised to check the trend of water surface profile using equation (47) in this case.

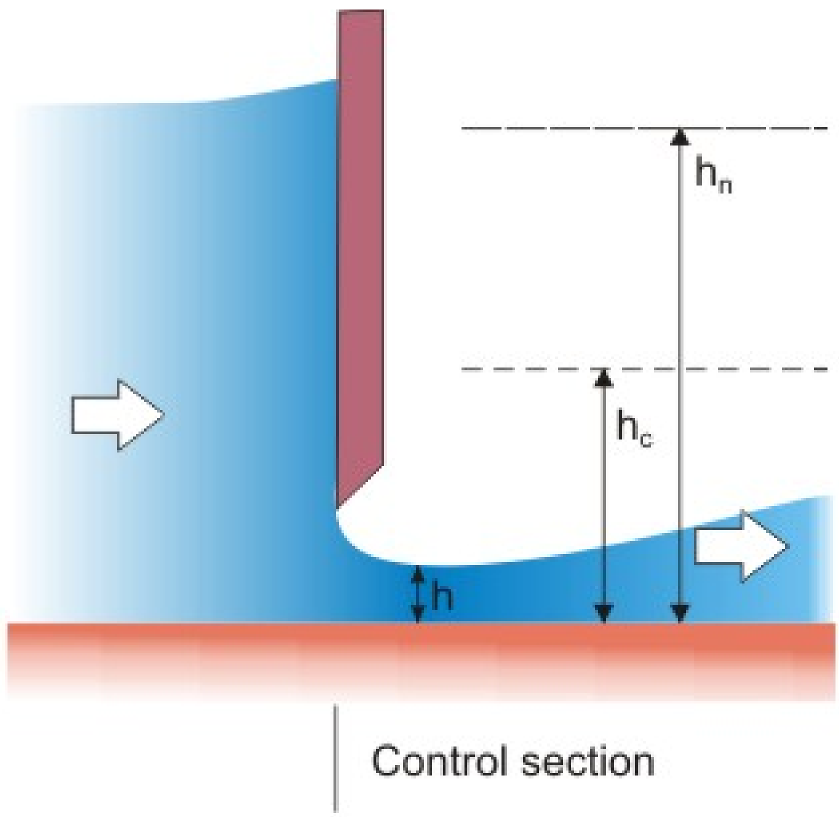


FIGURE 34. M3 type of water surface profile downstream of a gate

2.8.12 Important terms, definitions and procedures

This lesson has used certain terms, which are discussed to some detail here.

Newton's Method

This method is useful in finding a simple root of the function f(x) = 0, when the derivative of f(x) is easily obtainable. The iteration formula used in the method can be derived by the Taylor's series expansion of f(x) about $x=x_0$, the approximate value of the desired root. We have

$$f(x_0+h)=f(x_0)+h\,f^{'}(x_0)+\frac{h^2}{2!}f^{''}(x_0)+...........$$

Where h is the small correction to the root.
Now if h is relatively small, we may neglect terms containing n2 and higher powers of h. Then, we get

$$f(x_0)+h\,f^{'}(x_0) = 0$$

This gives $h=-\dfrac{f(x_0)}{f^{'}(x_0)}$

Thus, we can take the improved value of the root as

$$x_1=x_0-\frac{f(x_0)}{f^{'}(x_0)}$$

The Newton-Raphson iteration can thus be written as

$$x_{n+1}=x_n-\frac{f(x_n)}{f^{'}(x_n)},\quad n=0,1,2,.....$$

The sequence $\{x_n\}$, if it converges, gives the root.

Froude Number

This measures the ratio of inertia to gravity forces. In problems where there is an interface between two immiscible fluids the gravity forces are of importance. Froude number is defined by the relation

$$\mathrm{Fr}=\frac{V}{\sqrt{g\,D}}$$

Normal depth

For given values of channel roughness *n,* discharge *Q*, and the channel slope *S*, there is only one depth possible at which uniform flow occurs. It is known as normal depth.

Critical depth

The depth of flow at which the specific energy $\left(\mathrm{E}=y+\dfrac{V^2}{2g}\right)$ attains a minimum value is called critical depth.

2.8.13 References

- Sturm, T W (2001) Open Channel Hydraulics, McGraw Hill
- Julien, P (2002) River Mechanics, Cambridge University Press
- Chadwick, A, Morfett, J, and Borthwick, M (2004) Hydraulics in Civil and Environmental Engineering , Spon Press
- Subramanya, K (2002) Flow in Open Channels, Second edition, Tata McGraw Hill
- Ranga Raju, K G (2003) Flow through Open Channels, Second edition, Tata McGraw Hill

Module 2

The Science of Surface and Ground Water

Lesson
9
Geomorphology of Rivers

Instructional Objectives

On completion of this lesson, the student shall be able to learn the following:

1. The components of a fluvial system
2. Mechanics of sediment erosion, transportation and deposition by rivers
3. Causes of riverbed aggradation and degradation
4. Bar formation in rivers
5. Causes of meandering, lateral migration and river width adjustment of rivers
6. Relationships between velocity, perimeter, slope, velocity, etc. of alluvial rivers

2.9.0 Introduction

In the previous lesson, we discussed the mechanics flowing water. In the earth system, this water may be thought of as the water flowing downhill after a splash of rain, which carries with it some amount of soil that has been eroded by the action of flowing water. The flowing water of river moving down to the ocean also carries huge amounts of sediment which have been accumulated from other smaller streams joining the river.

In general, the water moving over the land surface is the dominant agent of land space alteration. Near surface weathering provide sediment load for the flowing streams. Some of the load gets deposited along the path of the river and only a fraction of the total material waste from the lands is carried by the rivers to the sea. In fact, the land space evolves essentially due to the water flowing over it in small rills and gullies, joining to form small streams, which combine to form rivers. The process of these watercourses eroding and conveying water is a continuous process and has been going on since the formation of this planet and the elements surrounding it. Hence rivers are ever changing but in a man's lifetime it may not be much depending on the land space through which it passes. The general adjective fluvial (from Latin fluvial meaning river) is applied for the work done by river and fluvial system applies to all the area draining a particular river extending from the drainage divides in the source areas of water and sediment, through the channels and valleys of the drainage basin, to depositional area such as the coasts.

In this lesson, it is intended to understand the flow and sediment transport behaviour of natural channels and rivers. More applied theories of sediment dynamics and their application to natural rivers and artificial channels have been discussed in Lesson 2.10.

2.9.1 The fluvial systems

Conceptually the fluvial system of the river valley can be divided into three main zones (Figure 1), and described as under:

1. An ***erosional zone*** of runoff production and sediment source
2. A ***transport zone*** of water and sediment conveyance, and
3. A ***depositional zone*** of runoff delivery and sedimentation

FIGURE 1. The three zones of a fluvial system

In the first or upper zone the erosional process predominates and the stream and riverbeds are generally degraded. The streams join together at Confluences and their slopes are generally steep. The bed material is characteristically composed of boulders, cobbles or gravels.

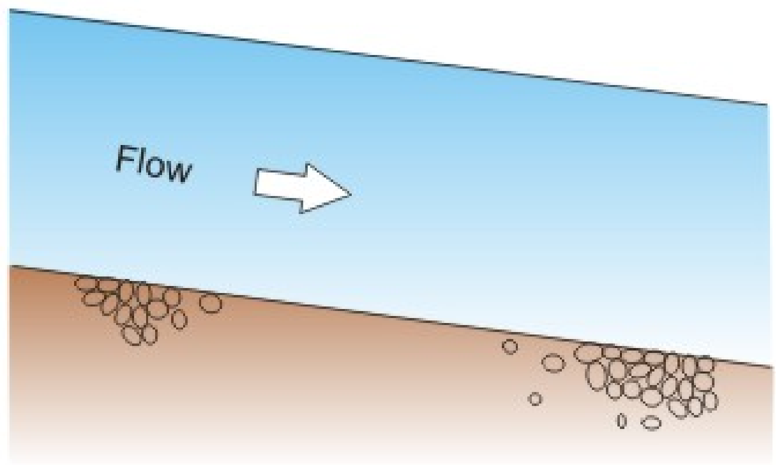
The second (middle) zone is characterized by near equilibrium condition between the inflow and outflow of water and sediment. The bed elevation in this equilibrium zone is fairly constant and the river generally flows in a single channel- there are few confluences or branching (as in the last zone) the sediment material generally composes gravels and sands of various sizes.

The lower zone is characterized by net sedimentation and riverbed aggradations. There is branching of the river into channels and the slope of these channels is rather flat. The bed material generally composes of fine sand to silt and clay.

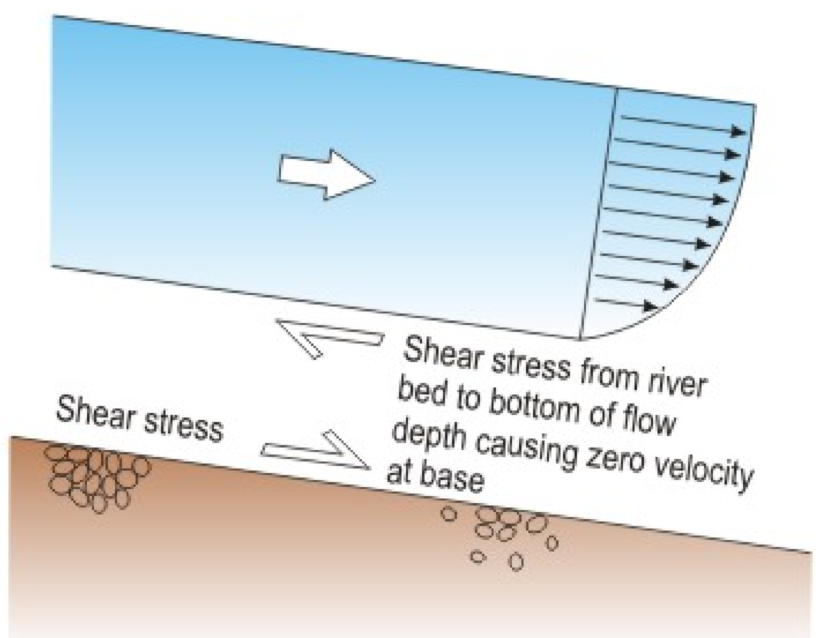
2.9.2 Sediment erosion, transport and deposition by river

It is amply clear that since rivers play a decisive role in land form evolution, the force of water is intricately connected to the dislodging of soil and rock particles and their conveyance. Where the power of water becomes less, it is forced to deposit the particles on its way.

When water flows over a surface, there exists a shear stress at the interface (Figure 2).



(a)



(b)

Figure 2. Interaction of forces between flow and riverbed
(a) General view
(b) Free body diagram showing shear stress at flow base-river bed interface.

Mathematically, the shear stress may be expressed in terms of the flow variables of a flowing river. If $\boldsymbol{S_0}$ is the slope of the channel bed, ***V*** is the velocity of flow and ***h*** is the water depth at a point then the shear stress (τ_0) at the interface of the water and the streambed is given as

$$\tau_0 = \rho g h S_f \tag{1}$$

Where S_f is the slope of the energy grade line (EGL) shown in the Figure 3. EGL represents the total energy of the stream at any point given by:

$$E = Z + h + \frac{V^2}{2g} \tag{2}$$

For explanation of the terms, one may refer to Lesson 2.8, and hydraulic text books like Ven Te Chow "Open Channel Hydraulics" (1959: McGraw Hill).

If the momentum of the moving water, its hydrostatic pressure its downstream weight component of water and the bottom friction are equated, the following equation results:

$$S_f = S_0 - \frac{\partial h}{\partial x} - \frac{\nu}{g}\frac{\partial V}{\partial x} - \frac{1}{g}\frac{\partial V}{\partial t} \tag{3}$$

From the equation (3) one may find S_f and evaluate shear stress (τ) using equation (1).

In deriving equation (3) it is assumed that the flow variables are not uniform and unsteady if they are not so then the derivatives with respect to the spatial dimension ***x*** and time ***t*** vanish, thus simplifying the equation to

$$S_f = S_0 \tag{4}$$

This is the case of the uniform flow and the corresponding equation for the shear stress (τ_0) is given as

$$\tau_0 = \rho g h S_0 \tag{5}$$

Thus, knowing the shear stress, it is possible to find out whether it is sufficient to dislodge a particle of diameter $\boldsymbol{d_s}$ from the streambed by a method suggested by Shields (Julien, 1995).

First, the dimensionless particle d_* is found out by the following formula

$$d_* = d_S \left[\frac{(G-1)\,g}{\nu^2}\right]^{1/3} \qquad (6)$$

Where ***G*** is the specific gravity of the sediment particle; ν is the kinematic viscosity of the fluid and ***g*** is the acceleration due to gravity. In some books, ***G*** is expressed as S_g.

The ratio of the shear force to bed particle defines the shields parameter τ_* as

$$\tau_* = \frac{\tau_0}{(\gamma_s - \gamma)\,d_S} = \frac{{u_*}^2}{(G-1)\,g\,d_S} \qquad (7)$$

Where τ_0 is the boundary shear stress and u_* is the shear velocity defined as

$$u_* = \sqrt{\frac{\tau_0}{\rho}} \qquad (8)$$

In equation (7), γ_s and γ are the specific weights have the sediment particle and water respectively. Note that τ_* depends on τ_0 , and which again is the function of S_0 (or S_f). Whether τ_* is competent enough to dislodge a particle depends on a critical value of shear stress given for particle of given diameter and density as given in Figure 3.

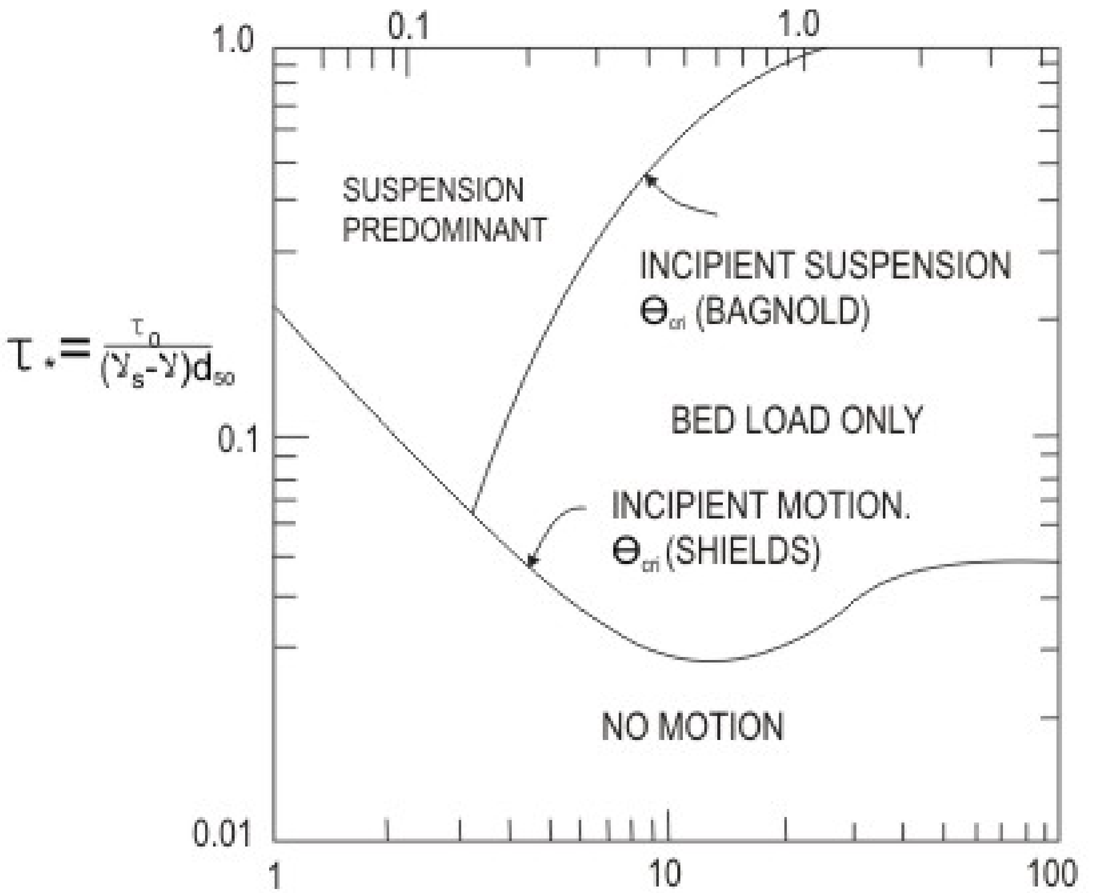


Figure 3. Particle motion diagram $d_* = d_{50}\left[\frac{(G-1)g}{v}\right]^{1/3}$

d_{50} : Mean diameter of particle(grain size)
G : Relative Density of particle
g : Accaleration due to gravidy
γ_s : Unit weight of sediment
γ : Unit weight of water
v : Kinematic viscosity of water
τ_* : Shields parameter or entrainment function

The parameter τ_{*c} given in Figure 3 is the critical value of Shields parameter corresponding to the beginning of the motion ($\tau_* = \tau_c$). This value of shear stress τ_c of certain particle sizes are as in the following table:

Particle Type	Diameter d_s (mm)	Critical Shear Stress τ_c (N/mm^2)
Cobble	130	111

Grand	8	5.7
Sand	0.25	0.194

Once the sediment particles are dislodged they get carried so long as τ_* is larger than τ_{*c} it may be remembered that in a natural stream, uniform flow rarely exists and the flow variables vary along the stream as the cross section and slope changes. However, if the shear stress τ_* may be sufficiently large it would continue to convey sediment particles of size smaller than a certain size $\boldsymbol{d_s}$.

2.9.3 Riverbed degradation

Channel degradation refers to the general lowering of the bed elevation that is due to erosion. In some cases, the bed material is fine and degradation will result in channel incision as shown in Figure 4.

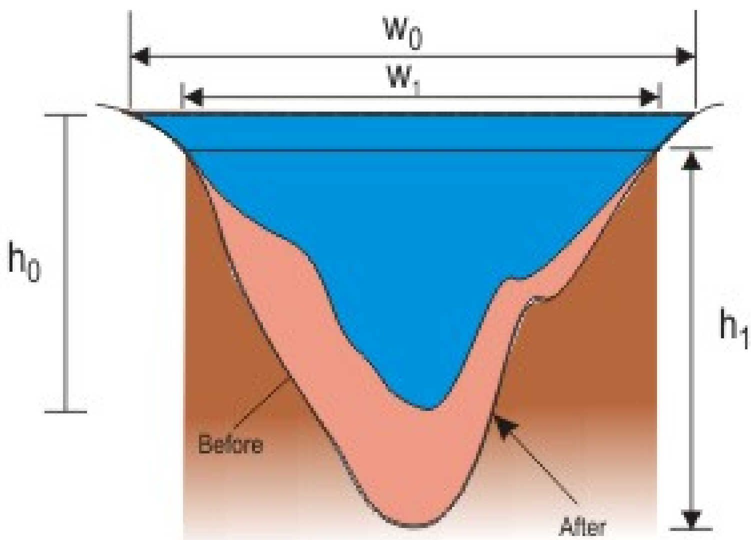


Figure 4. River cross section for bed degradation. Original river width w_0 and river depth h_0 befor degradation ; After degradation width changed to w_1 and depth to h_1

The phenomenon of degradation occurs when the sediment load being transported by a river is less than sediment transporting capacity of the river and the excess sediment needed to satisfy the capacity of the river will be scoured from erodable riverbed. Degradation results in channel incision and milder slopes, often this phenomenon is observed downstream of a dam constructed on a river (Figure 5).

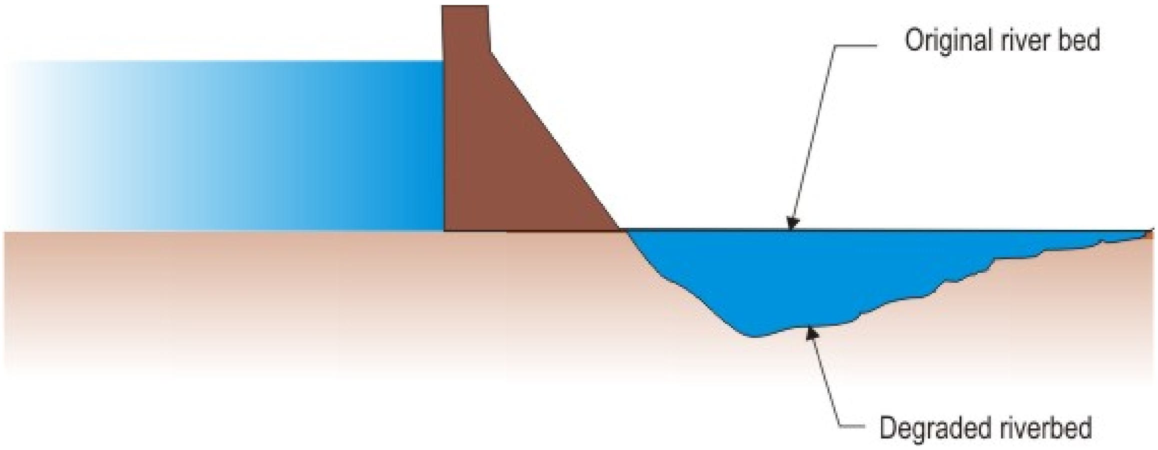


Figure 5. Longitudinal river profile showing river bed degradation along length of river below a dam

Dams constructed on rivers alter the equilibrium of flow of water and sediment in alluvial channels. Reservoirs tend to decrease the magnitude of flood flows by moderating them as a flood flows through, and conversely, they increase low flows by releasing the stored flood water at that time. The clear water release from the dam also causes the reach below the dam to degrade in the form of a wedge starting below the dam. The magnitude and extent of the degradation below the dams depends on the reservoir size and operation and on the size and availability of the alluvium below the dam.

A phenomenon related to streambed degradation is ***armouring***, where the coarsening of the bed material size results during degradation as finer particles get washed away. When the applied bed shear stress is sufficiently large to mobilize the large bed particles, degradation continues when the bed shear stress cannot mobilize the coarse bed particles, an ***armour layer*** forms on the bed surface. The armour layer becomes coarser and thicker as the bed degrades until it is sufficiently thick to prevent any further degradation.

2.9.4 River bed aggradation

When the sediment transporting capacity of a river at a point becomes less than the sediment load being carried, as a result of reduction the velocity due to an increase in cross section or reduction in slope of the river, the excess sediment get deposited on the river bed. As a result the riverbed rises, the phenomenon being termed as ***aggradation***. Often this phenomenon is noticed on the upstream of a dam (Figure 6), where the velocity of water in the reservoir is reduced as a result of increase in flow depth.

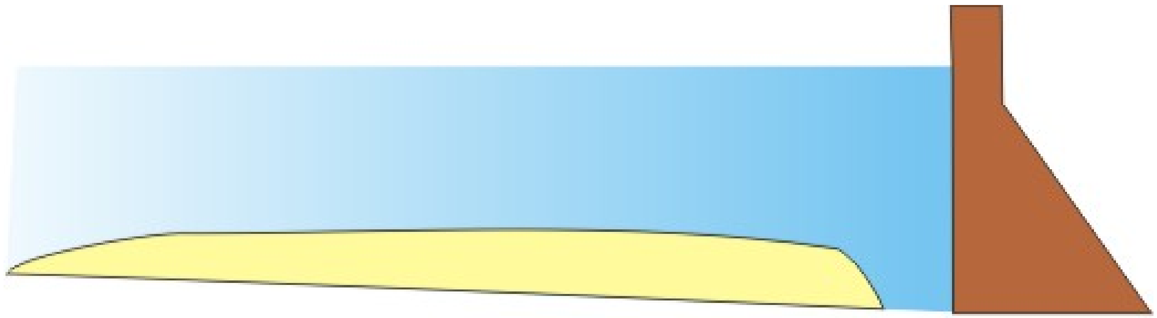


Figure 6. Longitudinal river profile showing sediment deposition on riverbed in the reservoir behind a dam

Channel aggradation may also occur in a river reach if due to geological reasons (say, increase of erosion of the catchment) the sediment load being conveyed to the river increases than that can be carried by the river in equilibrium. As a result the riverbed rises (Figure 7a) and forces the channel to carve out its path in a braided fashion (Figure 7b)

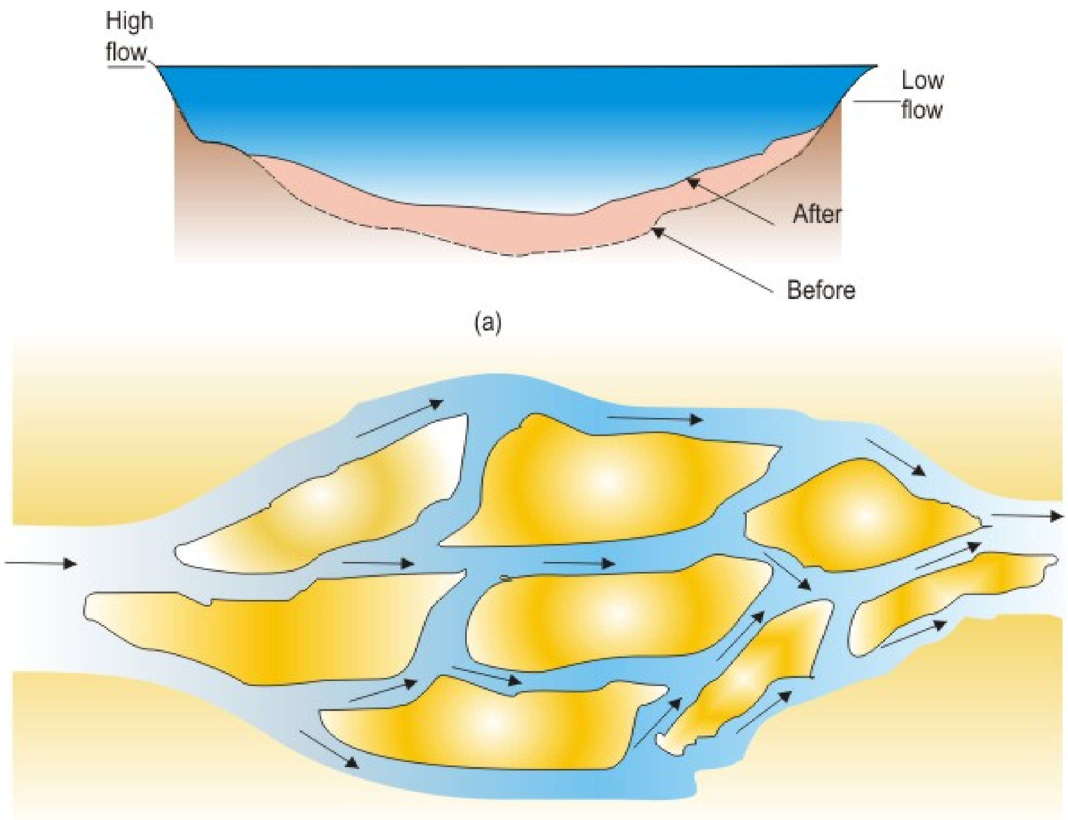


Figure 7. Effect of increase of sediment load in a river (a) Channel aggravation ; (b) Braiding

(b)

For ***braided rivers***, there is a tendency for stream to widen and become very shallow with bars subjected to rapid changes in morphology. At high flows braided streams have a low sinuosity and often appear to be straight at low flows, numerous small channels weave through the exposed bars.

Aggradation also occurs in a channel when there is a decrease of bed slope for example as the river emerges from the hills and enters relatively flat land. This has occurred markedly in the river Kosi, which has forced the river to change its course by more than a hundred kilometer westward in the last 200 years.

2.9.5 Bar formation in alluvial rivers

Bars refer to large bed forms on the bed of a river that are often exposed during low flows (Figure 8) these deposited segment mounds are not static and often get transported under high flows. They may again appear when the flow subsided but may not necessarily at the same location as the earlier ones.

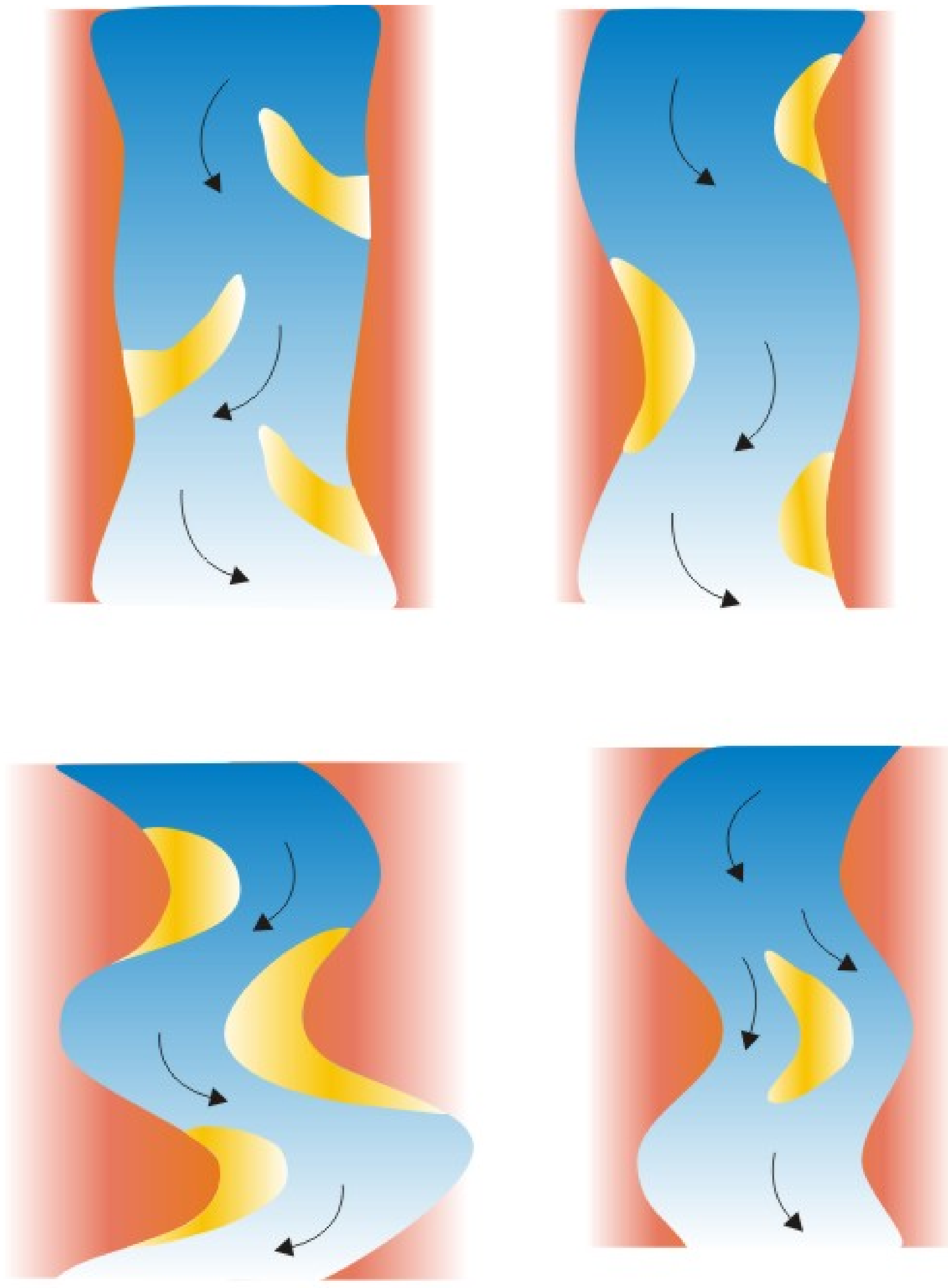


Figure 8. Bar for motion in rivers : (a) Alternate bars ; (b) Point bars ; and
(c) Mid-channel point bars

Alternate bars form in straight channels with deposits alternation from right bank to left bank. This type of bar forms where the Froude number of the river flow is high and the Shields parameter is close to incipient motion. Point bars form due to the presence of secondary flow of river bends (Figure 9).

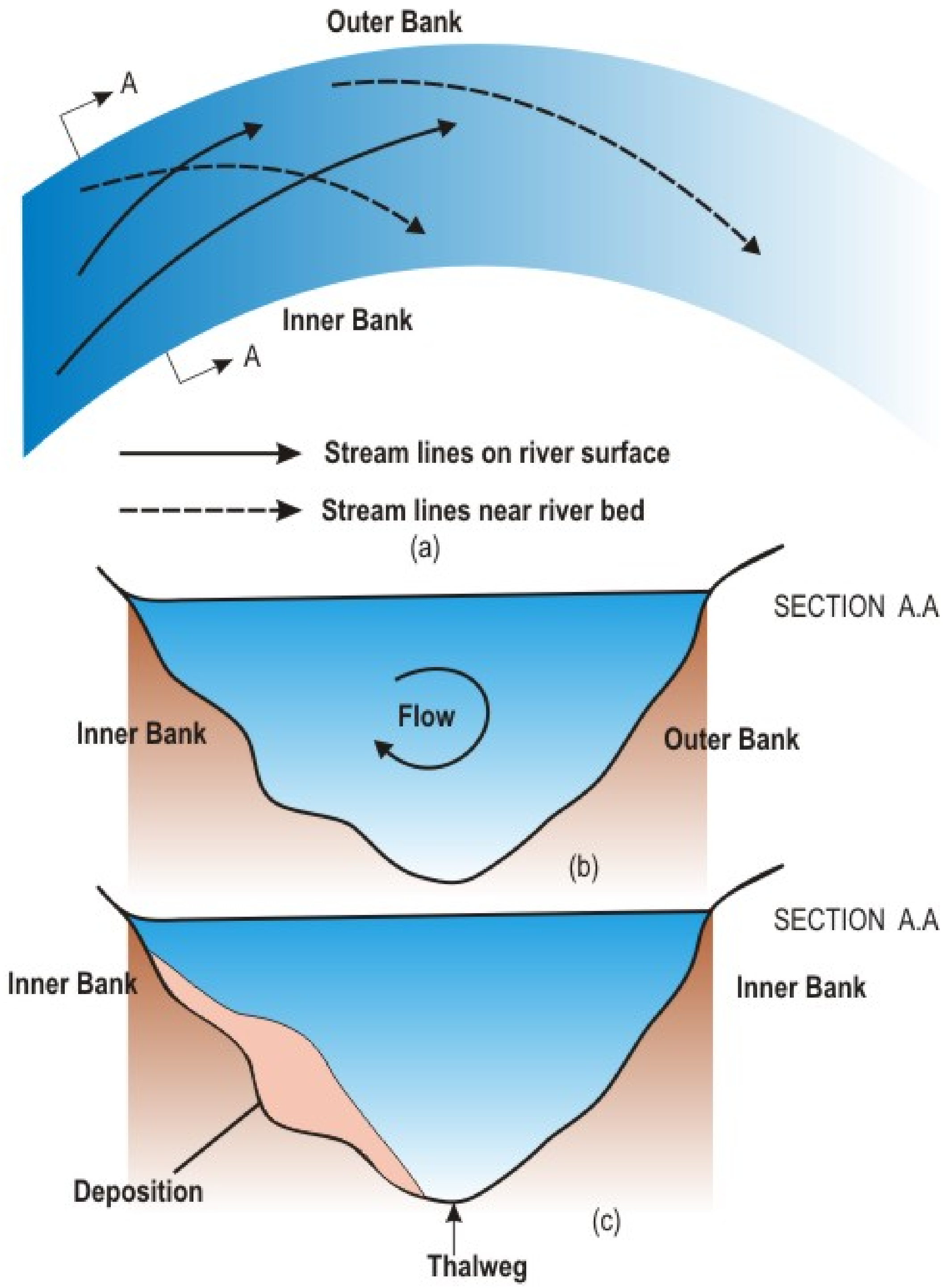


Figure 9. Secondary flow in river beds (a) Streamlines in plan at different levels ;
(b) Rotation movement of water in river cross-section ; and
(c) Effect of secondary flow : deposition on inner bank(compare point bars in fig 8 b)

As may be seen from the figure at river bends, there is a perceptible flow in a plane perpendicular to the river flow direction. At the outer bank the secondary flow causes erosion and at the inner bank it causes deposition, thus giving rise to point bar formation. The locus of the deepest points of the river along the length is called the thalweg. Most thalwegs pass through a succession of ***pools*** in the channel bed that are separated by ***riffles*** which might be sedimentary bed forms or bed rock ledges. The pools and riffles of the streambed cause the thalweg to have an irregular slope, rising and falling in the downstream direction.

2.9.6 River meandering

A river that winds a course not in a straight line but in a sinusoidal pattern (Figure 10) is called a ***meandering*** river.

Figure 10. A meandering river

It is the continued action of the secondary flow developed on the river bends that cause further erosion on the outer bank and deposition on the inner bank. The meandering action increases the length of the stream or river and tends to reduce the slope.

Many scientists have suggested different reasons for meandering to happen. The principles of external hypothesis have been used to explain the phenomenon like that of minimum variance proposed by Langbein and Leopold in 1962. The minimization involves the planimetric geometry and the hydraulic factors of depth, velocity and local slope. Mathematical explanations have also been put fort by Yang in 1976 that started the time rate of energy expenditure explains the formation of meandering streams. Other like Maddock (1970) and Chang (1984) use the principle of minimum stream power Julien (1985) treated meandering as a variation problem in which the energy integration corresponds to the functional variational problem, the solution of which is the sine-generated curve.

Quantitavely, scientists like Chitale (1970) argue that the primary cause of meandering is excess of total sediment load during floods. A river tends to build a steeper slope by depositing the sediment on the bed when the sediment load is in excess of that required for equilibrium. This increase in slope reduces the depth and increases the width of the river channel if the banks do not resist erosion. Only a slight deviation from uniform axial flow is then required to cause more flow towards one bank than the other. Additional flow is immediately attracted towards the former bank, leading to shoaling along the latter ascending the curvature of the flow and finally producing meanders in its wake.

2.9.7 Lateral movement of rivers and its bank instability

Channel meandering is a result of an ongoing bed and bank deformation by the flow in a self formed alluvial channel thus the meander sinuosity increases with the passage of time (Figure 11).

Figure 11. Sinuosity for a straight and meandering river (a) Straight: Sinuosity = 0; (b) Sinuosity = 1.1 ; and (c) Sinuosity = 1.5

In the above figure it is clear that the increase in sinuosity of a meandering river is associated with riverbank retreat. In some rivers, like Kosi the river has shifted or migrated laterally over large distances (predominantly in the westward direction). Here too the action is due to riverbank retreat, though the river doesn't display meandering significantly hence bank erosion is inseparably connected to lateral river migration.

Bank erosion consists of the detachment of grains or assemblages of grains from the bank, followed by fluvial entrainment. Though the riverbed may be composed of non-cohesive alluvial material the banks, on the other hand, may be composed of cohesive or non-cohesive soils. Cohesive, fine-grained bank material is easily eroded by the entrainment of the aggregates or the crumbs of the soil rather than individual particles, which are bound tightly together by electro-mechanical cohesive forces. Non-cohesive bank material is usually detached grain by grain and may leave a pronounced notch marking peak stage achieved.

When a section of bank-line fails and collapses lots of bank material slide or fall towards the toe of the bank. They may remain there until broken down insitu or entrained by the flow. Mass failures can be analyzed in geotechnical slope stability terms or as the result of fluvial and gravitational forces, which overcome resisting forces of friction, interlocking and cohesion, some of the bank failure modes are given in Figure 12.

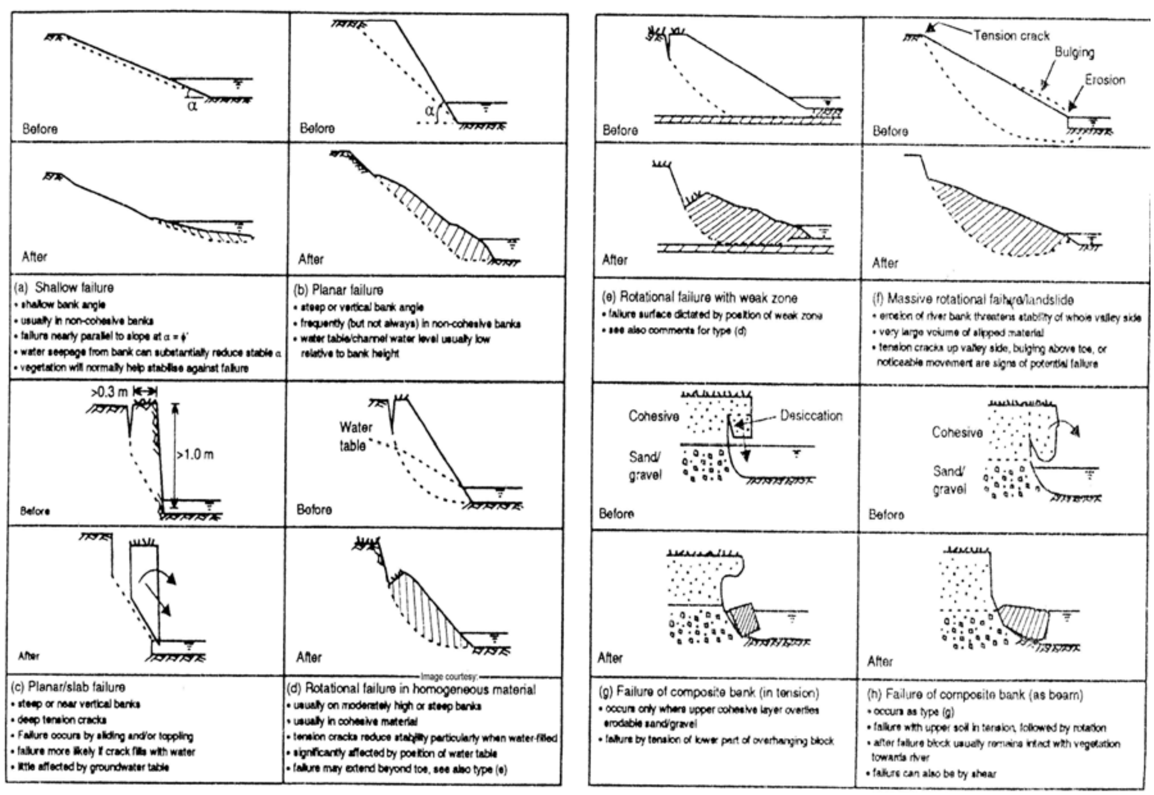


FIGURE 12. Modes of bank failure

Image courtesy: CR Thorne, RD Hey and MD Newson (Eds.) Applied Fluvial Geomorphology for River engineering and Management

2.9.8 River width adjustment

With increase or decrease of predominant flow and sediment load of a river, there is a change in river bed level, as discussed in 2.9.2 to 2.9.4. Although changes in channel depth caused by aggradation or degradation of the river bed can be simulated, changes in width cannot. When attempting to model a natural system like fluvial morphology this is a significant limitation because channel cross section usually changes with time, and adjustment of both width and depth (in addition to changes in planform, roughness and other attributes) are quite common.

River with adjustments may occur due to a wide range of morphological changes and channel responses. It may be widening (Figure 13) or narrowing (Figure 14).

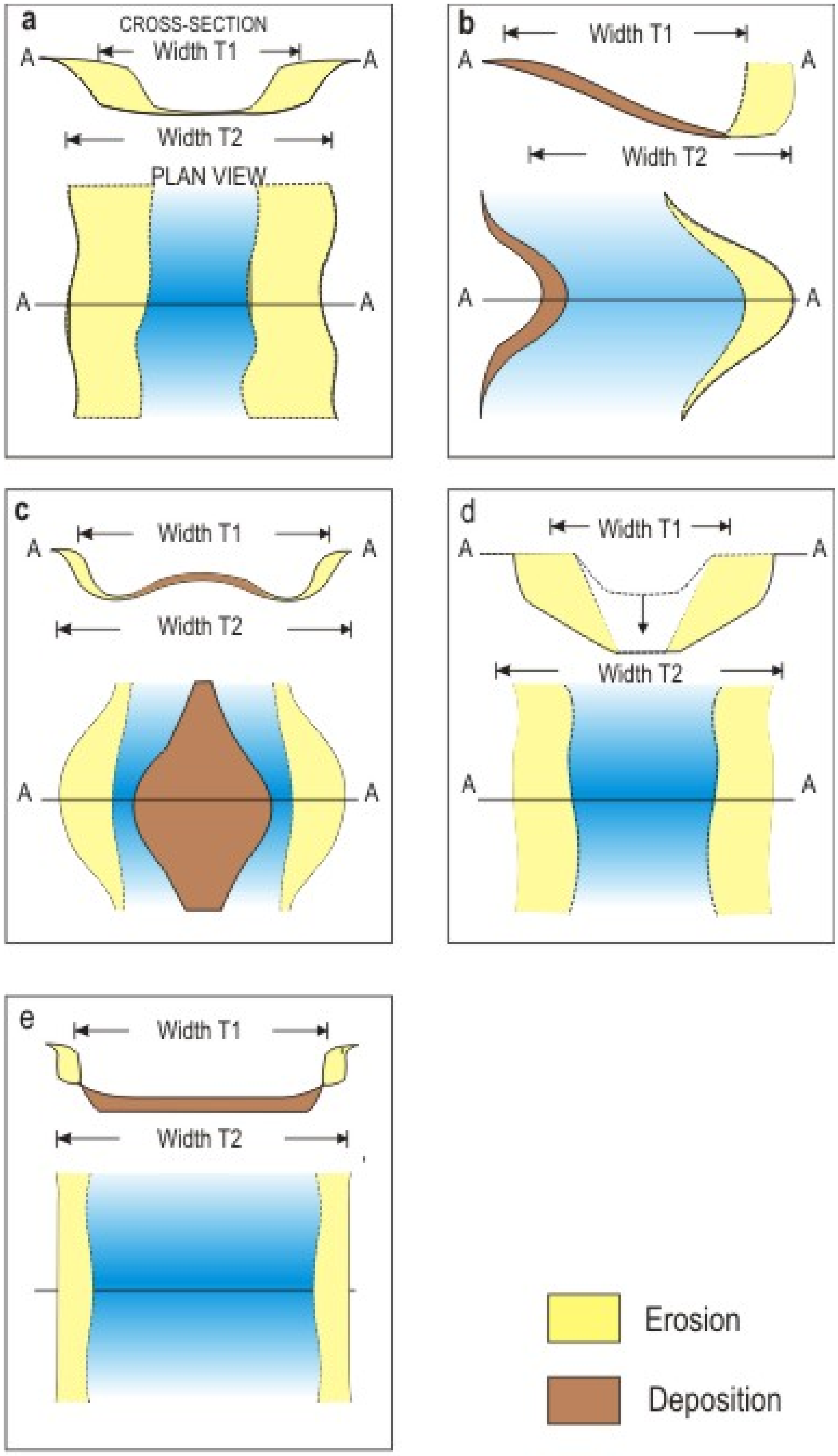


Figure 13. Modes of channel widening
Adapted from ASCE Task Committee , 1998 paper

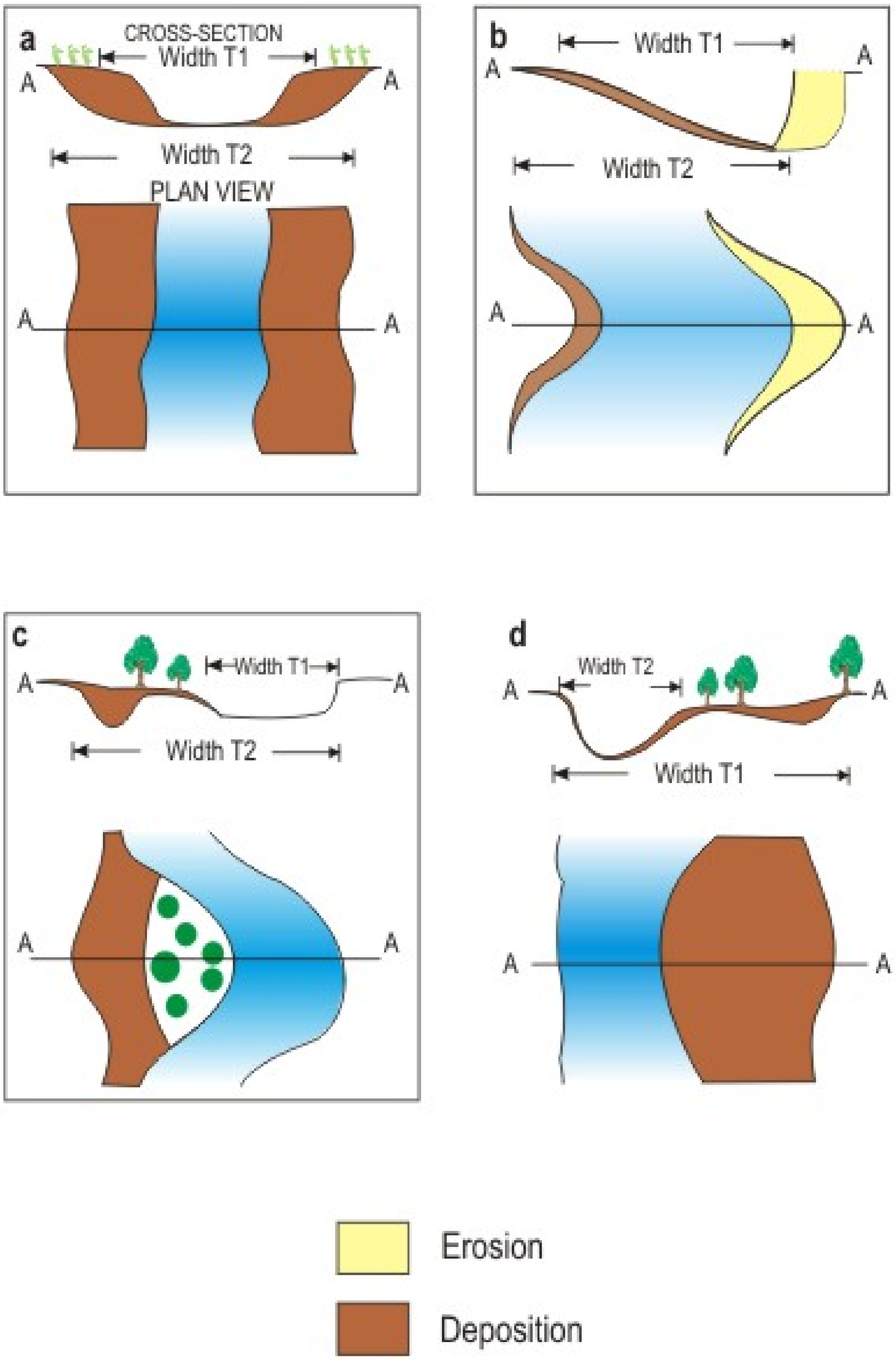


Figure 14. Modes of channel narrowing
Adapted from ASCE Task Committee , 1998 paper

Widening can occur by erosion of one or both banks without substantial incision (Figure 13a). Widening in sinuous channel may occur when outer bank retreat, as in a meandering channel Figure 13b. In braided rivers, bank erosion by flows deflected around growing braid bars is a primary cause of widening (Figure 13c). In degrading streams widening often flows the incision of the channel when the increased height and steepness of the banks causes then to become unstable (Figure 13d). Widening in coarse-grained, aggrading channels can occur when flow acceleration due to a decreasing cross sectional area, coupled with current deflection around growing bars, trigger bank erosion (Figure 13e).

Narrowing of rivers may occur through the formation of in-channels ***berms***, or ***benches*** at the margins (Figure 14a). Berm or bench often grows when bed levels stabilizes following a period of degradation and can eventually lead to creation of new, low- elevation flood plain and establishment of narrower quasi equilibrium channel. Narrowing in sinuous channels occur when the rate of alternate or point bar growth exceeds the rate of retreat of the cut bank opposite (Figure 14b). In braided channels, narrowing may result when a marginal ***anabranch*** (on offshoot channel) in the braided system is abandoned (Figure 14c). Sediment is deposited in the abandoned channel until it merges into the floodplain. Also, braided bars or islands may become attached to the floodplain, especially following a reduction in the formative discharge (Figure 14d). Island tops are already at about the flood plain elevation and attached bars are built up to flood plain elevation by sediment deposition on the surface of the bar, often in association with establishment of vegetation.

2.9.9 Hydraulic geometry of alluvial rivers

Width, depth, average velocity, average longitudinal bed slope of a natural river depends on many factors like discharge and sediment variation throughout the year and over the years, type of bed material and their variation in the river bed, type of bank material. There have been numerous attempts to find some relations between all the variables, which may be grouped as under (ASCE, 1998):

1) Regime theory and power law approach
2) External hypothesis approach
3) Tractive force methods.

The first of the three methods is traditionally the oldest and some of the initial contributors like Kennedy (1885), Lindley (1919), and Lacey (1929) based their hypotheses on research carried out on small rivers and artificial channels in India. Other noted contributors were Simons and Albertson (1963) and Blench (1969). The latter also based his study on the data gather from Indian rivers. Of the various formulas proposed so far by different researchers, the one by Lacey is quite popular in India though it has its own limitations as pointed out by Lane.

In fact, the data that was analyzed by him pertained to stable artificial channels of north India which were flowing through sandy materials and where carrying relatively small amount of bed material load. The following equations, which were originally presented by Lacey in FPS units, are being given here in terms of SI units

$$P = 4.75\sqrt{Q} \quad (9)$$

$$R = 0.47\left(\frac{Q}{f_1}\right)^{1/3} \quad (10)$$

$$S = 0.0003\ f_1^{5/3} / Q^{1/6} \quad (11)$$

$$U = 0.516\ f_1^{5/3} / Q^{1/6} = 10.8\ R^{2/3} S^{1/2} \quad (12)$$

$$f_1 = 1.76\sqrt{d_{50}} \quad (13)$$

In the above expressions, the variables stand for

- ***P:*** Wetted Perimeter (m)
- **R:** Hydraulic Radius (m)
- ***S:*** Channel Slope
- ***U:*** Average Velocity (m/s)
- ***Q:*** Discharge (m^3/s)
- ***f_1:*** Silt factor, a term used to define the fineness or coarseness of the channel bed material.
- ***d_{50}:*** Median size of bed material in mm

In India, hydraulic engineers have used these equations often for rivers, small and large. For wide rivers, the equations for ***R*** in terms of discharge ***q*** (discharge per unit width) obtained by combining the first two equations is used in the following form:

$$R = 1.35\ ((q^2/f_1)^{1/3} \quad (14)$$

A critical analysis of the formulae proposed by Lacey shows that sediment load is not included as one of the independent variables in the equation along with Q and d. Secondly, it has been pointed out by other researchers, that since the formulae of Lacey was based on data from channels of north India where the base is alluvial and banks are cohesive, the proposed equations are applicable largely to channels and canals with sandy bed and cohesive bars. Further the regime equations, as the equation proposed by Lacey are generally called, correspond to a given bank full discharge.

In the recent years the work by Julien and Wargadalam (1995) has attempted to refine the regime approach within a framework based on the governing principles of open channel flow. After studying data from 835 non-cohesive alluvial rivers and canals they arrived at the following relationship

$$h = 0.2\ Q^{0.33}\, d_{50}{}^{0.17} S_f{}^{-0.17} \tag{15}$$

$$W = 1.33\ Q^{0.44}\, d_{50}{}^{-0.11} S_f{}^{-0.22} \tag{16}$$

$$U = 3.76\ Q^{0.22}\, d_{50}{}^{-0.05} S_f{}^{0.39} \tag{17}$$

$$S_f = 0.121\ Q^{0.33}\, d_{50}{}^{-0.83} S_f{}^{0.33} \tag{18}$$

These are the simplified formulae of the more detailed ones and have been showed to agree quite well measured data. The variable sin the above equations are defined as follows

- ***h:*** Flow Depth (m)
- ***W:*** Water surface width (m)
- ***U:*** Average Velocity (m/s)
- ***S_f:*** Friction Slope
- ***d_{50}:*** Median slope of bed material (m)

The simplified equations proposed by Julian and Wargadalam are suitable when τ_* = 0.047, when Manning's equation is generally applicable to describe flow in the river. Higher segment transport implies higher velocity and slope and reduced width and depth. It may be recalled that Shield's parameter τ_* is defined as:

$$\tau_* = \left(\frac{\gamma\, h S_f}{(\gamma_S - \gamma)\, d_{50}} \right) \tag{19}$$

Two other approaches that have been proposed for describing the hydraulic geometry of an alluvial river are based on height following theories:

a) Extremal hypothesis
b) Tractive force approaches

These have been more commonly applied in the recent years. But the method proposed by Lane in threshold channel theory has also been used often to design canals in alluvium. This has been discussed in Lesson 2.10.

2.9.10 Bibliography

- ASCE Task Committee (1998) “River width adjustment. I: Processes and Mechanisms”, Jnl. of Hydraulic Engineering, American Society of Civil Engineers, Volume 124, Number 9, pp: 881-902.
- Leopold, L B and Langbein, W B (1962) “Concept of entropy in landscape evolution”, US geological surver professional paper number 500-A.
- Chang, H H (1984) “Analysis of river meanders” J Hydraulics Divn., ASCE, Volume 110.
- Chitale, S V (1970) “River channel patterns”, ” J Hydraulics Divn., ASCE, Volume 96.
- Kennedy, R G (1895) “The prevention of silting in irrigation canals”, Paper number 2826, Proc. Institution of Civil Engineers, London, Volume 119.
- Lacey, G (1929) “Stable channels in alluvium” Min. Proc. Institution of Civil Engineers, London, Volume 229.
- Maddock, T (1970) “Intermediate hydraulics of alluvial channels”, J Hydraulics Divn., ASCE, Volume 96.
- Lindley, E S (1919) “Regime channels”, proc. Punjab Engineering Congress, Volume 7.
- Blench, T (1969) Mobile-bed Fluviology, University of Alberta Press, Edmonton, Canada.
- Simons, D B and Albertson, M L (“Uniform water conveyance channels in alluvial material”, Trans. ASCE, Volume 128.
- Julien, P Y and Wargadalam, J (1995) “Alluvial channel geometry: Theory and applications”, J Hydraulic Engineering, Volume 121.

Module 2

The Science of Surface and Ground Water

Lesson 10

Sediment Dynamics in Alluvial Rivers and Channels

Instructional Objectives

On completion of this lesson, the student shall be able to learn the following:

1. The mechanics of sediment movement in alluvial rivers
2. Different types of bed forms in alluvial rivers
3. Quantitative assessment of sediment transport
4. Resistance equations for flow
5. Bed level changes in alluvial channels due to natural and artificial causes
6. Mathematical modelling of sediment transport

2.10.0 Introduction

In Lesson 2.9, we looked into the aspects of sediment generation due to erosion in the upper catchments of a river and their transport by the river towards the sea. On the way, some of this sediment might get deposited, if the stream power is not sufficient enough. It was noted that it is the shear stress at the riverbed that causes the particles near the bed to move provided the shear is greater than the critical shear stress of the particle which is proportional to the particle size. Hence, the same shear generated by a particular flow may be able to move of say, sand particles, but unable to cause movement of gravels. The particles which move due to the average bed shear stress exceeding the critical shear stress of the particle display different ways of movement depending on the flow condition, sediment size, fluid and sediment densities, and the channel conditions.

At relatively slow shear stress, the particles roll or slide along the bed. The particles remain in continuous contact wit the bed and the movement is generally intermittent. Sediment material transported in this manner is termed as the ***contact load***. On increasing the shear stress, some particles loose contact with the bed for some time, and hop or bounce from one point to another in the direction of flow. The sediment particles moving in this manner fall into the category of ***saltation load***.

Contact load and saltation load together is generally termed as ***bed load***, that is, the sediment load that is transported on or near the bed.

The further increase in shear stress, the particles may go in suspension and remain thus due to the turbulent fluctuations and get carried downstream by stream flow. These sediment particles are termed as ***suspension load***. In most natural rivers, sediments are mainly transported as suspended load.

Bed load and suspended load together constitute, what is termed as, ***total load***. A knowledge of the rate of total sediment transport for given flow, fluid and sediment characteristics is necessary for the study of many alluvial river processes. Engineers always need to bear in mind the fact that alluvial streams carry not only water but also sediment and the stability of a stream is closely linked with the sediment and transport rate. Alluvial channels must be designed to carry definite water and sediment discharges. In effect, the rate of total load transport must be treated as a variable affecting the design of a channel in alluvium. Knowledge of total sediment transport rate is essential for estimating the amount of siltation in the reservoir upstream of a dam or the erosion and scour of the river bed below a dam, as discussed in Lesson 2.9.

Analysis of suspended load and the corresponding bed materials of various streams for their size analysis have shown that the suspended load can be divided into two parts depending on the sizes of material in suspension vis-à-vis the size analysis of the bed material. One part of the suspended load is composed of these sizes of sediment found in abundance in the bed. The second part of the load is composed of those fine sizes not available in appreciable quantities in the bed. These particles, termed as the ***wash load***, actually originate from the channel bank and the upslope area.

2.10.1 Regimes of flow

It has been explained in Lesson 2.9 that when the average shear stress due to moving water on the river bed exceeds the critical shear stress, individual particles or grains making up the bed start moving. Since the particles are generally not exactly alike in size, shape or weight and also since a flow in a river with random turbulent fluctuations, all the bed particles do not start moving at the same time. Some particles move more than the rest, some slide and some hop depending on the uncertainties associated with the turbulent flow field and also the variation of drag due to particle shape. Gradually, a plane channel bed develops irregular or regular shapes of unevenness which are called bed forms which vary according to the flow conditions and are termed as "Regimes of flow" (Garde and Ranga Raju 2000), which is explained further below in this section. Regimes of flow will considerably affect the velocity distribution, resistance relation and the transport of sediment in an alluvial river or channel. The regimes of flow can be divided into the following categories:

1. Plane bed with no motion of sediment particles
2. Ripples and dunes
3. Transition, and
4. Antidunes

Plane bed with no motion of sediment particles:

When the sediment and flow characteristics are such that the average shear stress on the bed is les than the critical shear stress, the sediment particle on the bed does not move. The water surface remains fairly smooth if the Froude number is low. Resistance offered to the flow is on account of the grain roughness only, and the Manning's equation can be used for prediction of the mean velocity of flow.

Ripples and Dunes:

The sediment particles on the bed start moving when the average shear stress of the flow exceeds the critical shear stress. This results in small triangular undulations as the channel bed and is known as ripples (Figure 1).

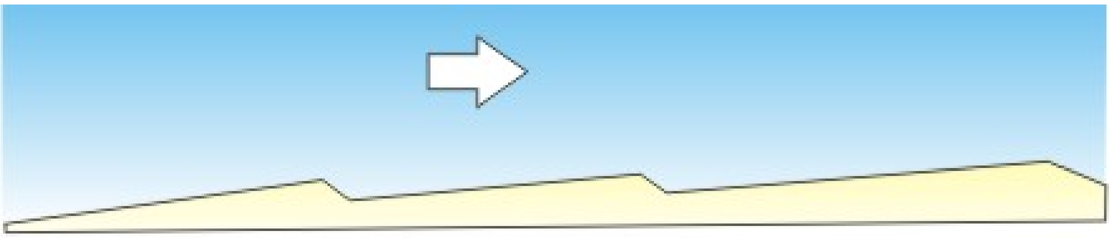


FIGURE 1. Ripples on the bed of alluvial river or channel for low flow velocity

Ripples do not occur for sediment particles coarser than 0.6 mm. The distance between the successive crests of the ripples is usually less than 0.4m and the height from the crest to the trough is usually less than 0.04m. The sediment movement is confined in the region near the channel bed.

With increase in discharge, and consequently the average bed shear stress, the ripples grow in sizes which are then termed as dunes (Figure 2).

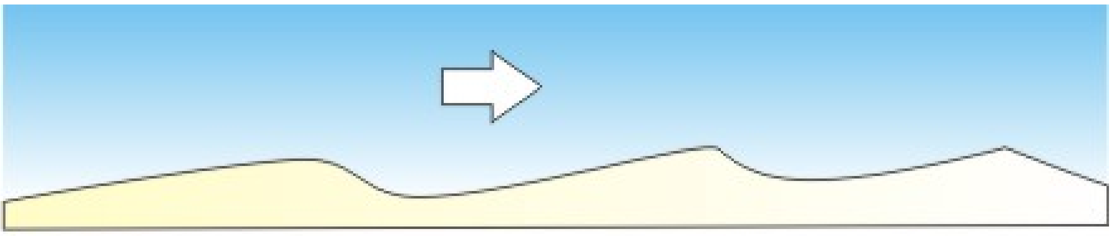


FIGURE 2. Dunes or channel bed for higher velocities

Dunes are also triangular in shape but are larger than ripples. The triangular sections are not symmetric and the upstream face is inclined at about 10 to 20 degrees and downstream face at an angle of about 30 to 40 degrees with the horizontal. In rivers, dunes may be quite long and also the height (vertical distance between the crest and troughs) may be great. For example, the dunes found in Lower Mississippi river have been found to be about 12m height on an average and length of the order of few hundred meters (Garde and Ranga Raju, 2000). These bed forms are not static, which means that they gradually move forward with time, of course at a very slow and creeping velocity much less than the velocity of flow.

Transition:

With further increase in discharge over the dune bed, the ripples and dunes are washed away, and only some very small undulations are left. In some cases, the bed may become nearly flat but the sediment particles remain in motion. With slight increase in discharge, the bed and water surfaces attain a shape of sinusoidal wave form, which are called standing waves (Figure 3).

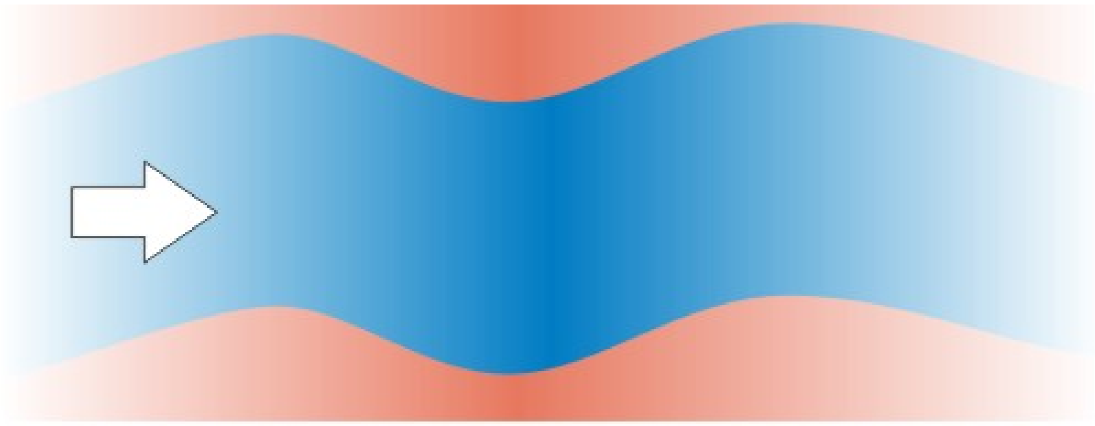


FIGURE 3. Standing waves or symmetrical sand waves and water surface waves in phase for even higher velocities

These waves form and disappear and their size doesn't increase much. Thus, in transition regime, rapid changes in bed and water configuration occur with relatively small changes in flow conditions. The Froude number is relatively high but the flow conditions are sub-critical.

Antidunes:

When the discharge is increased further and the Froude number increases to more than one, indicating super critical flow, the standing waves, which are symmetrical sand and water waves in the phase, move slowly upstream and break intermittently. These are called antidunes because the movement of the direction of dunes is backwards compared to the direction of flow. Since supercritical flow is rare in case of natural streams and channels, this type of bed forms do not occur generally in nature.

2.10.2 Quantities of sediment transport rates

There are various formulae predicting the amount of sediment transported as

- Bed load
- Suspended load
- Total load

From water resources engineering point of view one generally requires the sediment transport rate of total load, and hence, the methods for predicting this would be discussed here. Of course, in many practical situations, the suspended load is measured or estimated, through the equation proposed by Einstein (1942), as this constitutes about 80 to 90 percent of the total load. After estimating suspended load a certain percentage of it is added to estimate total load. [Please note that the researcher Einstein mentioned here is Hans Albert Einstein, the son of the famous physicist Albert Einstein].

Though there are probably over a dozen total load relationships essentially using a single representative size of the sediment mixture. While some of the methods may be considered semi-empirical, most of them are based on dimensional analysis and graphical plotting or regression analysis. Hence the basis for the choice of an appropriate sediment transport relation in practice can only be the relative accuracy of these methods. Yang (1996) has shown through examples that the prediction of total load by different formulae may vary by as much as four times of one another. According to Garde and Ranga Raju (2000), the methods proposed by the following researchers give better results than other methods:

- Ackers-White (1973)
- Engelund-Hansen (1967)
- Brownlie (1981)
- Yang (1973)
- Karim-Kennedy (1990)

There are other methods like that proposed by Van Rijn (1984a and b) which estimate the bed load and suspended load components of the total load separately.

Some of the methods to calculate total load are mentioned in the following sections.

2.10.3 Ackers and White (1973) method

Ackers and White (1973) postulated that one part of the shear stress on the channel bed is effective in causing motion of coarse sediment, while in the case of fine sediment, suspended load movement predominates for which total shear stress is effective in causing sediment motion. This method can be applied by following the steps mentioned below:

1. Determine the value of d_*, the dimensional particle diameter, defined as:
2.

$$d_* = d\left[\frac{g}{\upsilon^2}\left(\frac{\gamma_s}{\gamma}-1\right)\right]^{1/3} \qquad (1)$$

Where the parameters on the right hand size of equation (1) are:

- *d*: Average particle diameter
- *g*: Acceleration due to gravity
- υ: Kinematic Viscosity of water

- γ_s : Specific weight of sediment
- γ : specific weight of water

3. Determine the values of the coefficients c_1, c_2, c_3 and c_4 as:

For 1< d_* <60

c_1 = 1.00 – 0.56 log d_*

$$c_2 = e^{\left[2.86 \log d_* - (\log d_*)^2 - 3.53\right]}$$

$$c_3 = \frac{0.23}{d_*^{1/2}} + 0.14$$

$$c_4 = \frac{9.66}{d_*} + 1.34$$

For $d_* > 60$

$$c_1 = 0.00 \quad c_2 = 0.025 \; c_3 = 0.17 \quad c_4 = 1.5$$

4. Compute the value of particle mobility number F_g, given by the following expression

$$F_{gr} = U_*^{c_1} \left[g.d \left(\frac{\gamma_s}{\gamma} - 1 \right) \right]^{-1/2} \left[\frac{V}{\sqrt{32} \log \frac{10h}{d}} \right]^{1-C_1} \qquad (2)$$

where U_* = Shear velocity = $\sqrt{\frac{\tau_0}{\rho}}$

τ_0 = bed shear stress

V= Average flow velocity

h = water depth

5. Compute the value of dimensionless sediment transport function G_{gr} from the following expression

$$G_{gr} = c_2 \left(\frac{F_{gr}}{c_3} - 1 \right)^{C_4} \qquad (3)$$

6. Compute sediment concentration, X, in ppm (parts per million) by weight of fluid using the following expression:

$$X = G_{gr} \frac{d}{h} \frac{\gamma_s}{\gamma} \left(\frac{V}{U_*} \right)^{C_1} \qquad (4)$$

7. Compute total sediment load Q_T by multiplying sediment concentration (X), with discharge of the following water Q, that is,

$$Q_T = Q.X \tag{5}$$

2.10.4 Engelund and Hansen's method

Engelund and Hansen (1967) proposed a total load equation relating the sediment transport to the shear stress and the friction factor of the bed. The following steps illustrate the method of application of their theory:

1. Compute the parameter θ, the dimensionless shear stress parameter by the following equation

$$\theta = \frac{\tau_0}{(\gamma_S - \gamma)d} \tag{6}$$

Where

- τ_0 is the bed shear stress
- γ_S is the density of sediment particles
- γ is the density of water
- d is the diameter of bed particles

2. Compute f ' the friction factor of the bed using the following expression

$$f^1 = \frac{2 g S_f h}{V^2} \tag{7}$$

Where

- g is the acceleration due to gravity
- S_f is the energy slope
- h is the depth of flow
- V is the average flow velocity

3. Obtain the total sediment load Q_T from the following equation

$$Q_T = 0.1\left[\gamma_s\left(\frac{\gamma_s - \gamma}{\gamma}\right) g\, d^3\right]^{1/2} \frac{\theta^{5/2}}{f^1} \tag{8}$$

There are several other methods for computing total load or the components suspended load and bed load separately. The interested reader may refer to the book by Garde and Ranga Raju (2000).

2.10.5 Resistance to flow in alluvial rivers

Open channels with movable beds and boundaries are commonly encountered in water resources engineering. In contrast, some artificial channels and hydraulic structures with solid floors constitute a relatively smaller portion where the roughness coefficient can be treated as constant. In these cases, a resistance formula can be applied directly for the computation of velocity, slope, or depth, once a roughness coefficient has been determined. Although this concept is also used for natural channels but strictly speaking, for these kinds of channels, a resistance formula cannot be applied directly without knowledge of how the resistance coefficient will change under different flow and sediment conditions. Extensive studies have been made by different researchers for determination of roughness coefficients of alluvial beds. Their results differ from each other. Most of these studies have been based on limited laboratory data. Uncertainties remain regarding the applicability and accuracy of laboratory results to field conditions.
The resistance equation expresses relationship among the mean velocity of flow ***V***, the hydraulic radius ***R***, and characteristics of the channel boundary. For steady and uniform flow in rigid boundaries boundary channels, the Keulegan's equations (logarithmic type) or power-law type of equations (like the Chezy's and the Mannings equations) are used. Keulegan(1938) obtained the following logarithmic relations for rigid boundary channels:

For smooth boundaries

$$\frac{V}{\sqrt{\tau_0 / \rho}} = 5.75 \log\left[\sqrt{\frac{\tau_0}{\rho}} \frac{R}{\upsilon}\right] + 3.25 \qquad (9)$$

For rough boundaries,

$$\frac{V}{\sqrt{\tau_0 / \rho}} = 5.75 \log\left[\frac{R}{k_s}\right] + 6.25 \qquad (10)$$

where

- τ_0 is the bed shear stress
- ***R*** is the hydraulic radius
- ρ is the density of water
- υ is the Kinematic viscosity
- k_s is the grain roughness height
- ***V*** is the average velocity at a point

For the range of 5< R/k_s<700, the Mannings equation is

$$V = \frac{1}{n} R^{2/3} S^{1/2} \qquad (11)$$

This has been found to be as satisfactory as the Keulegan's equation for rough boundaries (equation 10). In equation (11), ***n*** is the Manning's roughness coefficient, which can be calculated using the Strickler's equation

$$\boldsymbol{n} = \frac{k_S^{1/6}}{25.6} \qquad (12)$$

Where, $\boldsymbol{k_s}$ is the grain roughness height in metres.

Another power-law type of equation is given by Chezy in the following form:

$$V = C\sqrt{RS} \qquad (13)$$

Where ***C*** is the Chezy's coefficient of roughness. Comparing Manning's and Chezy's equations, one obtains:

$$\frac{V}{\sqrt{\tau_0/\rho}} = \frac{C}{\sqrt{g}} = \frac{R^{1/6}}{n\sqrt{g}} = \left(\frac{R}{K_S}\right)^{1/6} \cdot \frac{25.6}{\sqrt{g}} \qquad (14)$$

In case of alluvial channels, where the bed is composed of mobile material, like sand, so long as average bed shear stress to on the boundary of the channel is less than the critical shear □$_c$ the channel boundary can be considered rigid and any of the resistance equations valid for rigid boundary channels would yield results for alluvial channels too. However, as soon as sediment movement starts, undulations appear on the bed, thereby increasing the boundary resistance. Besides, some energy is required to move the grains. Further, the sediment particles in suspension also affect the resistance of alluvial streams. The suspended sediment particles dampen the turbulence or interfere with the production of turbulence near the bed where the concentration of these particles as well as the rate of turbulence production is maximum. It is therefore, obvious that the problem of resistance in alluvial channels is very complex and the complexity further increases if one includes the effect of channel shape, non-uniformity of sediment size, discharge variation, and other factors on channel resistance. None of the resistance equations developed so far takes all these factors into consideration.

The methods for computing resistance in alluvial channels can be grouped into two broad categories.

a) Those which deal with the overall resistance offered to the flow using either a logarithmic or power type relationship for the mean velocity, and
b) Those in which the total resistance is separated into the resistance given by the grains of sand forming the channel bed and the resistance of the undulations in the bed. Thus, in this method, the total resistance is studied as a combination of grain resistance and form resistance.

These two methods are discussed in the following sections.

2.10.6 Formula for total resistance in alluvial channels

One of the earliest resistance relationships for alluvial channel flow was proposed by Lacey on the basis of stable canal data from northern India. The equation for mean velocity in SI units is

$$V = 10.8\, R^{2/3} S^{1/3} \tag{15}$$

where ***R*** is the hydraulic radius, ***S*** is the friction slope; ***V*** is the average velocity. This equation, however, is not applicable at all stages of the river and hence, it cannot be used reliably for all types of alluvial rivers and channels.

Another relation obtained by Indian researchers is that by Garde and Ranga Raju and is expected to yield results with accuracy $\pm$30 percent (Ranga Raju, 1993). The method can be applied by following the procedural steps mentioned below. Remember that the average velocity ***V*** to be obtained for a given friction slope $\boldsymbol{S_f}$. Another parameter that is related to the roughness of the bed material is $\boldsymbol{d_{50}}$, or the mean diameter of bed grains.

a) For a particular water level, find out the hydraulic radius ***R*** and the area of cross section ***A***.

b) From Figures 4 and 5, determine factors $\boldsymbol{k_1}$ and $\boldsymbol{k_2}$ corresponding to $\boldsymbol{d_{50}}$.

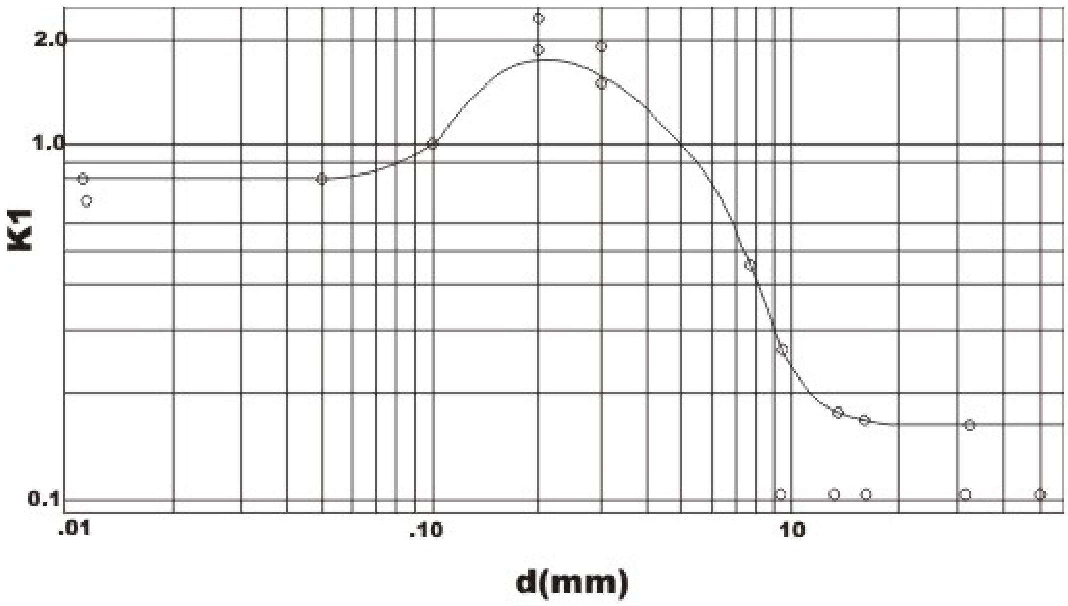


FIGURE 4. Variation of K1 with sediment size

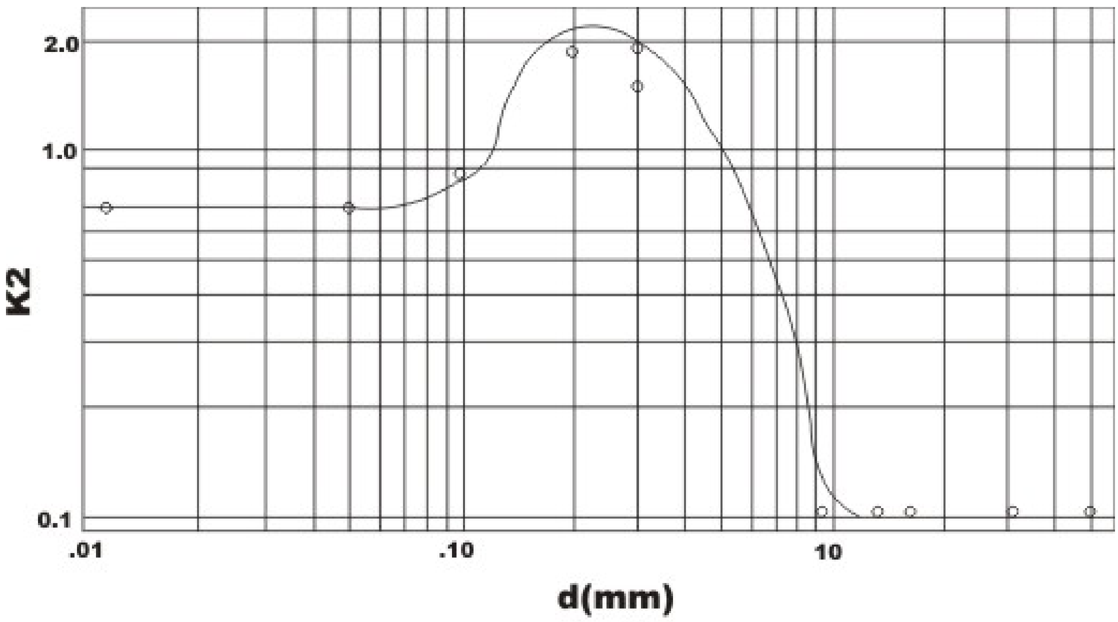


FIGURE 5 Variation of K2 with sediment size

c) Compute $f_1 = K_2 \left(\frac{R}{D} \right)^{1/3} \frac{S_f}{\left(\Delta \gamma_s / \gamma \right)}$

Where

- ***D*** is the water depth
- S_f is the friction slope
- $\Delta\gamma_s$ is equal to $\gamma_s - \gamma$ in which
- γ_s is the specific weight of sediment and
- γ is the specific weight of water

d) From Figure 6, read the value of ***f₂*** $= K_1 \frac{V}{\sqrt{\left(\frac{\Delta\gamma_S}{\gamma} \right).g.R}}$

Corresponding to f_1 computed in step (c).

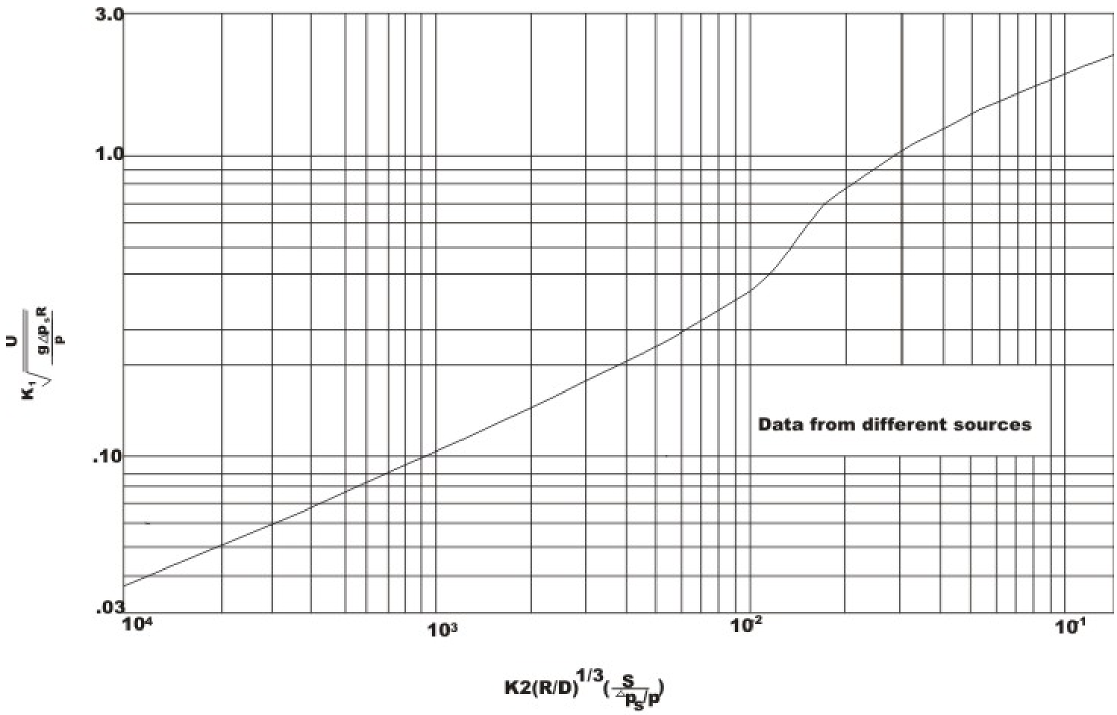


FIGURE 6 Roughness predictor for alluvial channels

e) Knowing f_2, compute the value of ***V***

There are several other methods available to find the total resistance for an alluvial stream and the interested reader may refer to Garde and Ranga Raju (2000).

2.10.7 Formula for separate resistance due to grain and form

Here too, there are several methods and the important ones are explained in Garde and Ranga Raju (2000). One of the formulae is quoted here, due to van Rijn (1984), states the following:

$$\frac{V}{\sqrt{\tau_0 / \rho}} = 5.75 \log\left(\frac{12R}{K_S}\right) \tag{16}$$

Where

- V is the average velocity
- τ_0 is the bed shear stress
- R is the hydraulic radius

K_s is the sum of the roughness corresponding to grain and form resistance, that is

$$K_s = K_{s1} + K_{s2}$$

in which $K_{s1} = 3\, d_{90}$ and
K_{s2} is related to the height of undulation (h) and the length of the dunes (L) as given by the formula
$K_{s2} = 1.1\, h\, (1\text{-}e^{-2gh/l})$

2.10.8 Bed level changes in alluvial channels

In Lesson 2.9, qualitative description of alluvial stream bed changes due to construction of a dam was discussed (Figure 7).

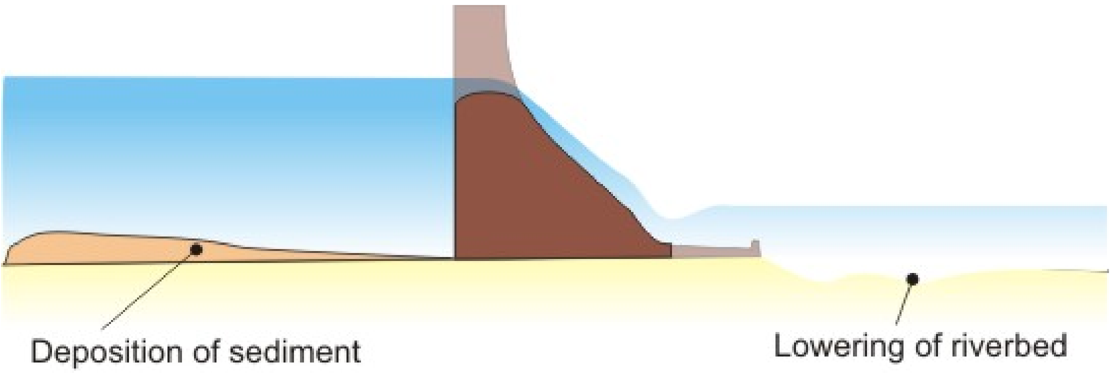


FIGURE 7. Alluvial riverbed level changes due to a Daim constructed across a river

As water resource engineers, one is interested to find a quantitative assessment to the amount of sediment deposition in the reservoir on the dam upstream or the extent of riverbed scour in the reach downstream. In the following sections we discuss the means by which it may be done. Other situations in which alluvial riverbed levels get changed due to the presence of a structure in the river are given in the following examples.

Scour around bridge piers

Bridges, crossing alluvial rivers and channels have their piers resting on foundations within the rivers. As may be seen from the figure, the foundation well extends within the river bed and determination of its depth depends to what extent the riverbed would scour during floods (Figure 8).

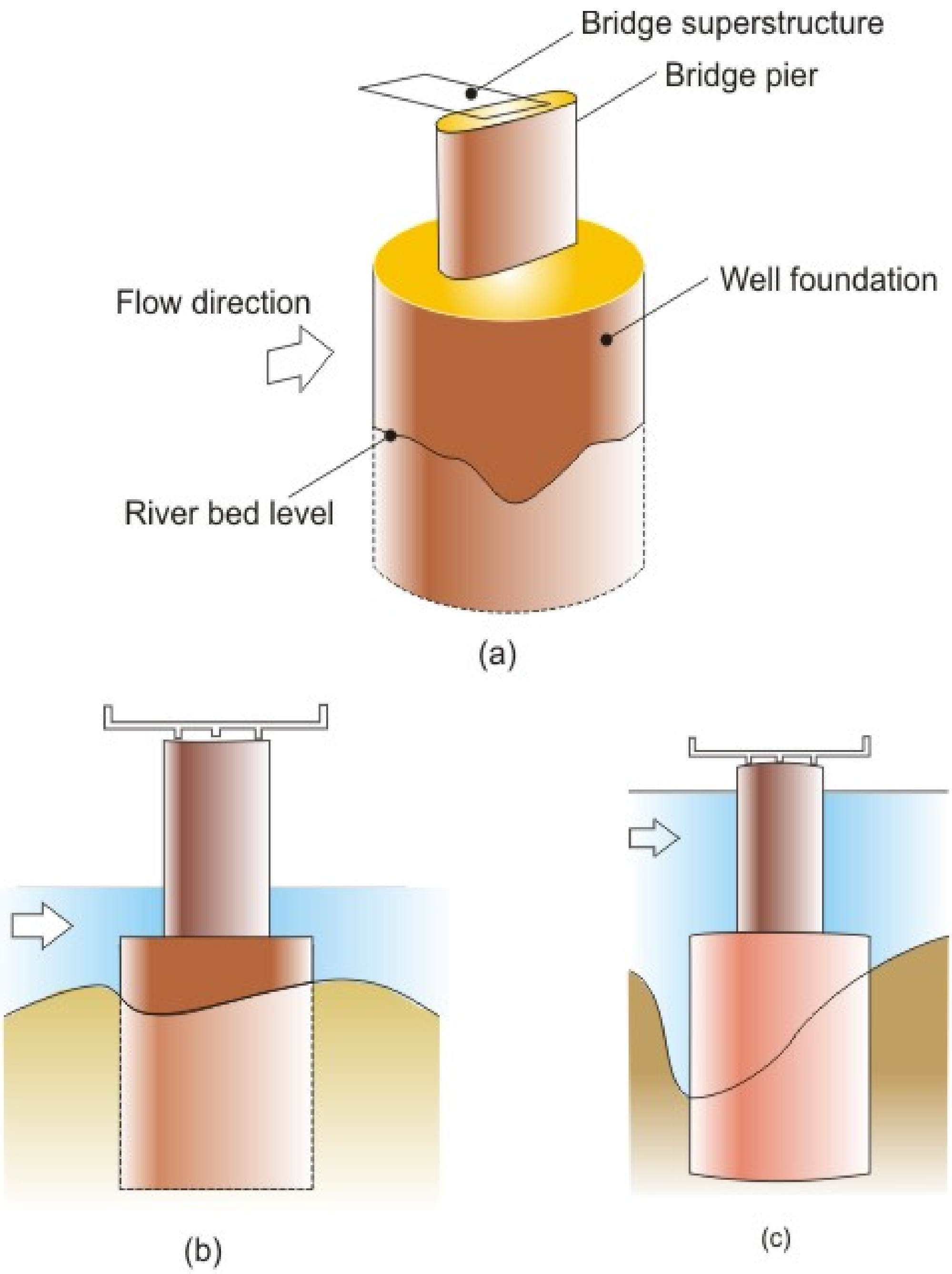


FIGURE 8 . Alluvial riverbed level changes near bridge pier
a) General view of bridge and pier
b) Bed level changes not much during low flows
c) Scour or lowering of riverbed levels during floods

Some bridges are constructed on piles, instead of wells but there too the length of the piles depends on the extent of scour that is expected during floods.

It may be mentioned that the deepening of riverbed around bridge foundations occurs only during the passage of a high flood. Once the flood peak passes and the flood starts receding, the scoured riverbeds start getting filled up with sediment carried by the river.

Sediment movement near intakes

Water intakes for irrigation or water supply are often faced with the problem of excess sediment removal or deposition near its vicinity (Figure 9). Sometimes, the gates of the intake and that of the barrage are not properly coordinated which results at undesirable sediment erosion and deposition.

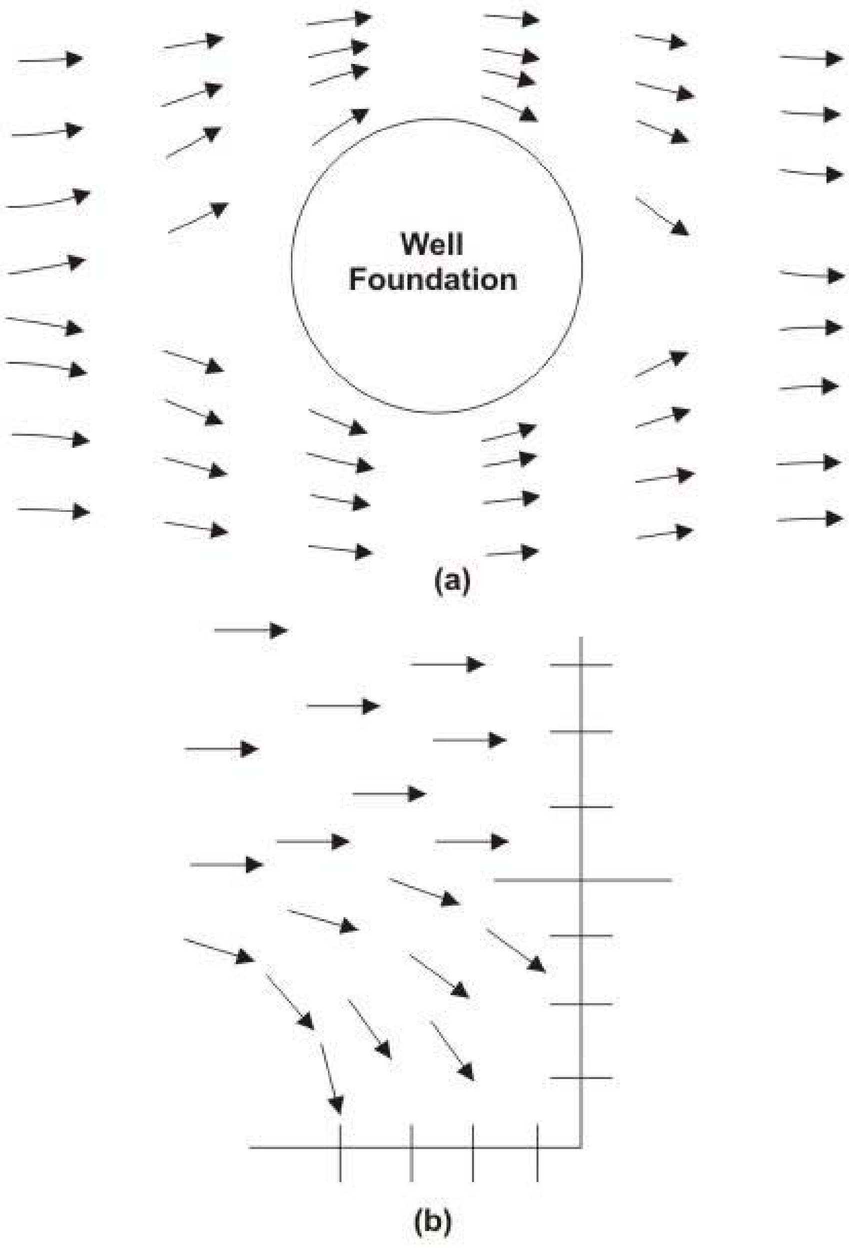


FIGURE 9 . Examples of two dimensional flow in plan
a)Flow around bridge pier well foundation
b) Flow near barrage and canal head regulator

The example of sediment deposition behind dam and erosion on its downstream may be treated as that of one dimensional flow (Figure 7). The other two examples that of scour around bridge piers and sediment movement near water intakes are examples of two dimensional flow (Figure 10).

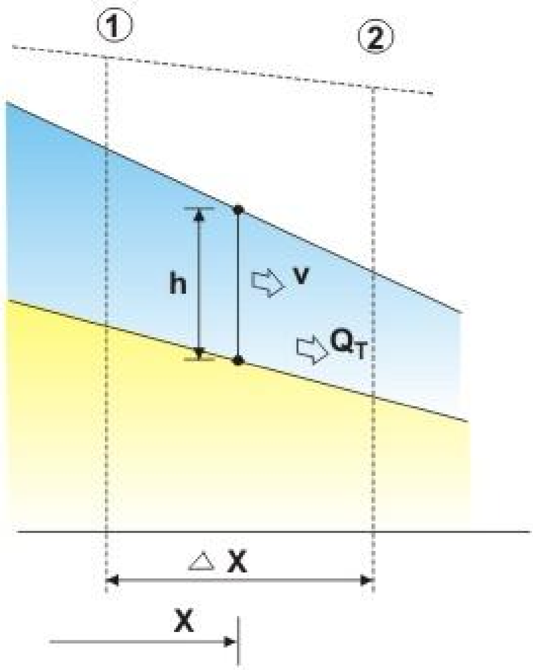


FIGURE 10 Definition sketch for derivation of bed level change due to a given sediments transport rate Q_T at a location x

In order to quantitatively evaluate the amount of sediment movement, it is necessary to solve the relevant governing equations. An introduction to these is given in the following sections.

2.10.9 Mathematical models for unsteady sediment movement

Consider a length Δ**x** of a channel of width ***B*** transporting sediment at a weigh rate of ***Q_T*** (Figure 11) at a location ***x*** in the channel. The volume rate of sediment transport through the section is **Q_T/γ_s**, where ***γ_s*** is the specific weight of the sediment particles.

If ***Q_T*** is the weight rate of sediment transport at ***x***, then that at section ***1***, it is $Q_t - \frac{\partial Q_t}{\partial x}\frac{\Delta x}{2}$ and at section **2** is $Q_t + \frac{\partial Q_t}{\partial x}\frac{\Delta x}{2}$.

Correspondingly, the volume of sediment entering through section ***1*** is $\frac{1}{\gamma_S}\left[Q_t - \frac{\partial Q_t}{\partial x}\frac{\Delta x}{2}\right]$ and that leaving through section ***2*** is $\frac{1}{\gamma_S}\left[Q_t + \frac{\partial Q_t}{\partial x}\frac{\Delta x}{2}\right]$. It follows that the net amount of sediment causes a change in the bed level of the channel between sections ***1*** and ***2***, which is given by $\frac{\partial}{\partial t}\left[Z\,B\,\Delta x(1-\lambda)\right]$ where ***Z*** is the elevation of the bed above a datum and ***λ*** is the porosity of the bed. It has been assumed that the amount of sediment carried in suspension does not change appreciably with time.

Combining the above derivations, it may be shown that the unsteady continuity equation of sediment conservation in a channel is given as

$$\frac{\partial Z}{\partial t} + \frac{1}{B\gamma_S(1-\lambda)}\frac{\partial Q_T}{\partial x} = 0 \qquad (17)$$

This equation states that the bed is lowered if the transport increases along the length and vice versa. It can be seen that when the flow is steady, the bed level cannot change with respect to time.

In equation (17), an estimate for $\boldsymbol{Q_T}$ may be obtained using equations (5) or (8), which however, contain the flow variables ***h*** (depth of flow) and ***V*** (average velocity). Hence, equation (17) cannot be solved independently, but has to be considered together with the equations of continuity and of motion for water, as given below:

Equation of flow continuity

$$B\frac{\partial h}{\partial t} + \frac{\partial Q}{\partial x} = q_l \qquad (18)$$

Where

- ***B*** is the channel width
- ***H*** is the water depth
- ***Q*** is the discharge = *A* ·• *V*, where A is the cross sectional area and **V** is the average velocity at a section
- $\boldsymbol{q_l}$ is any discharge that is entering the channel from its sides
- ***t*** and ***x*** are the dimensions in time and space

Equation of flow motion:

$$\frac{\partial V}{\partial t} + g\frac{\partial}{\partial x}\left(\frac{V^2}{2g} + h\right) = g\,(S_0 - S_f) \qquad (19)$$

Where

$\boldsymbol{g}$ is the acceleration due to gravity

S_0 is the riverbed slope

S_f is the friction slope

It may be noted that the friction slope, $\boldsymbol{S_f}$, in equation (19) may be evaluated using the Manning's equation ($S_f = \frac{V^2 n^2}{R^{4/3}}$) or one of the formulae from which Section 2.10.5 which gives specific resistance formulae for movable bed rivers and channels.

Solution of equations (17), (18) and (19) together with appropriate boundary conditions like the given discharge on the upstream of the channel (which could be a time dependent variable) and the water depth at the downstream end would provide a solution for the variables ***Z*** (the riverbed elevation), ***h*** (water depth) and ***v*** (average velocity) at each point of the channel. It may be noted that analytical solution of these simultaneous partial differential equations is possible only under very simplified and ideal conditions. In practice, however, a numerical solution has to be adopted which includes methods like that of ***finite difference***, ***finite element***, or ***finite volume***. Interested readers may refer to Chaudhry (1993) for extensive treatment on numerical methods in open channel hydraulics.

It may be mentioned that around the world, many research workers have tried to develop computer codes for solving the equations and a comprehensive list of such mathematical models may be found in Garde and Ranga Raju (2000). However, only a few of these are public domain model, which means they are freely accessible to the general public. One of these public domain software is the HEC-6 model produced by the United States Army Corps of Engineers (USACE) and may be downloaded from the following website.

www.hec.usace.army.mil/software/legacysoftware/hec6/hec6-download.htm

Equations (17), (18) and (19) when solved together for appropriate boundary conditions may be used to simulate one dimensional flow and sediment transport problems like that of sedimentation behind reservoirs and bed level lowering in the reaches downstream of a dam. Generally, for two dimensional flows and sediment movement, like that encountered due to scour near bridge piers or sediment movement upstream of barrages and water headworks, a two dimensional flow model has to be adopted. Since flow fields are, strictly speaking, three dimensional, the most appropriate model for simulation of such phenomenon would be a three dimensional model. Prof. N.R.B Olsen – of the Norwegian Technical University has prepared such software which is free for public use and may be downloaded from the following website.

http://folk.ntnu.no/nilsol/

2.10.10 Bed level changes during floods in alluvial rivers

Water resources engineers need to know the behaviour of alluvial rivers during floods for planning of some of their structures in natural rivers. It is true that their effect on riverbed may not be obvious immediately. In fact, it is not only the water discharge in the river that causes the bed level to change, but also the amount of sediment conveyed by the river which is generally increased many times during floods compared to the normal.

In many alluvial streams, the stream bed has been observed to rise during floods, while the bed is lowered after the flood recedes. On the other hand, there are other streams where the bed level has been found to be lowered during rising floods and aggraded (that is, raised) during the receding flood. It has been observed that where a river width is narrower than usual, a rise in flood usually causes a lowering of the river bed and conversely, for wider sections of rivers, there is mostly riverbed rising during floods.

It is obvious that the variation of the river bed during floods is dependent on the difference between sediment supply into the reach and the sediment transporting capacity of the reach.

2.10.11 References

1. Einstein, H A (1942) "Formolas for transportation of bed load", Trans. ASCE, Volume 107.
2. Garde, R J and Ranga Raju, K G (2000) Mechanics of Sediment Transport, New age International Publishers.
3. Yang, C T (1973) "Incpient motion and sediment transport", JHD, Proc. ASCE, Volume 99.
4. Ackers, P and White, W R (1973) "Sediment transport: New approach and analysis", J Hydraulic Divn., ASCE, Volume 99.
5. Engelund, F and Hansen, E (1972) A Monograph for Sediment Transport and Alluvial Streams, Teknisk Forlag, Copenhagen.
6. Brownlie, W R (1981) Prediction of Flow Depth and Sediment Discharge in Open Channels, Report number KH-R-43A, W.M. Keck Laboratory, Caltech, USA.
7. Karim, M F and Kennedy, J F (1990) "Menu of coupled velocity and sediment –discharge relations for rivers", J Hydraulic Engineering, ASCE, Volume 116.
8. Keulegan, G H (1938) "Laws of turbulent flows in open channels", US Department of Commerce.
9. Yang, C T (1996) Sediment Transport: Theory and Practice, McGraw Hill International Edition.
10. van-Rijn, L C (1984) "Sediment transport, Part II: Suspended load transport", J Hydraulic Engineering, ASCE, Volume 110.
11. Chaudhry, M H (1993) Open channel hydraulics, Prentice Hall of India.
12. Ranga Raju (1993) Flow through Open Channels, Second Edition, Tata McGraw Hill.

Module
3
Irrigation Engineering Principles

Lesson
1
India's Irrigation Needs and Strategies for Development

Instructional Objectives

On completion of this lesson, the student shall come to know about the following:

1. How can the development in irrigation be sustainable
2. What are the basic requirements of irrigation and how can they be managed
3. What are irrigation schemes and how are they classified
4. What has the nation planned about irrigation
5. What are Command Area development and participatory Irrigation Management
6. How can water be managed for irrigation
7. What should be the rationale for pricing irrigation water
8. How has irrigation progressed through ages in India

3.1.0 Introduction

It is known fact that India has a very large population and different studies show that it will continue to rise at least till 2050 A.D. The United Nations publication "Sustaining Water - An Update", published in 1994 indicates that the population of India by that time could vary between 1349 million (low projection) and 1980 million (high projection), with a medium projected figure as 1640 million. At present the country's food grain availability is around 523 grams per capita per day (though it varies significantly with the economic level). In China and USA, the corresponding figures are 980 grams and 2850 grams respectively. Assuming a population figure of 1800 million by 2050A.D, and a small rise in per capita consumption of food grain at about 650 grams, the annual requirement of the country would be around 430 million tonnes. The present productivity of irrigated land is about 2.5 tonnes/hectare and les than 0.5 tonnes/hectare for rainfed lands. Assuming that these levels can go up to 3.5 and 1.0 T/hr respectively by 2050 AD (which should be the urgent needs that has to be addressed to by the water resource engineer), it is imperative that the irrigation potential of at least 130 million hectare is created for food crop alone and 160 million hectare for all crops to be able to meet the demands of the country by 2050 AD.

Since the land and water resource of any country is not going to change much over the years, the water resource planners have to make cautious decisions on optimizing the available resources for maximum benefit. In the next section, we look into the options available and the decisions that may be taken.

3.1.1 Sustainable development in Irrigation

For the survival of the country, there is an urgent need to implement and plan irrigation strategies for now, and in future, as the population continues to grow. But that should not be at the cost of degradation of the present available resources of land and water, which means the natural resources that we have, should more or less remain the same after 50 or 100 years and beyond. This concept is termed "***sustainable***", which was not much of a problem earlier when compared to the resources the demand was less. But now it is reversed and for devising any planning strategy the constraints have to be kept in mind. As an example, the utilization of ground water may be cited. In many regions of India, there has been alarming withdrawal of ground water for meeting demands of irrigation and drinking water demand than that which can be naturally recharged. This has led to rise of further problems like arsenic and fluoride contamination. Since ground water recharge by natural means takes a long time, perhaps years and even decades, there is little hope of regaining the depleted table near future.

Next we look into the options available to the irrigation engineer for development of irrigation facilities within the constraints.

3.1.2 Future directions

The question that is to be resolved is this: Are we capable of producing the required amount of food grain for the country? Apart from spreading the network of irrigation system further into the country, there is an urgent need on research- for better seeds, better water management and distribution practices, low cost fertilizer etc., Nevertheless, the possible options available to water resource engineer to meet the future irrigation and food requirement may, therefore, include evaporation control and reduction and losses during conveyance of water through channels, recycling of water, inter basin transfer, desalination of sea-water in coastal areas, rainfall by cloud seeding, improved technology, etc. Loss of top soil due to erosion in one of the forms of degradation which can be contained on a limited scale but problems of salinity, alkalinity, water logging, etc. reduce the productivity. The future lies in considering and bringing under cultivation additional area and considering intensified production on existing good agricultural land and water resource system optimization.

3.1.3 Constraints of land and water

The total geographical area of land in India is about 329 million hectare (M-ha), which is 2.45% of the global land area. The total arable land, according to an estimate made by the Food and Agricultural Organization (FAO), made available through the web-site ***Aquastat***, is about 165.3 M-ha which is about 50.2 % of the total geographical area (the average world figure is 10.2%). The web-site that may be checked for this and relevant data is that of FAO:

http://www.fao.org/ag/agl/aglw/aquastat/countries/india/index.stm

India possesses 4% of the total average annual run off in the rivers of the world. The per capita water availability of natural run off is at least 1100 cubic meters/yr. The utilizable surface water potential of India has been estimated to be 1869 cubic kms.but the amount of water that can actually be to put to beneficial use is much less due to severe limitations imposed by physiographic, topographic, interstate issues and the present technology to harness water resources economically. The recent estimates made by the Central Water Commission indicate that the water resources utilizable through surface structures is about 690 cubic kms only (about 36% of the total ground water is another important source of water).

Ground water is another important source of water. The quantum of water that can be extracted economically from the ground water aquifers every year is generally known as ground water potential. The preliminary estimates made by the Central Ground Water Board indicate that the utilizable ground water is about 432 cubic km.thus the total utilizable water resource is estimated to 1122 cubic km. It must be remembered that this amount of water is unequally spread over the entire length and breadth of the country.

Of the total 329 M-ha of land, it is estimated that only 266 M-ha possess potential for production. Of this, 143 M-ha is agricultural land. It is estimated that 85 M-ha of land suffers from varying degrees of soil degradation. Of the remaining 123 M-ha, 40 M-ha is completely unproductive. The balance 83 M-ha is classified as forest land, of which over half is denuded to various degrees.

It is alarming to note that the percapita availability of land is half of what it used to be same 35 years ago. This would further reduce as our country's population continuous to grow. At present 141 M-ha of land is being used for cultivation purposes. Between 1970-71 and 1987-88 the average net sown area has been 140.4 M-ha. The need for production of food, fodder, fibre, fuel in the crop growing areas have to compete with the growing space require for urbanization. The factors of land degradation, like water logging, salinity, alkalinity and erosion of soils on account of inadequate planning and inefficient management of water resources projects will severely constrain the growth of net sown area in the future.

3.1.4 Benefits of irrigation

With the introduction of irrigation, there have been many advantages, as compared to the total dependence on rainfall. These may be enumerated as under:

1. ***Increase in crop yield***: the production of almost all types of crops can be increased by providing the right amount of later at the right time, depending on its shape of growth. Such a controlled supply of water is possible only through irrigation.

2. ***Protection from famine***: the availability of irrigation facilities in any region ensures protection against failure of crops or famine due to drought. In regions without irrigation, farmers have to depend only on rains for growing crops and since the rains may not provide enough rainfall required for crop growing every year, the farmers are always faced with a risk.

3. ***Cultivation of superior crops***: with assured supply of water for irrigation, farmers may think of cultivating superior variety of crops or even other crops which yield high return. Production of these crops in rain-fed areas is not possible because even with the slight unavailability of timely water, these crops would die and all the money invested would be wasted.

4. ***Elimination of mixed cropping***: in rain-fed areas, farmers have a tendency to cultivate more than one type of crop in the same field such that even if one dies without the required amount of water, at least he would get the yield of the other. However, this reduces the overall production of the field. With assured water by irrigation, the farmer would go for only a single variety of crop in one field at anytime, which would increase the yield.

5. ***Economic development***: with assured irrigation, the farmers get higher returns by way of crop production throughout the year, the government in turn, benefits from the tax collected from the farmers in base of the irrigation facilities extended.

6. ***Hydro power generation***: usually, in canal system of irrigation, there are drops or differences in elevation of canal bed level at certain places. Although the drop may not be very high, this difference in elevation can be used successfully to generate electricity. Such small hydro electric generation projects, using ***bulb-turbines*** have been established in many canals, like Ganga canal, Sarada canal, Yamuna canal etc.

7. ***Domestic and industrial water supply***: some water from the irrigation canals may be utilized for domestic and industrial water supply for nearby areas. Compared to the irrigation water need, the water requirement for domestic and industrial uses is rather small and does not affect the total flow much. For example, the town of Siliguri in the Darjeeling district of West Bengal, supplies its residents with the water from Teesta Mahananda link canal.

3.1.5 Classification of irrigation schemes

Irrigation projects in India are classified into three categories –major medium & minor according to the area cultivated the classification criteria is as follows:-

1) Major irrigation projects: projects which have a ***culturable command area*** (CCA) of more than 10,000 ha but more than 2,000 ha utilize mostly surface water resources.

2) Medium irrigation projects: projects which have CCA less than 10,000 ha. But more than 2,000 ha utilizes mostly surface water resources.

3) Minor irrigation projects: projects with CCA less than or equal to 2,000 ha. utilizes both ground water and local surface water resources. Ground water

development is primarily done through individual and cooperative effort of farmers with the help of institutional finance and their own savings. Surface water minor irrigation schemes are generally funded from the public sector only. The ultimate irrigation potential from minor irrigation schemes have been assessed as 75.84 million ha of which partly would be ground water based (58.46 million ha) and covers about two thirds. By the end of the ninth plan, the total potential created by minor irrigation was 60.41 million ha.

The ultimate irrigation potential of the country from major and medium irrigation projects has been assessed as about 64 M-ha. By the end of the ninth plan period, the total potential created from major and medium projects was about 35 M-ha.

3.1.6 Major and medium irrigation projects vis-à-vis minor irrigation projects

While formulating strategies for irrigation development the water resources planner should realize the benefits of each type of project based on the local conditions. For example, it may not always be possible to benefit remote areas using major/medium projects. At these places minor irrigation schemes would be most suitable. Further, land holding may be divided in such a way that minor irrigation becomes inevitable. However, major and medium projects wherever possible is to be constructed to reduce the overall cost of development of irrigation potential.

According to the third minor irrigation census carried out in 2000-01, there are about 5.56 lakh ***tanks*** in the country, with the most occurring in the following states

1. West Bengal: 21.2 percent of all the tanks in the country
2. Andhra Pradesh: 13.6
3. Maharashtra: 12.5
4. Chhattisgarh: 7.7
5. Madhya Pradesh: 7.2
6. Tamilnadu: 7.0
7. Karnataka: 5.0

This data has been gathered from the web-site of the Ministry of Water Resources, Government of India: http://www.wrmin.nic.in/.

Due to non use of these 15 percent tanks nearly 1 M-ha of Irrigation potential is lost. Another, around 2 M-ha of potential is lost due to under utilisation of tanks in use. Loss of potential due to non use is more pronounced in Meghalaya, Rajasthan and Arunachal Pradesh (above 30%), whereas loss of potential due to under utilisation is more than 50 percent in case of Gujarat, Nagaland, Rajasthan, A&N Island and Dadar and Nagar Haveli.

It also appears that the maintenance of the tanks has been neglected in many parts of the country and their capacity has been reduced due to siltation. It has been estimated that about 1.7 M-ha of net area has been lost under tank irrigation due to drying up of tanks and encroachment of foreshore area. Some advantages of minor irrigation should also be kept in mind. These are: small investments, simpler components, labour intensive, quick maturing and most importantly it is farmer friendly.

On the other hand, it is seen that of the assessed 64 M-ha of irrigation potential that may be created through major and medium projects, only about 35 M-ha have so far been created. Hence a lot of scope for development in this sector is remaining. These may be realized through comprehensive schemes including storage, diversion and distribution structures. Some of these schemes could even be multi-purpose thus serving other aspects like flood control and hydro power.

3.1.7 Outlook of the national water policy

Our country had adapted a national water policy in the year 1987 which was revised in 2002. The policy document lays down the fact that planning and development of water resources should be governed by the national perspective. Here we quote the aspects related to irrigation from the policy.

1. Irrigation planning either in an individual project or in a watershed as a whole should take into account the irrigability of land, cost-effective irrigation options possible from all available sources of water and appropriate irrigation techniques for optimizing water use efficiency. Irrigation intensity should be such as to extend the benefits of irrigation to as large a number of farm families as possible, keeping in view the need to maximize production.
2. There should be a close integration of water use and land use policies.

3. Water allocation in an irrigation system should be done with due regard to equity and social justice. Disparities in the availability of water between head-reach and tail end farms and between large and small farms should be obviated by adoption of a rotational water distribution system and supply on a volumetric basis subject to certain ceilings and rational pricing.

4. Concerted efforts should be made to ensure that the irrigation potential created is fully utilised. For this purpose, the command area development approach should be adopted in all irrigation projects.

5. Irrigation being the largest consumer of fresh water, the aim should be to get optimal productivity per unit of water. Scientific management farm practices and sprinkler and drip system of irrigation should be adopted wherever feasible.

6. Reclamation of water-logged/saline affected land by scientific and cost effective methods should form a part of command area development programme.

3.1.8 Command Area Development Programme (CADP)

This scheme, sponsored by the central government was launched in 1974-75 with the objective of bridging the gap between irrigation potential created and that utilized for ensuring efficient utilization of created irrigation potential and increasing the agricultural productivity from irrigated lands on a sustainable basis. The programme envisages integrating various activities relating to irrigated agriculture through a multi-disciplinary team under an area development authority in a coordinated manner. The existing components of the CADP are as follows:-

1. On farm development works, that is, development of field channels and field drains within the command of each ***outlet***, land leveling on an outlet command basis; reclamation of ***water logged*** areas; enforcement of a proper system of rotational water supply (like the ***warabandi***) and fair distribution of water to individual fields; realignment of field boundaries, wherever necessary (where possible, consolidation of holding are also combined)supply of all inputs and service including credit; strengthening of extension services; and encouraging farmers for participatory irrigation management.

2. Selection and introduction of suitable cropping patterns.

3. Development of ground water to supplement surface irrigation (conjunctive use under minor irrigation sector)

4. Development and maintenance of the main and intermediate drainage system.

5. Modernization, maintenance and efficient operation of the irrigation system up to the outlet of one cusec ($1ft^3/sec$) capacity.

For an overall appreciation of an entire irrigation project it is essential that the objectives of the CAD be kept in mind by the water resources engineer.

3.1.9 Participatory irrigation management (PIM)

Any irrigation project cannot be successful unless it is linked to the stakeholders, that is, the farmers themselves. In fact, people's participation in renovation and maintenance of field channels was the established practice during the pre independence days. However, the bureaucracy encroached on this function in the post independence period and a realization has dawned that without the participation of farmers, the full potential of an irrigation scheme may not be realized. Though a water resources engineer is not directly involved in such a scheme, it is nevertheless wise to appreciate the motive behind PIM and keep that in mind while designing an irrigation system.

The national water policy stresses the participatory approach in water resources management. It has been recognized that participation of the beneficiaries would help greatly for the optimal upkeep of irrigation system and utilization of irrigation water. The participation of farmers in the management of irrigation would give responsibility for operation and maintenance and collection of water rates from the areas under the jurisdiction of the water user's association of concerned hydraulic level. Under the command area development programme (CADP), presently a provision exists for a one-time functional grant to farmer's associations at the rate of Rs. 500 per hectare of which Rs. 225 per hectare is provided by the central government and state government each, and Rs. 50 per hectare is contributed by the farmer's associations.

It may be mentioned that so far, that is by year 2004, the state governments of Andhra Pradesh, Goa, Karnataka, Tamilnadu, Rajasthan and Madhya Pradesh have enacted legislations for the establishment of the water user's associations.

The sustainability and success of PIM depends on mutual accountability between the water user's association and the irrigation department of the concerned state, attitudinal change in the bureaucracy, autonomy for the water user's associations, multifunctional nature of the water user's association and the choice of appropriate model for PIM with appropriate legal and institutional framework. If the farmers have to take over and manage the system, then the system must be rectified by the irrigation department to a minimum standard to carry the design discharge before it is handed over to the water user's association. The success of the PIM is also linked to the introduction of rotational water supply and water charges with rationalized establishment costs. Unlined field channels need to be manually constructed in a 'V' shape which is considered stable and efficient for carrying water.

3.1.10 Management of water for irrigation

Of the two resources –land and water, management of the former is largely in the domain of agricultural engineers. Management of water, on the other hand, is mostly the purview of the water resources engineer who has to decide the following:

- How much water is available at a point of a surface water source, like a river (based on hydrological studies)

- How much ground water is available for utilization in irrigation system without adversely lowering the ground water table?

- For the surface water source, is there a need for construction of a reservoir for storing the monsoon runoff to be used in the lean seasons?

- What kind of diversion system can be constructed across the river for diverting part of the river flow into a system of canal network for irrigating the fields?

- How efficient a canal network system may be designed such that there is minimum loss of water and maximum agricultural production?

- How can excess water of an irrigated agricultural fields be removed which would otherwise cause water logging of the fields?

In order to find proper solution to these and other related issues, the water resources engineer should be aware of a number of components essential for proper management of water in an irrigation system. These are:-

1. ***Watershed development:*** since the water flowing into a river is from a watershed, it is essential that the movement of water over ground has to be delayed. This would ensure that the rain water falling within the catchment recharges the ground water, which in turn replenishes the water inflow to the reservoir even during the lean season. Small check dams constructed across small streams within the catchment can help to delay the surface water movement in the watershed and recharge the ground water. Measures for the water shed development also includes aforestation within the catchment area which is helpful in preventing the valuable top-soil from getting eroded and thus is helpful also in preventing siltation of reservoirs. Other soil conservation methods like regrassing and grass land cultivation process, galley plugging, nullah bunding, contour bunding etc. also come under watershed development.

2. ***Water management:*** surface water reservoirs are common in irrigation systems and these are designed and operated to cater to crop water requirement throughout the year. It is essential, therefore, to check loss of water in reservoir due to
    - Evaporation from the water surface
    - Seepage from the base
    - Reduction of storage capacity due to sedimentation

3. ***Water management in conveyance system:*** In India the water loss due to evaporation, seepage and mismanagement in the conveyance channels (for canals and its distributaries) is exceptionally high-nearly 60%. Some countries like Israel have reduced this loss tremendously by taking several measures like lining of water courses, lining not only reduces seepage, but also minimizes weed infestation and reduces overall maintenance cost though the initial cost of providing lining could be high depending on the material selected.

4. ***On farm water management:*** Though this work essentially is tackled by agricultural engineers, the water resources engineers must also be aware of the problem so that a proper integrated management strategy for conveyance-delivery-distribution of irrigation water is achieved. It has been observed from field that the water delivered from the canal system to the agricultural fields are utilized better in the head reaches and by the time it reaches the tail end, its quantity reduces. Often, there are land holding belonging to different farmers along the route of the water course and there is a tendency of excess withdrawal by the farmers at the upper reaches. In order to

tackle this kind of mismanagement a proper water distribution roster has to be implemented with the help of farmers' cooperatives or water user's associations. At times farmers are of the opinion that more the water applied more would be the crop production which is generally not true beyond a certain optimum water application rate. Education of farmers in this regard would also ensure better on-farm water management.

5. ***Choice of irrigation method:*** Though irrigation has been practiced in India from about the time of the Harappa civilization, scientific irrigation based on time variant crop water need within the constraints of water and land availability is rather recent. It is important to select the right kind of irrigation method to suit the particular crop and soil. For example, following is a short list of available methods corresponding to the kind of crop.

Method of irrigation	Suitable for crops
Border strip method	Wheat, Leafy vegetables, Fodders
Furrow method	Cotton, Sugarcane, Potatoes
Basin method	Orchard trees

Other methods like sprinkler and drip irrigation systems are adapted where water is scarce and priority for its conservation is more than the consideration for cost. Although most advanced countries are adopting these measures, they have not picked up as much in India mainly due to financial constraints. However, as time passes and land and water resources get scarce, it would be essential to adopt these practices in India, too.

6. ***Choice of cropping pattern:*** Scientific choice of cropping pattern should be evolved on the basis of water availability, soil type, and regional agro-climate conditions. Crop varieties which give equivalent yield with less water requirement and take less time to mature should be popularized. Scientific contribution in the form of double or multiple cropping can be achieved if the sowing of crops such as paddy, groundnut, arhar etc. is advanced, if necessary, by raising the nurseries with the help of ground-water. Selection of crops planting sequence per unit weight of water applied.

7. ***Scheduling of irrigation water:*** Traditional farmers engaged in crop production are aware of some kind of scheduling of water to the crops, but their knowledge is based mostly on intuition and traditional wisdom rather than on any scientific basis. Modern scientific study on crop growth has shown that a correlation can be established between the climatic parameters, crop water requirement and the moisture stored in the soil especially in the root zone. It has now been established by scientific

research that the application of irrigation should be such that the available water in the soil above the ***permanent wilting point*** is fully utilized by the crop before requiring application of water to replenish the depleted moisture in the soil. Since any canal would be delivering water at the same time to different fields growing different crops, the demand of the various fields have to be calculated at any point of time or a certain period of time(days, weeks),and the water distributed accordingly through the canal network.

8. ***Development of land drainage:*** Due to improper application of water and inadequate facilities for drainage of excess water from irrigated lands, large tracks of land near irrigated areas have been affected with water logging and excess salt concentration in soils. Adequate drainage measures like surface and subsurface drainage systems, vertical drainage, bio-drainage etc. should be developed as an integral part of the irrigation system.

9. ***Command area development:*** We have already seen that the government has initiated the command area development programme (CADP) which would ensure efficient water utilization and integrated area developments in the irrigation command.

10. ***Canal automation:*** At present, the water entering the canal network through the ***headworks*** as well as the water getting distributed into the various branches and finally reaching the fields through the outlets are controlled manually. However, if these operations are carried out through automated electro-mechanical systems which can communicate to a central computer, then the whole process can be made more efficient. This would also help to save water and provide optimal utilization of the availability water.

3.1.11 Classification of irrigation schemes

The classification of the irrigation systems can also be based on the way the water is applied to the agricultural land as:

1. ***Flow irrigation system*:** where the irrigation water is conveyed by growing to the irrigated land. This may again be classified into the following.

   - **Direct irrigation*:*** Where the irrigation water is obtained directly from the river, without any intermediate storage. This type of irrigation is possible by constructing a weir or a barrage across a river to raise the level of the river water and thus divert some portion of the river flow through an adjacent canal, where the flow takes place by gravity.

   - **Reservoir/tank/storage irrigation:** The irrigation water is obtained from a river, where storage has been created by construction an obstruction across the river, like a dam. This ensures that even when there is no inflow into the river from the catchment, there is enough stored water which can continue to irrigate fields through a system of canals.

2. ***Lift irrigation system:*** Where the irrigation water is available at a level lower than that of the land to be irrigated and hence the water is lifted up by pumps or by other mechanical devices for lifting water and conveyed to the agricultural land through channels flowing under gravity.

Classification of irrigation systems may also be made on the basis of duration of the applied water, like:

1. **Inundation/flooding type irrigation system:** In which large quantities of water flowing in a river during floods is allowed to inundate the land to be cultivated, thereby saturating the soil. The excess water is then drained off and the land is used for cultivation. This type of irrigation uses the flood water of rivers and therefore is limited to a certain time of the year. It is also common in the areas near river deltas, where the slope of the river and land is small. Unfortunately, many of the rivers, which were earlier used for flood inundation along their banks, have been embanked in the past century and thus this practice of irrigation has dwindled.

2. **Perennial irrigation system:** In which irrigation water is supplied according to the crop water requirement at regular intervals, throughout the life cycle of the crop. The water for such irrigation may be obtained from rivers or from walls. Hence, the source may be either surface or ground water and the application of water may be by flow or lift irrigation systems.

3.1.12 Pricing of water

This is more of a management issue than a technical one. After all, the water being supplied for irrigation has a production cost (including operation cost and maintenance cost) which has to be met from either the beneficiary or as subsidy from the government. Since water is a state subject (as the matter included in entry 17 of list 11 that is, the state list of the Constitution of India), every state independently fixes the rates of water that it charges from the beneficiaries, the remaining being provided from state exchequer.

There are wide variations in water rate structures across states and the rate per unit volume of water consumed varies greatly with the crop being produced. The rates charged in some states for irrigation vary even for different projects depending on the mode of irrigation. The rates, at present, also vary widely for the same crop in the same state depending on irrigation season, type of system, etc.

As such, right now, there is no uniform set principles in fixing the water rates and a wide variety of principles for pricing are followed for the different states, such as:

- recovery cost of water
- capacity of irrigators to pay based on gross earning or net benefit of irrigation
- water requirement of crops
- sources of water supply and assurance
- classification of land linked with land revenue system

In some states water cess, betterment levy, etc. are also charged. Hence, there is an urgent need of the water resources planners to work out a uniform principle of pricing irrigation water throughout the country, taking into account all the variables and constraints.

3.1.13 Procedure for setting up a major/medium irrigation project scheme in India

The central design organization of each state desiring to set up an irrigation project shall have to prepare a detailed project report of the proposed irrigation scheme based on the document "Guidelines for Submission, Appraisal and Clearance of Irrigation and Multipurpose Projects" brought out by the Central Water Commission. This report has to be sent to the project appraisal organization of the Central Water Commission for the clearance with a note certifying the following:

1. All necessary surveys and investigations for planning of the project and establishing its economic feasibility have been carried out according to the guidelines mentioned above.

2. 10% or 5000 ha (whichever is minimum) of the command area of the proposed project has been investigated in details in three patches of land representing terrain conditions in the command for estimation of the conveyance system up to the last farm outlets.

3. 10% of the canal structures have been investigated in full detail.

4. Detailed hydrological, geological, construction material investigations have been carried out for all major structures, that is, dams, weirs (or barrages, as the case may be), main canal, branch canal up to distributaries carrying a discharge of 10 cumecs.

5. Soil survey of the command area has been carried out as per IS 5510-1969.

6. Necessary designs for the various components of the project have been done in accordance with the guidelines and relevant Indian standards.

7. Necessary studies for utilization of ground water have been done with special regard to the problem of water logging and suitable provisions have been made for conjunctive use of ground water and drainage arrangements.

8. The cropping pattern has been adopted in consultation with the state agriculture department and is based on soil surveys of the command keeping in view the national policy in respect of encouraging crops for producing oil seeds and pulses.

9. The cost estimates and economic evaluations were carried out as per guidelines issued by the Central Water Commission.

It may be noted that similar report has to be made even for multipurpose projects having irrigation as a component. Apart from the above techno-economic studies carried out by the state central design organization, the project report should be examined by the state-level project appraisal/technical advisory committee comprising representatives of irrigation, agriculture, fisheries, forests, soil conservation, ground water, revenue and finance departments and state environmental management authority.

It may be noted that a project of the magnitude of a major or medium irrigation scheme has wide impacts. Hence, the techno-economic feasibility report should also be supplemented with “Environmental Impact Assessment Report” and “Relief and Rehabilitation Plan”. The latter touches the issue of the plans for appropriate compensation to the affected persons due to the construction of the projects.

The project proposal submitted to the Central Water Commission shall be circulated amongst the members of the advisory committee of the ministry of water resources for scrutiny. Once the project is found acceptable it shall be recommended for investment clearance to the planning commission and inclusion in the five year plan/annual plan.

3.1.14 Plan development

Since the commencement of the first five year plan in 1951, the developmental schemes of India have been planned in a systematic programme. Under this scheme, each state of the union of India has seen development in the field of irrigation development. A brief description of the development in each state so far, has been indicated in the following paragraphs.

Andhra Pradesh:

At the beginning of plan development there were about 56 major and medium projects in addition to several minor projects in the state. Irrigation potential of about 2.4 M-ha was created through these projects which was about one-fourth of the ultimate irrigation potential of the state. About one hundred major and medium projects were undertaken during plan development. These along with a large number of minor irrigation projects created an additional potential of about 4M-ha thus raising the irrigation potential created to about two-thirds of ultimate potential of 9.2 M-ha. This has resulted in bringing a net area of 4.3 M-ha under irrigation out of 11 M-ha of net sown area in the state. The important projects taken up in the last 50 years include large projects like

Nagarjuna Sagar, Sriram Sagar and Srisailam inaddition to projects like Tungabhadra high level canal, Somasila etc.

Arunachal Pradesh:

Situated in the hilly region of the North east, irrigation potential of the state is as low as 0.16 M-ha. There was hardly any irrigation projects have been undertaken which have created an irrigation potential of 82,000 ha, which accounts for about half of the ultimate potential. A net area of about 36,000 ha out of the net sown area of 149000 ha has been brought under irrigation by these projects.

Assam:

Before the commencement of plan development, irrigation development in the state was limited to minor projects utilizing surface and ground water which provided an irrigation potential of 0.23 M-ha under plan development about 20 major and medium irrigation projects and a large number of minor irrigation projects were undertaken resulting in creation of an irrigation potential of 0.85 M-ha which is about one-third of the ultimate irrigation potential 2.67 M-ha for the state. A net area of 0.57 M-ha has been brought under irrigation through these projects. Important taken up during the period include Bodikerai, Dhansiri, Koilong etc.

Bihar:

Irrigation development in the pre-plan period included four major and medium projects in addition to several minor irrigation projects. These projects accounted for an irrigation potential of 1.4 M-ha. About 105 major and medium irrigation projects in addition to a large number of minor irrigation projects were undertaken during plan development, resulting in creation of additional irrigational potential of about 7V. Two-third of the ultimate irrigation potential of 12.4 M-ha for the state has thus been created so far. An area of 3.3 M-ha out of net sown area of 7.7 M-ha in the state has been brought under irrigation from these projects. Important projects undertaken during the plan period include Gandak, Kosi barrage and Eastern canal, Rajpur canal, Sone high level canal, Subernarekha, North Koel reservoir etc.

Goa:

Pre-plan irrigation development in Goa was limited to construction of a few minor projects. During plan development, four major and medium irrigation projects and a number of minor irrigation projects were undertaken. These projects have created an irrigation potential of 35000 ha, which accounts for about 43 percent of the ultimate irrigation potential of 82000 ha for the state. A net area of 23000 ha of the sown area of 131000 ha available to the state has been brought under irrigation. The inportant projects undertaken during the period include Salauli and Anjunem.

Gujarat:

Two major projects and a number of minor irrigation projects were undertaken in the state during the pre-plan period which created an irrigation potential of 0.46 M-ha.

About 130 major and medium irrigation projects were undertaken during plan development and these projects have created an irrigation potential of 3.4 M-ha Thus 74 percent of the ultimate irrigation potential of 4.75 M-ha for the state has been created. An area of 2.64 M-ha out of net sown area of 9.3 M-ha has been brought under irrigation under the above projects. Important projects taken up in the state include Ukai, Kakarpur, Mahi etc. The giant project of Sardar Sarovar is one of the projects presently ongoing in the state.

Harayana:

The major project of Western Yamuna canal was constructed in the state during pre-plan period. Irrigation potential created in the pre-plan period was about 0.72 M-ha. About 10 major projects were taken up during the plan period in addition to a number of tubewells and other minor irrigation projects. The total irrigation potential created by projects so far undertaken amounts to 3.7 M-ha which accounts for about 80 percent of the ultimate irrigation potential for the state. Out of the net sown area of 3.6 M-ha, an area of 2.6 M-ha has been brought under irrigation. Important projects undertaken during the plan period include inter-state projects like Bhakra-Nangal, Sutlej-Yamuna link canal etc.

Himachal Pradesh:

Irrigation development in the hilly state of Himachal Pradesh was restricted to minor projects in the preplan period. During the plan development, 5 major and medium projects and a number of minor projects were undertaken in the state. Sofar, these projects have created a total irrigation potential of 0.16 M-ha and have brought an area of 99,000 ha under irrigation out of the net sown area of 5,83,000 ha in the state. Important projects undertaken in the State include Balh valley, Shahanahar etc.

Jammu & Kashmir:

Pre-plan irrigation development in the state included seven major and medium projects in addition to minor irrigation schemes. These projects accounted for creation of an irrigation potential of 0.33 M-ha. During the plan development, about 20 major and medium irrigation projects and a number of minor projects were undertaken in the state. With the addition of about 0.22 M-ha from these projects the irrigation potential, so far, created has risen to 0.55 M-ha which is about 70 percent of the ultimate irrigation potential of 0.8 M-ha Net area brought under irrigation is 0.3 M-ha Important projects undertaken during plan period include Kathua Canal, Ravi-Tawi Lift, Karwal etc.

Karnataka:

Pre-plan irrigation development in the state included 11 major and medium projects, in addition to a large number of minor projects. These projects created an irrigation potential of 0.75 M-ha. During plan period about 54 major and medium irrigation projects and a number of minor irrigation projects were undertaken. These projects have raised the irrigation potential created in the state to 0.32 M-ha which is about 70 percent of the ultimate irrigation potential of the state. Net area brought under irrigation is 2.1 M-ha Important projects undertaken during the plan period include Ghataprabha, Malaprabha, Tungabhadra, Upper Krishna stage-I, Kabini, Harangi, Hemavati etc.

Kerala:

Irrigation development in the state in the pre-plan period was limited to minor irrigation which had created a potential of 2,25,000 ha. During plan development, about 22 major and medium irrigation projects were undertaken which have raised the irrigation potential created to about 1.2 M-ha thus achieving 70 percent of the ultimate potential of 2.1 M-ha for the state. Net area brought under irrigation is 0.33 M-ha. Important projects taken up during plan development include Malampuzha, Chalakudi, Periyar Valley, Kallada etc.

Madhya Pradesh:

A little over 1 M-ha of irrigation potential was created in the state in the pre-plan period through the construction of about 20 major and medium and a number of minor projects. During the period of plan development about 160 major and medium projects were undertaken along with minor irrigation schemes. With this, the irrigation potential created has gone upto over 5 M-ha which is about half of the ultimate potential of 10.2 M-ha. About 4.8 M-ha of land has been broght under irrigation through these projects. Important projects undertaken during the period include Chambal, Barna, Tawa, mahanadi Reservoir, Hasdeo-Bango, Bargi, Upper Wainganga etc.

Maharashtra:

Irrigation development in pre-plan period in Maharashtra was also over 1 M-ha achieved through the construction of about 21 major and medium irrigation projects along with a number of minor irrigation projects. During plan development, over 250 major and medium projects and a large number of minor projects were added which raised the irrigation potential created to about 4.9 M-ha , which is about two-third of the ultimate potential of 7.3 M-ha Out of the net sown area of 18 M-ha available to the state, 2.5 M-ha has been brought under irrigation. The important projects undertaken during the period of the plan development included Jayakwadi, Pench, Bhima, Mula, Purna, Khadakwasla, Upper Penganga etc.

Manipur:

An irrigation potential of about 5,000 M-ha was created in the state during pre-plan period through minor irrigation projects. During plan development, six major and medium projects and a number of minor irrigation projects were added which has raised the irrigation potential to 0.14 M-ha which is about 60 percent of the ultimate potential of 0.24 M-ha A net area of 65,000 ha has been brought under irrigation through these projects. One of the important irrigation projects taken up is the Loktak lift.

Meghalaya:

Minor irrigation projects are source of irrigation in this hilly state. A potential of 7,000 ha developed in the pre-plan period was enhanced to about 53,000 ha by taking up more minor irrigation projects. The state has little scope of taking up major and medium projects. A net area of about 45,000 ha has, so far, been provided with irrigation.

Mizoram:

There was hardly any irrigation development in the state in the pre-plan period. The terrain is not suitable for taking up major and medium projects. In the period of plan development, an irrigation potential of 13,000 ha was created through minor irrigation projects, thereby bringing a net area of about 8,000 ha under irrigation. The ultimate irrigation potential of the state is about 70,000 ha, which is one of the lowest of all states.

Nagaland:

Nagaland too has a low irrigation potential of about 90,000 ha out of which about 5,000 ha was created in the pre-plan period through minor irrigation schemes. This has been raised to about 68,000 ha (about three-fourth of the ultimate) by taking up minor irrigation projects. The net area provided with irrigation is of the order of 60,000 ha.

Orissa:

Pre-plan development in the state amounted to 0.46 M-ha through 5 major and medium and a large number of minor irrigation projects. During the period of plan development, 55 major and medium projects and a host of minor irrigation projects were undertaken in the state bringing up the irrigation potential created to about 3 M-ha which is more than half the ultimate irrigation potential of 5.9 M-ha for the state. A net area of 2 M-ha has been brought under irrigation through these projects. Important projects taken up during the project include Hirakud, Mahanadi Birupa Barrage, upper Kolab etc. Schemes like inter-state Subernarekha project and upper Indravati are presently in progress.

Punjab:

Punjab was one of the states where significant development in irrigation was made during pre-plan period. An irrigation potential of about 0.21 M-ha was created in the state through construction of 5 major and a number of minor lift irrigation projects. During the period of plan development 10 major and medium and a large number of minor projects were undertaken which has raised the irrigation potential of 6.5 M-ha. A net area of about 3.9 M-ha has been brought under irrigation in the state out of net sown area of 4.2 M-ha Important major and medium projects undertaken in the state include inter-state Bhakra Nangal,Sutlej-Yamuna Link Canal, Beas etc.

Rajasthan:

Out of the ultimate irrigation potential of 5.15 M-ha for the state, a potential of 1.5 M-ha was developed during the pre-plan period through the conctruction of one major and 42 medium irrigation projects in addition to several minor projects. About 67 major and medium projects and a large number of minor projects were undertaken during the plan period which has raised the irrigation potential created to about 4.8 M-ha which is more than 90 percent of ultimate irrigation potential of the state. Net area brought under irrigation is 3.9 M-ha out of net sown area of 16.4 M-ha Important projects undertaken during plan development include inter-state Chambal project, Bhakra Nangal, Beas, Indira Nahar, Mahi Bajaj Sagar etc.

Sikkim:

There was hardly any irrigation development in Sikkim at the time when plan development started in India. After Sikkim joined as a part of India, plan development was extended to the state in the seventies and an irrigation potential of about 25,000 ha is likely t be developed by the end of 1996-97 through minor irrigation projects.

Tamil Nadu:

The state of Tamil Nadu was one of the forerunners in development of irrigation during the British period. An irrigation potential projects of 2.4 M-ha was developed through 24 major and medium irrigation projects and a large number of minor irrigation projects during the pre=plan period. About 1.3 M-ha of irrigation potential was added during plan development through addition of 24 major and medium projects and a number of minor irrigation projects. Out of the ultimate irrigation potential of 3.9 M-ha, over 3.7 M-ha (about 95 percent) has so far been created. Out of the net sown of 5.6 M-ha, net area brought under irrigation is 2.7 M-ha Important projects undertaken during the peiod include Cauvery Delta, Lower Bhavani, Parambikulam Aliyar etc.

Tripura:

Irigation development in the pre-plan period in Tripura was mainly through minor irrigation. Potential of 10,000 ha was created during the pre-plan period. With the addition of 3 medium and several minor irrigation projects during plan development, the irrigation potential, so far, developed has risen to over 100,000 ha which is nearly half of the ultimate potential of 215,000 ha. About 16,000 ha of sown area have been brought under irrigation. One of the important projects taken up in the state is Gumti.

Uttar Pradesh:

With vast development in the Ganga valley an irrigation potential of 5.4 M-ha was created in the state during the pre-plan period, through 15 major schemes and a host of minor schemes. During plan development over 90 major and medium projects and a very large number of minor irrigation projects were undertaken in the plan period thereby raising the potential created to about 30 M-ha against the originally assessed ultimate potential of about 26 M-ha Three-fourth of this development (22.7 M-ha) is attributed to minor irrigation projects- largely ground water works. About 11.3 M-ha of area out of net sown area of 17.3 M-ha has been brought under irrigation through these projects. Important projects undertaken during the period include Ramganga, Sarda Sahayak, Saryu Nahar, Gandak, Madhya Ganga Canal, Tehri etc.

West Bengal:

Pre-plan irrigation development in West Bengal saw the implementation of major projects in addition to a large number of minor projects which resulted in creation of an irrigation potential of about 0.94 M-ha During plan period, about 13 major and medium and large number of minor schemes were added which have raised the irrigation potential of 6.1 M-ha A net area of 1.9 M-ha has been brought under irrigation through these projects out of net sown area of 5.3 M-ha Important projects undertaken during the period include DVC Barrage and canal system, Mayurakshi, Kangsabati etc.

Since the above information has been based on the available data from Central Water Commission, and Ministry of Water Resources, Government of India for the last decade, the smaller states carved out later have not been included and the data represents that for the undivided state.

3.1.15 History of Irrigation Development

(This section has been adapted from the information provided in the Ministry of Water Resources, Government of India web-site: http://wrmin.nic.in/).

Earliest evidences of irrigation

The history of irrigation development in India can be traced back to prehistoric times. Vedas and ancient Indian scriptures made reference to wells, canals, tanks and dams which were beneficial to the community and their efficient operation and maintenance was the responsibility of the State. Civilization flourished on the banks of the rivers and harnessed the water for sustenance of life. According to the ancient Indian writers, the digging of a tank or well was amongst the greatest of the meritorious act of a man. Brihaspathi, an ancient writer on law and politics, states that the construction and the repair of dams is a pious work and its burden should fall on the shoulders of rich men of the land. Vishnu Purana enjoins merit to a person who patronages repairs to well, gardens and dams.

In a monsoon climate and an agrarian economy like India, irrigation has played a major role in the production process. There is evidence of the practice of irrigation since the establishment of settled agriculture during the Indus Valley Civilization (2500 BC). These irrigation technologies were in the form of small and minor works, which could be operated by small households to irrigate small patches of land and did not require co-operative effort. Nearly all these irrigation technologies still exist in India with little technological change, and continue to be used by independent households for small holdings. The lack of evidence of large irrigation works at this time signifies the absence of large surplus that could be invested in bigger schemes or, in other words, the absence of rigid and unequal property rights. While village communities and co-operation in agriculture did exist as seen in well developed townships and economy, such co-operation in the large irrigation works was not needed, as these settlements were on the fertile and well irrigated Indus basin. The spread of agricultural settlements to less fertile and irrigated area led to co-operation in irrigation development and the emergence of larger irrigation works in the form of reservoirs and small canals. While the construction of small schemes was well within the capability of village communities, large irrigation works were to emerge only with the growth of states, empires and the intervention of the rulers. There used to emerge a close link between irrigation and the state. The king had at his disposal the power to mobilize labour which could be used for irrigation works.

In the south, perennial irrigation may have begun with construction of the Grand Anicut by the Cholas as early as second century to provide irrigation from the Cauvery river. Wherever the topography and terrain permitted, it was an old practice in the region to impound the surface drainage water in tanks or reservoirs by throwing across an earthen dam with a surplus weir, where necessary, to take off excess water, and a sluice at a suitable level to irrigate the land below. Some of the tanks got supplemental supply from stream and river channels. The entire land-scape in the central and southern India is studded with numerous irrigation tanks which have been traced back to many centuries before the beginning of the Christian era. In northern India also there are a number of small canals in the upper valleys of rivers which are very old.

Irrigation during Medieval Period

In the medieval India, rapid advances also took place in the construction of inundation canals. Water was blocked by constructing bunds across steams. This raised the water level and canals were constructed to take water to the fields. These bunds were built by both the state and private sources. Ghiyasuddin Tughluq (1220-250 is credited to be the first ruler who encouraged digging canals. However, it is Fruz Tughlug (1351-86) who inspired from central Asian experience, is considered to be the greatest canal builder before the nineteenth century. Irrigation is said to be one of the major reasons for the growth and expansion of the Vijayanagar empire in southern India in the fifteenth century. It may be noted that, but for exceptional cases, most of the canal irrigation prior to the arrival of the British was of the diversionary nature. The state, through the promotion of irrigation, had sought to enhance revenue and provide patronage through rewards of fertile land and other rights to different classes. Irrigation had also increased employment opportunities and helped in the generation of surplus for the maintenance of the army and the bureaucracy. As agricultural development was the pillar of the economy, irrigation systems were paid special attention to, as irrigation was seen to be a catalyst for enhanced agricultural production. This is demonstrated by the fact that all the large, powerful and stable empires paid attention to irrigation development. It may be said that the state in irrigation was commensurate with its own interest.

Irrigation development under British rule

Irrigation development under British rule began with the renovation, improvement and extension of existing works, like the ones mentioned above. When enough experience and confidence had been gained, the Government ventured on new major works, like the Upper Ganga Canal, the Upper Bari Doab Canal and Krishna and Godavari Delta Systems, which were all river-diversion works of considerable size. The period from 1836 to 1866 marked the investigation, development and completion of these four major works. In 1867, the Government adopted the practice of accepting works, which promised a minimum net return. Thereafter, a number of projects were taken up. These included major canal works like the Sirhind, the Lower Ganga, the Agra and the Mutha Canals, and the Periyar Dam and canals. Some other major canal projects were also completed on the Indus system during this period. These included the Lower Swat, the Lower Sohag and Para, the Lower Chenab and the Sidhnai Canals, ali of which went to Pakistan in 1947.

The recurrence of drought and famines during the second half of the nineteenth century necessitated the development of irrigation to give protection against the failure of crops and to reduce large scale expenditure on famine relief. As irrigation works in low rainfall tracts were not considered likely to meet the productivity test, they had to be financed from current revenues. Significant protective works were constructed during the period were the Betwa Canal, the Nira Left Bank Canal, the Gokak Canal, the Khaswad Tank and the Rushikulya Canal. Between the two types of works namely productive and protective, the former received greater attention from the Government. The gross area irrigated in British India by public works at the close of the nineteenth century was about 7.5 M-ha Of this, 4.5 M-ha came from minor works like tanks, inundation canals etc. for which no separate capital accounts were maintained. The area irrigated by protective works was only a little more than 0.12 M-ha

Irrigation development at the time of independence

The net irrigated area in the Indian sub continent, comprising the British Provinces and Princely States, at the time of independence was about 28.2 M-ha , the largest in any country of the world. The partition of the country, however, brought about sudden and drastic changes, resulting in the apportionment of the irrigated area between the two countries; net irrigated area in India and Pakistan being 19.4 and 8.8 M-ha respectively. Major canal systems, including the Sutlej and Indus systems fell to Pakistan's share. East Bengal, now Bangladesh, which comprises the fertile Ganga Brahmaputra delta region also went to Pakistan. The irrigation works which remained with India barring some of the old works in Uttar Pradesh and in the deltas of the South, were mostly of a protective nature, and meant more to ward off famine than to produce significant yields.

Plan development

In the initial phase of water resources development during the plan period after independence, rapid harnessing of water resources was the prime objective. Accordingly, the State Governments were encouraged to expeditiously formulate and develop water resources projects for specific purposes like irrigation, flood control, hydro-power generation, drinking water supply, industrial and various miscellaneous uses. As a result, a large number of projects comprising dams, barrages, hydro-power structures, canal net work etc. have come up all over the country in successive Five Year Plans. A milestone in water resources development in India is creation of a huge storage capability. Because of these created storage works it has now become possible to provide assured irrigation in the command area, to ensure supply for hydro-power and thermal power plants located at different places and to meet requirement for various other uses. Flood moderation could be effected in flood prone basins, where storage have been provided. Besides, supply of drinking water in remote places throughout the year has become possible in different parts of the country.

At the time of commencement of the First Five Year Plan in 1951, population of India was about 361 million and annual food grain production was 51 million tonnes which was not adequate. Import of food grains was then inevitable to cover up the shortage. Attaining self sufficiently in food was therefore given paramount importance in the plan period and in order to achieve the objective, various major, medium and minor irrigation and multi-purpose projects were formulated and implemented through successive Five Year Plans to create additional irrigation potential throughout the country. This drive compounded with green revolution in the agricultural sector, has enabled India to become marginally surplus country from a deficit one in food grains.

Thus the net irrigated area is 37 percent of net sown are and 29 percent of total cultivable area. As stated earlier, the ultimate potential due to major and medium projects has been assessed as 58 M-ha of which 60 per cent estimated to be developed.

Scenario of development of irrigation in the states during plan development is discussed in the following paragraphs.

Minor irrigation

While the development of irrigation is most essential for increasing food and other agricultural production to meet the needs of the growing population, development of Minor Irrigation should receive greater attention because of the several advantages they posses like small investments, simpler components as also being labour intensive, quick maturing and mot of all farmer friendly.

Minor Irrigation development programmes in the state is being implemented by many Departments/Organisations like Agriculture, Rural Development, Irrigation, Social Welfare etc At the central level also, different departments launch schemes having Minor Irrigation component.

The Ministry of Rural Areas and Employment launched a Million Wells Schemes (WMS) in 1988-89. Till 1997-98, a total of 12,63,090 wells have been constructed under MWS with an expenditure of Rs. 4,728.17 crore. The Ministry of Rural Areas and Employment is also implementing Drought Prone Area Programme(DPAP) on watershed basis.

The Ministry of Agriculture has been instrumental in providing credit to farmers for the development of Minor Irrigation through Commercial Banks, Regional Rural Banks, Co-operatives and National Bank for Agriculture and Rural Development (NABARD).

The Minor Irrigation Division of the Ministry of Water Resources monitors the progress of development of irrigation created through Minor Irrigation Project. It implements the Centrally Sponsored Scheme Rationalisation of Minor Irrigation Statistics (RMIS) and conducts census of Minor Irrigation structures on quinquennial basis with a view to create a reliable database for the development of Minor Irrigation Sector. It also assists the State Governments in preparation of schemes for posing to external funding agencies for attracting external assistance for Minor Irrigation Schemes.

Ground Water Development is primarily done through individual and co-operative efforts of the farmers with the help of institutional finance and their own savings. Surface Water minor Irrigation Scheme i.e. surface lift schemes and surface flow schemes are generally funded from the public sector outlay. NABARD provides finance to the banks for installation of Minor Irrigation works in the States. In addition, the Land Development Banks provide bank credit to the farmers under their normal programmes also. During the year 1997-98, the total credit disbursement for minor irrigation works out of Rs. 488.65 crores. Further, many old schemes go out of use due to one reason or the other. The Irrigation Potential created and utilised through ground water as well as surface water. Minor Irrigation Schemes are not being recorded systematically in most cases as there schemes are implemented and monitored by individual farmers. Further, the norms being adopted for assessing the irrigation potential of Minor Irrigation Schemes are also not uniform. All the Ground Water and Surface Water Schemes having Cultivable Command Area (CCA) up to 2,000 hectares are included in the Minor Irrigation Sector.

Reservoir storage

The storage position in 68 important reservoirs in different parts of the country is monitored by the Central Water Commission. Against the designed live capacity at full reservoir levels of 129.4 Th. Million Cubic meters (TMC) in these reservoirs the total live storage was 95.3 TMC at the end of September, 1999 and 106.3 TMC at the same point of time last year.

Irrigation potential

The reassessed Ultimate Irrigation Potential (UIP) is 139.89 million hectare (M-ha). This re-assessment has been done on the basis of the re-assessment of the potential of ground water from 40 M-ha to 64.05 M-ha and re-assessment of potential of surface minor irrigation from 15 M-ha to 17.38 M-ha Thus, there has been an increase of 26.39 M-ha in the UIP of the country, which was 113.5 M-ha Before re-assessment. At the inception of planning in India in 1951 the created irrigation potential was 22.60 M-ha The irrigation potential created upto the end of Eighth Plan has increased to 89.56 M-ha

Major and medium irrigation

The Ultimate Irrigation Potential of the country from Major and Medium Irrigation projects has been assessed as 58.5 m ha. This includes projects with a culturable command area of more than two thousand hectare. The potential created up to the end of the Seventh Plan (1985-90) was 29.92 M-ha and at the beginning of the Eighth Five Year Plan (1992-93) was 30.74 M-ha

A target 5.09 M-ha had been set for creation of additional potential during the Eighth Plan (1992-97) against which, the potential created was about 2.22 M-ha Thus, at the end of the Eighth Plan, the cumulative irrigation potential created from major and Medium irrigation was about 32.96 M-ha According to a provisional estimate, the irrigation potential through Major & Medium projects has reached the level of 34.5 M-ha By 1998-99.

Minor irrigation

All ground water and surface water schemes having culturable command area (CCA) upto 2000 ha individually are classified as minor irrigation schemes. Ground water development is primarily done through individual and cooperative efforts of the farmers with the help of institutional finance and their own savings. Surface water minor irrigation schemes are generally funded from the public sector outlay.

3.1.16 Some important terms

Culturable Command Area (CCA): The gross command area contains unfertile barren land, alkaline soil, local ponds, villages and other areas as habitation. These areas are called unculturable areas. The remaining area on which crops can be grown satisfactorily is known as cultivable command area (CCA). Culturable command area can further be divided into 2 categories

1. Culturable cultivated area: It is the area in which crop is grown at a particular time or crop season.
2. Culturable uncultivated area: It is the area in which crop is not sown in a particular season.

Gross command area (GCA): The total area lying between drainage boundaries which can be commanded or irrigated by a canal system.

G.C.A = C.C.A + UNCULTURABLE AREA

Water Tanks: These are dug areas of lands for storing excess rain water.

Outlet: This is a small structure which admits water from the distributing channel to a water course of field channel. Thus an outlet is a sort of head regulator for the field channel delivering water to the irrigation fields.

Water logged area: An agricultural land is said to be waterlogged when its productivity or fertility is affected by high water table. The depth of water-table at which it tends to make the soil water-logged and harmful to the growth and subsistence of plant life depends upon the height of capillary fringe, which is the height to which water will rise due to capillary action. The height of capillary fringe is more for fine grained soil and less for coarse grained ones.

Permanent wilting point: or the wilting coefficient is that water content at which plants can no longer extract sufficient water from the soil for its growth. A plant is considered to be permanently wilted when it will not regain its turbidity even after being placed in a saturated atmosphere where little or no consumptive water use occurs.

Module
3

Irrigation Engineering Principles

Lesson 2

Soil Water Plant Relationships

Instructional objectives

On completion of this lesson, the student shall learn:

1. The soil and water system that is needed for plant growth
2. Classification of soils with regards to agriculture
3. Classification of water held within soil pores
4. Soil – water constants and their significance
5. Watering interval for crops
6. Importance of water for plant growth

3.2.0 Introduction

Both soil and water are essential for plant growth. The soil provides a structural base to the plants and allows the root system (the foundation of the plant) to spread and get a strong hold. The pores of the soil within the ***root zone*** hold moisture which clings to the soil particles by surface tension in the driest state or may fill up the pores partially or fully saturating with it useful nutrients dissolved in water, essential for the growth of the plants. The roots of most plants also require oxygen for respiration. Hence, full saturation of the soil pores leads to restricted root growth for these plants. (There are exceptions, though, like the rice plant, in which the supply of oxygen to the roots is made from the leaves through aerenchyma cells which are continuous from the leaves to the roots).

Since irrigation practice is essentially, an adequate and timely supply of water to the plant root zone for optimum crop yield, the study of the inter relation ship between soil pores, its water-holding capacity and plant water absorption rate is fundamentally important. Though a study in detail would mostly be of importance to an agricultural scientist, in this lesson we discuss the essentials which are important to a water resources engineer contemplating the development of a command area through scientifically designed irrigation system.

3.2. 1 Soil-water system

Soil is a heterogeneous mass consisting of a three phase system of solid, liquid and gas. Mineral matter, consisting of sand, silt and clay and organic matter form the largest fraction of soil and serves as a framework (matrix) with numerous pores of various proportions. The void space within the solid particles is called the soil pore space. Decayed organic matter derived from the plant and animal remains are dispersed within the pore space. The soil air is totally expelled from soil when water is present in excess amount than can be stored.

On the other extreme, when the total soil is dry as in a hot region without any supply of water either naturally by rain or artificially by irrigation, the water molecules surround the soil particles as a thin film. In such a case, pressure lower than atmospheric thus results due to surface tension capillarity and it is not possible to drain out the water by gravity. The salts present in soil water further add to these forces by way of osmotic pressure. The roots of the plants in such a soil state need to exert at least an equal amount of force for extracting water from the soil mass for their growth.

In the following sections, we discuss certain important terms and concepts related to the soil-water relations. First, we start with a discussion on soil properties and types of soils.

3.2.2 Soil properties

Soil is a complex mass of mineral and organic particles. The important properties that classify soil according to its relevance to making crop production (which in turn affects the decision making process of irrigation engineering) are:

- Soil texture
- Soil structure

Soil texture:

This refers to the relative sizes of soil particles in a given soil. According to their sizes, soil particles are grouped into gravel, sand, silt and day. The relative proportions of sand, silt and clay is a soil mass determines the soil texture. Figure 1 presents the textural classification of 12 main classes as identified by the US department of agriculture, which is also followed by the soil survey organizations of India.

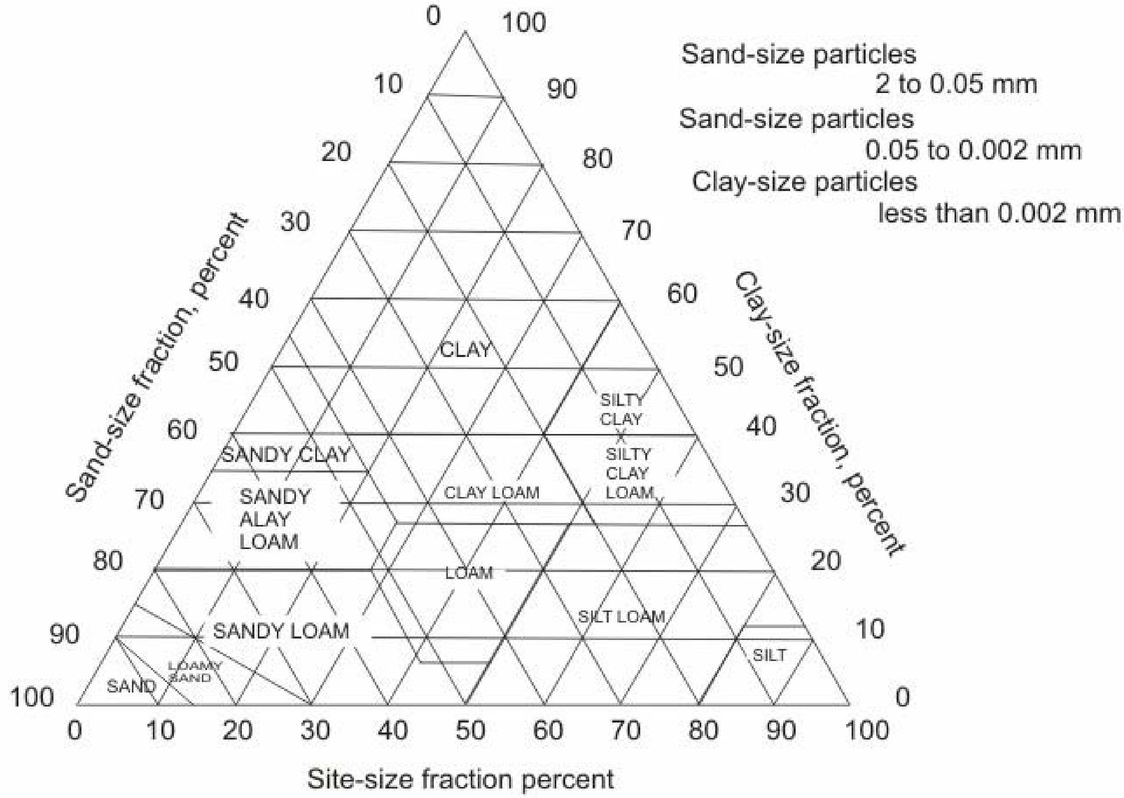


FIGURE 1. USDA textural classification chart

According to textural gradations a soil may be broadly classified as:

- Open or light textural soils: these are mainly coarse or sandy with low content of silt and clay.
- Medium textured soils: these contain sand, silt and clay in sizeable proportions, like loamy soil.
- Tight or heavy textured soils: these contain high proportion of clay.

Soil structure:

This refers to the arrangement of soil particles and aggregates with respect to each other. Aggregates are groups of individual soil particles adhering together. Soil structure is recognized as one of the most important properties of soil mass, since it influences aeration, permeability, water holding capacity, etc. The classification of soil structure is done according to three indicators as:-

- ***Type:*** there are four types of primary structures-platy, prism-like, block like and spheroidal.

- ***Class:*** there are five recognized classes in each of the primary types. These are very fine, fine, medium, coarse and very coarse.

- ***Grade:*** this represents the degree of aggradation that is the proportion between aggregate and unaggregated material that results when the aggregates are displaced or gently crushed. Grades are termed as structure less, weak, moderate, strong and very strong depending on the stability of the aggregates when disturbed.

3.2.3 Soil classification

Soils vary widely in their characteristics and properties. In order to establish the interrelation ship between their characteristics, they need to be classified. In India, the soils may be grouped into the following types:

- **Alluvial soils:** These soils are formed by successive deposition of silt transported by rivers during floods, in the flood plains and along the coastal belts. This group is by for the largest and most important soil group of India contributing the greatest share to its agricultural wealth. Though a great deal of variation exists in the type of alluvial soil available throughout India, the main features of the soils are derived from the deposition laid by the numerous tributaries of the Indus, the Ganges and the Brahmaputra river systems. These streams, draining the Himalayas, bring with them the products of weathering rocks constituting the mountains, in various degrees of fineness and deposit them as they traverse the plains. Alluvial soils textures vary from clayey loam to sandy loam. The water holding capacity of these soils is fairly good and is good for irrigation.

- **Black soils:** This type of soil has evolved from the weathering of rocks such as basalts, traps, granites and gneisses. Black soils are derived from the Deccan trap and are found in Maharashtra, western parts of Madhya Pradesh, parts of Andhra Pradesh, parts of Gujarat and some parts of Tamilnadu. These soils are heavy textured with the clay content varying from 40 to 60 percent.the soils possess high water holding capacity but are poor in drainage.

- **Red soils:** These soils are formed by the weathering of igneous and metamorphic rock comprising gneisses and schist's. They comprise of vast areas of Tamil nadu, Karnataka, Goa, Daman & Diu, south-eastern Maharashtra, Eastern Andhra Pradesh, Orissa and Jharkhand. They also are in the Birbhum district of West Bengal and Mirzapur, Jhansi and Hamirpur districts of Uttar pradesh. The red soils have low water holding capacity and hence well drained.

- **Laterites and Lateritic soils:** Laterite is a formation peculiar to India and some other tropical countries, with an intermittently moist climate. Laterite soils are derived from the weathering of the laterite rocks and are well developed on the summits of the hills of the Karnataka, Kerala, Madhya Pradesh, The eastern ghats of Orissa, Maharashtra, West Bengal, Tamilnadu and Assam. These soils have low clay content and hence possess good drainage characteristics.

- **Desert soils:** A large part of the arid region, belonging to western Rajasthan, Haryana, Punjab, lying between the Indus river and the Aravalli range is affected by the desert conditions of the geologically recent origin. This part is covered by a mantle of blown sand which, combined with the arid climate, results in poor soil development. They are light textured sandy soils and react well to the application of irrigation water.

- **Problem soils:** The problem soils are those, which owing to land or soil characteristics cannot be used for the cultivation of crops without adopting proper reclamation measures. Highly eroded soils, ravine lands, soils on steeply sloping lands etc. constitute one set of problem soils. Acid, saline and alkaline soils constitute another set of problem soil.

Some of the major soil groups of the country are listed in the following table:

Zone	Name	Climate	Regions	Major soil group
1	Western Himalayan Region	Humid	Jammu & Kashmir, Himachal pradesh, Uttaranchal	Submontane soils, Hill and terai soils
2	Bengal-Assam Basin	Humid	West Bengal, Assam	Riverine alluvium, terai soils, lateritic soils, red-yellow loams
3	Eastern Himalayan Region and bay islands	Humid	Andaman & Nicobar Islands, Arunachal Pradesh, Nagaland, Manipur, Tripura, Meghalaya	Red loamy soils, lateritic soils, red yellow soils, alluvial soils
			Punjab, Uttar	Calcareous alluvial soils, riverine

4	Sutlej-Ganga Plains	Sub-Humid	Pradesh, Bihar, Delhi, Uttaranchal	alluvium alkaline soils, red yellow loams, mixd red and black soils
5	Eastern and south eastern uplands	Sub-Humid to Humid	Orissa, Jharkhand, Chattisgarh, Andhra Pradesh	Lateritic soils, red yellow loams, mixed red and black soils, red loamy soils, coastal alluvium alluvial soils, red yellow soils, medium to deep black soils
6	Western plains	Arid	Harayana, Rajasthan, Dadra & Nagar Haveli	Lateritic soils, red yellow loams, mixed red and black soils, red loamy soils, coastal alluvium alluvial soils, red yellow soils, medium to deep black soils
7	Lava plateau and central highlands	Semi arid	Maharashtra, Goa, Madhya Pradesh, Daman & Diu	Riverine alluvium, coastal alluvium, mixed red and black soils, skeletal soils, shallow deep black soils and red sandy soils
8	Western Ghats and	Humid to semi	Karnataka, tamil Nadu, Kerala,	Lateritic soils, red sandy soils, deltaic

	Karnataka Plateau	arid	Pondicherry, Lakshadweep islands	coastal alluvium and red loamy soils.

3.2.4 Classification of soil water

As stated earlier, water may occur in the soil pores in varying proportions. Some of the definitions related to the water held in the soil pores are as follows:

- **Gravitational water:** A soil sample saturated with water and left to drain the excess out by gravity holds on to a certain amount of water. The volume of water that could easily drain off is termed as the gravitational water. This water is not available for plants use as it drains off rapidly from the root zone.

- **Capillary water:** the water content retained in the soil after the gravitational water has drained off from the soil is known as the capillary water. This water is held in the soil by surface tension. Plant roots gradually absorb the capillary water and thus constitute the principle source of water for plant growth.

- **Hygroscopic water:** the water that an oven dry sample of soil absorbs when exposed to moist air is termed as hygroscopic water. It is held as a very thin film over the surface of the soil particles and is under tremendous negative (gauge) pressure. This water is not available to plants.

The above definitions of the soil water are based on physical factors. Some properties of soil water are not directly related to the above significance to plant growth. These are discussed next.

3.2.5 Soil water constants

For a particular soil, certain soil water proportions are defined which dictate whether the water is available or not for plant growth. These are called the soil water constants, which are described below.

- **Saturation capacity:** this is the total water content of the soil when all the pores of the soil are filled with water. It is also termed as the maximum water holding capacity of the soil. At saturation capacity, the ***soil moisture tension*** is almost equal to zero.

- **Field capacity:** this is the water retained by an initially saturated soil against the force of gravity. Hence, as the gravitational water gets drained off from the soil, it is said to reach the field capacity. At field capacity, the macro-pores of the soil are drained off, but water is retained in the micropores. Though the soil moisture tension at field capacity varies from soil to soil, it is normally between 1/10 (for clayey soils) to 1/3 (for sandy soils) atmospheres.

- **Permanent wilting point:** plant roots are able to extract water from a soil matrix, which is saturated up to field capacity. However, as the water extraction proceeds, the moisture content diminishes and the negative (gauge) pressure increases. At one point, the plant cannot extract any further water and thus ***wilts.***

  Two stages of wilting points are recognized and they are:

  - Temporary wilting point: this denotes the soil water content at which the plant wilts at day time, but recovers during right or when water is added to the soil.

  - Ultimate wilting point: at such a soil water content, the plant wilts and fails to regain life even after addition of water to soil.

It must be noted that the above water contents are expressed as percentage of water held in the soil pores, compared to a fully saturated soil. Figure 2 explains graphically, the various soil constants; the full pie represents the volume of voids in soil.

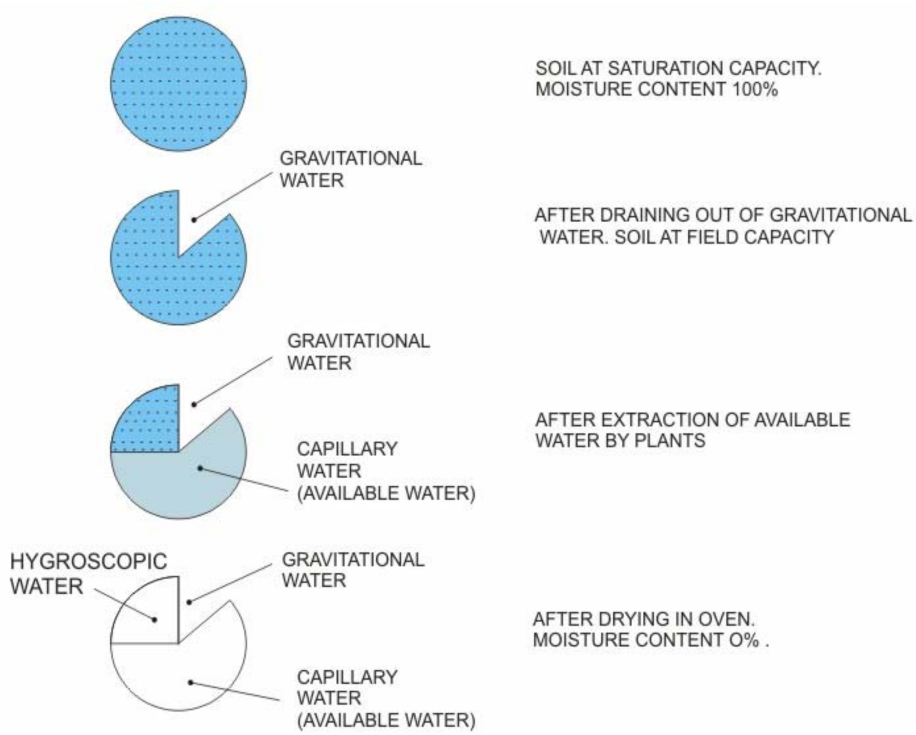


FIGURE 2 . Classification of soil water

As shown in Figure 2, the available water for plants is defined as the difference in moisture content of the soil between field capacity and permanent wilting point.

Field capacity and Permanent wilting point: Although the pie diagrams in Figure 2 demonstrate the drying up of saturated soil pores, all the soil constants are expressed as a percentage by weight of the moisture available at that point compared to the weight of the dried soil sand sample.

3.2.6 Soil water constants expressed in depth units:

In the last section, the soil water constants were mentioned as being expressed as weight percentages of the moisture content (that is amount of water) held by the water at a certain state with respect to the weight of the dried soil sample. The same may also be expressed as volume of water stored in the root zone of a field per unit area. This would consequently express the soil water constants as units of depths. The conversion from one form to the other is presented below:

Assume the following:

- Root zone depth = D (m)
- Specific weight of soil = γ_s (kg/m^3)
- Specific weight of water = γ_w (kg/m^3)
- Area of plot considered = 1m x 1m

Hence, the weight of soil per unit area would be: γ_s x 1 x D (kg)

The weight of water held by the soil per unit area would be equal to: γ x 1 x d

Where d is equivalent depth of water that is actually distributed within the soil pores. Hence the following constants may be expressed as:

$$\text{Field capacity} = \frac{\text{Weight of water held by soil per unit area}}{\text{Weight of soil per unit area}}$$

$$= \frac{\gamma_w * 1 * d}{\gamma_s * 1 * D} \qquad (1)$$

Thus, depth of water (***d*** $_{Fc}$) held by soil at field capacity (***FC***)

$$= \frac{\gamma_s}{\gamma_w} * D * FC \qquad (2)$$

Similarly, depth of water (***d*** $_{wp}$) held by soil at permanent wilting point (***PWP***)

$$= \frac{\gamma_s}{\gamma_w} * D * PWP \qquad (3)$$

Hence, depth of water (***d** $_{Aw}$*) available to plants

$$= \frac{\gamma_s}{\gamma_w} * D * [FC - PWP] \qquad (4)$$

Therefore, the depth of water available to plants per meter depth of soil

$$= \frac{\gamma_s}{\gamma_w} \ [FC - PWP] \qquad (5)$$

It may be noted that plants cannot extract the full available water with the same efficiency. About 75 percent of the amount is rather easily extracted, and it is called the readily available water. The available water holding capacity for a few typical soil types are given as in the following table:

Soil Texture	Field Capacity (FC) percent	Permanent Wilting Point (PWP) percent	Bulk Density(γ_s) Kg/m^3	Available water per meter depth of soil profile(m)
Sandy	5 to 10	2 to 6	1500 to 1800	0.05 to 0.1
Sandy loam	10 to 18	4 to 10	1400 to 1600	0.09 to 0.16
Loam	18 to 25	8 to 14	1300 to 1500	0.14 to 0.22
Clay loam	24 to 32	11 to 16	1300 to 1400	0.17 to 0.29
Clay	32 to 40	15 to 22	1200 to 1400	0.20 to 0.21

3.2.7 Water absorption by plants

Water is absorbed mostly through the roots of plants, though an insignificant absorption is also done through the leaves. Plants normally have a higher concentration of roots close to the soil surface and the density decreases with depth as shown in Figure 3.

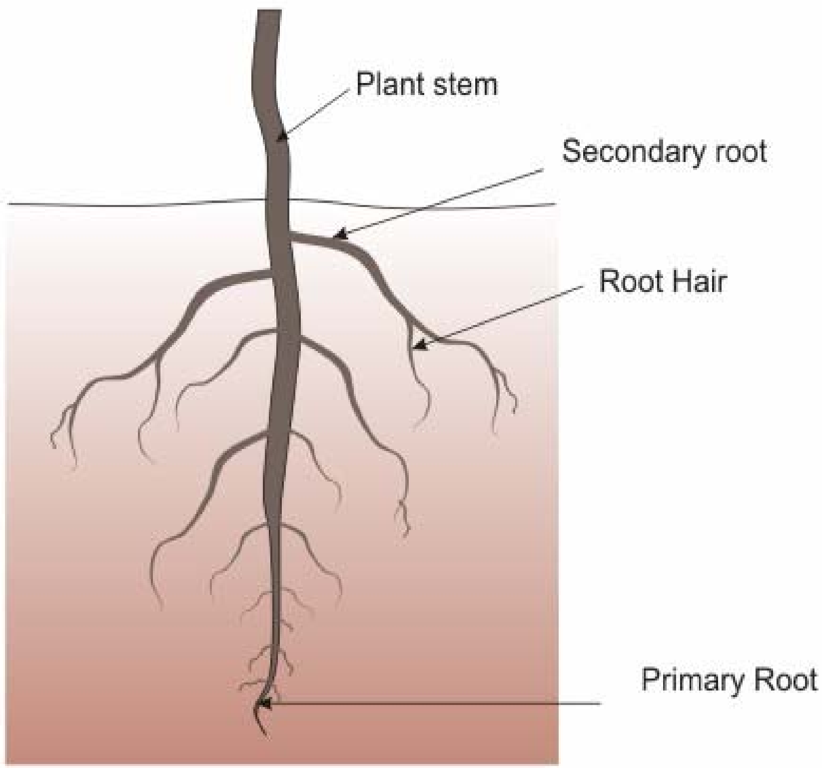


FIGURE 3. Typical root density variation of a plant with depth.

In a normal soil with good aeration, a greater portion of the roots of most plants remain within 0.45m to 0.60m of surface soil layers and most of the water needs of plants are met from this zone. As the available water from this zone decreases, plants extract more water from lower depths. When the water content of the upper soil layers reach wilting point, all the water needs of plants are met from lower layers. Since there exists few roots in lower layers, the water extract from lower layers may not be adequate to prevent wilting, although sufficient water may be available there.

When the top layers of the root zone are kept moist by frequent application of water through irrigation, plants extract most of the water (about 40 percent) from the upper quarter of their root zone. In the lower quarter of root zone the water extracted by the plant meets about 30 percent of its water needs. Further below, the third quarter of the root zone extracts about 20 percent and the lowermost quarter of root zone extracts the remaining about 10 percent of the plants water. It may be noted that the water extracted from the soil by the roots of a plant moves upwards and essentially is lost to the atmosphere as water vapours mainly through the leaves. This process, called transpiration, results in losing almost 95percent of water sucked up. Only about 5percent of water pumped up by the root system is used by the plant for metabolic purpose and increasing the plant body weight.

3.2.8 Watering interval for crops

A plot of land growing a crop has to be applied with water from time to time for its healthy growth. The water may come naturally from rainfall or may supplemented by artificially applying water through irrigation. A crop should be irrigated before it receives a set back in its growth and development. Hence the interval between two irrigations depends primarily on the rate of soil moisture depletion. Normally, a crop has to be irrigated before soil moisture is depleted below a certain portion of its availability in the root zone depending on the type pf plant. The intervals are shorter during summer than in winter. Similarly, the intervals are shorter for sandy soils than heavy soils. When the water supply is very limited, then the interval may be prolonged which means that the soil moisture is allowed to deplete below 50percent of available moisture before the next irrigation is applied. The optimum rates of soil moisture for a few typical crops are given below (Reference: Majumdar, D K, 2000)

- Maize : Field capacity to 60 percent of availability
- Wheat : Field capacity to 50 percent of availability
- Sugarcane: Field capacity to 50 percent of availability
- Barley : Field capacity to 40 percent of availability
- Cotton : Field capacity to 20 percent of availability

As for rice, the water requirement is slightly different than the rest. This is because it requires a constant standing depth of water of about 5cm throughout its growing period. This means that there is a constant percolation of water during this time and it has estimated that about 50 to 70 percent of water applied to the crop is lost in this way.

For most of the crops, except rice, the amount of water applied after each interval should be such that the moisture content of the soil is raised to its field capacity. The soil moisture depletes gradually due to the water lost through evaporation from the soil surface and due to the absorption of water from the plant roots, called transpiration more of which has been discussed in the next session. The combined effect of evaporation and transpiration, called evapo-transpiration (ET) decides the soil water depletion rate for a known value of ET (which depends on various factors, mainly climate); it is possible to find out the irrigation interval.

Some of the operational soil moisture ranges of some common crops are given below:

Rice:

This crop is grown both in lowland and upland conditions and throughout the year in some parts of the country. For lowland rice, the practice of keeping the soil saturated or upto shallow submergence of about 50mm throughout the growing period has been found to be the most beneficial practice for obtaining maximum yields. When water resources are limited, the land must be submerged atleast during critical stages of growth. The major portion of the water applied to the rice crop, about 50-75% is lost through deep percolation which varies with the texture of the soil. Since the soil is kept constantly submerged for rice growth, all the pores are completely filled with water through it is in a state of continuous downward movement. The total water required by the rice plant is about 1.0 to 1.5m for heavy soils and soils with high water table; 1.5 to 2.0m for medium soils and 2.0 to 2.5 for light soils with deep water table.

Wheat:

The optimum soil moisture range for tall wheats is from the field capacity to 50% of availability. The dwarf wheats need more wetness, and the optimum moisture range is from 100 to 60 percent availability. The active root zone of the crops varies from 0.5 to 0.75m depending upon the soil type. The total water requirement for wheat plants vary from 0.25m to 0.4 m in northern India to about 0.5m to 0.6m in Central India.

Barley:

This crop is similar to wheat in its growing habits, but can withstand more droughts because of the deeper and well spread root system. The active root zone of Barley extends between 0.6m to 0.75m on different soil types. The optimum soil moisture ranges from the field capacity to 40% of availability.

Maize:

The crop is grown almost all over the country. The optimum soil moisture range is from 100 to 60% of availability in the maximum root zone depth which extends from 0.4 to 0.6 on different soil types. The actual irrigation requirement of the crop varies with the amount of rainfall. In north India, 0.1m and 0.15m is required to establish the crop before the onset of monsoon. In the south, it is found that normal rain fall is sufficient to grow the crop in the monsoon season where as 0.3m of water is required during water.

Cotton:

The optimum range of soil moisture for cotton crop is from the field capacity to 20% of available water. He root zone varies upto about 0.75m. The total water requirement is about 0.4m to 0.5m.

Sugarcane:

The optimum soil moisture for sugarcane is about 100 to 50 percent of water availability in the maximum root zone, which extends to about 0.5m to 0.75m in depth. The total water depth requirement for sugarcane varies from about 1.4m to 1.5m in Bihar; 2.2m – 2.4m in Karnataka; and 2.0 – 2.3m in Madhya Pradesh.

3.2.9 Importance of water in plant growth

During the life cycle of a plant water, among other essential elements like air and fertilizers, plays a vital role, some of the important ones being:

- Water maintains the turgidity of the plant cells, thus keeping the plant erect. Water accounts for the largest part of the body weight of an actively growing plant and it constitutes 85 to 90 percent of the body weight of young plants and 20 to 50 percent of older or mature plants.

- Water provides both oxygen and hydrogen required for carbohydrate synthesis during the photosynthesis process.

- Water acts as a solvent of plant nutrients and helps in the uptake of nutrients from soil.

- Food manufactured in the green parts of a plant gets distributed throughout the plant body as a solution in water.

- Transpiration is a vital process in plants and does so at a maximum rate (called the potential evapo transpiration rate) when water is available in adequate amount. If soil moisture is not sufficient, then the transpiration rate is curtailed, seriously affecting plant growth and yield.

- Leaves get heated up with solar radiation and plants help to dissipate the heat by transpiration, which itself uses plant water.

3.2.10 Irrigation water quality

In irrigation agriculture, the quality of water used for irrigation should receive adequate attention. Irrigation water, regardless of its source, always contains some soluble salts in it. Apart from the total concentration of the dissolved salts, the concentration of some of the individual salts, and especially those which are most harmful to crops, is important in determining the suitability of water for irrigation. The constituents usually determined by analyzing irrigation water are the electrical conductivity for the total dissolved salts, soluble sodium percentage, sodium absorption ratio, boron content, PH, cations such as calcium, magnesium, sodium, potassium and anions such as carbonates, bicarbonates, sulphates, chlorides and nitrates.

Water from rivers which flow over salt effected areas or in the deltaic regions has a greater concentration of salts sometimes as high as 7500 ppm or even more. The quality of tank or lake water depends mainly on the soil salinity in the water shed areas and the aridity of the region. The quality of ground water resources, that is, from shallow or deep wells, is generally poor under the situations of

- high aridity
- high water table and water logged conditions
- in the vicinity of sea water

on the basis of suitability of water for irrigation, the water may be classified under three categories, which are shown in the following table:

Class	Electrical Conductivity (micro-ohm/cm)	Total Dissolved Solids (ppm)	Exchangeable sodium (percentage)	Chloride (ppm)	Sulphates (ppm)	Boron (ppm)	Remarks
I	0-1000	0-700	0-60	0-142	0-192	0-0.5	Excellent to good for irrigation
II	1000-3000	700-2000	60-75	142-355	192-480	0.5-2.0	Good to injurious; suitable only with permeable soils and moderate teaching. Harmful to more sensitive crops.
III	>3000	>2000	>75	>355	>480	>2.0	Unfit for irrigation

3.2.11 Important Definitions

1. Root Zone: The soil root zone is the area of the soil around the plant that comes in contact with the plant root (Figure 4).

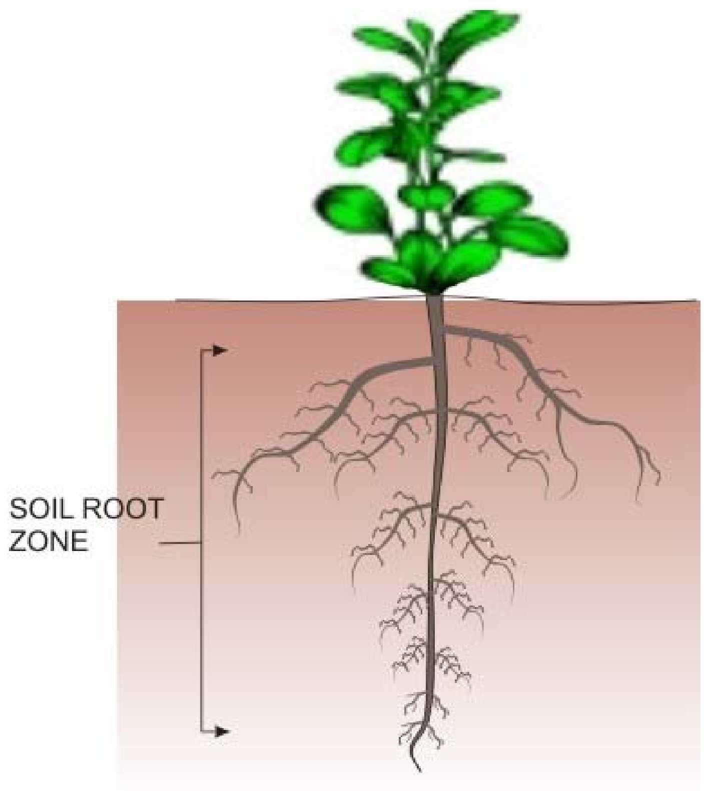


FIGURE 4 **Definition of soil root zone**

2. Soil Moisture tension: In soils partially saturated with water there is moisture tension, which is equal in magnitude but opposite in sign to the soil water pressure. Moisture tension is equal to the pressure that must be applied to the soil water to bring it to a hydraulic equilibrium, through a porous permeable wall or membrane, with a pool of water of the same composition.

3. Wilts: Wilting is drooping of plants. Plants bend or hang downwards through tiredness or weakness due to lack of water.

3.2.12 Bibliography

- Majumdar, D K (2000) *Irrigation Water Management* by, Prentice Hall of India.

Module 3

Irrigation Engineering Principles

Lesson 3

Estimating Irrigation Demand

Instructional objectives

On completion of this lesson, the student shall learn the following:

1. How much water is required for the proper growth of important crops
2. How to estimate the water demand of crops
3. What are the different seasons of crop growth
4. What are the usual cropping patterns
5. On what variables does the crop water requirement vary?

3.3.0 Introduction

A plot of land growing a certain crop or a combination of crops has to be supplied with water from time to time. Primarily, the plot or field is expected to receive water from rain falling on the land surface. But, as we know, the distribution of rain is rather uncertain both in time and space. Also some of the rain as in a light shower does not reach the ground as it may be intercepted by the leaves of the plant during a heavy downpour; much of the water might flow away as surface runoff. Hence, only a certain amount of falling rain may be effective in raising the soil moisture that is actually useful for plant growth. Hence, for proper crop growth, the effective rain has to be supplemented by artificially applying water to the field by irrigation.

If the area of the field is small, water may be supplied from the local ground water source. If the field is large, supplemented irrigation water may be obtained from a local surface water source, like a river, if one is available nearby. The work of a water resources engineer therefore would be to design a suitable source for irrigation after knowing the demand of water from field data. In this lesson, we proceed on to find out the methods by which estimation may be made for irrigation water demand.

3.3.1 Crop water requirement

It is essential to know the water requirement of a crop which is the total quantity of water required from its sowing time up to harvest. Naturally different crops may have different water requirements at different places of the same country, depending upon the climate, type of soil, method of cultivation, effective rain etc.

The total water required for crop growth is not uniformly distributed over its entire life span which is also called crop period. Actually, the watering stops same time before harvest and the time duration from the first irrigation during sowing up to the last before harvest is called base period. Though crop period is slightly more than the base period, they do not differ from practical purposes. Figure 1 indicates the relative usage of water for a typical crop during its entire growth period.

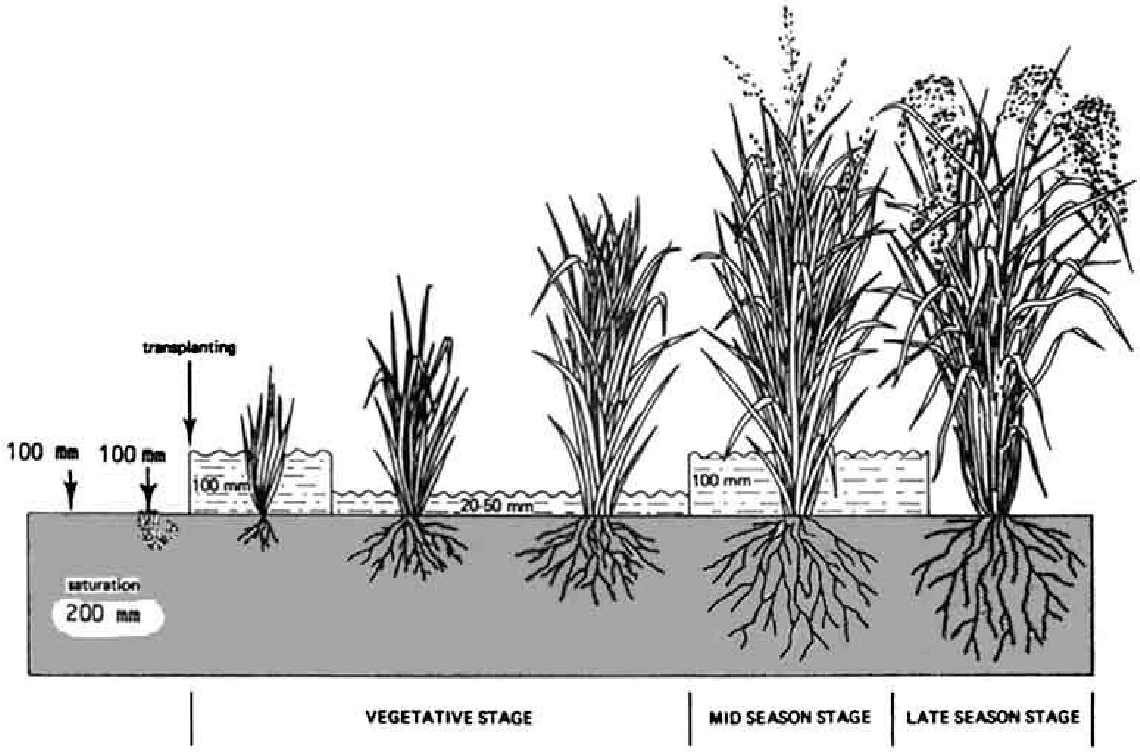


FIGURE 1. Variation in the requirement of water for paddy with stage of growth
(Image courtesy: Food and Agriculture Organisation, FAO)

Sometimes, in the initial stages before the crop is sown, the land is very dry. In such cases, the soil is moistened with water as t helps in sowing the crops. This is known as ***paleo*** irrigation. A term ***kor*** watering is used to describe the watering given to a crop when the plants are still young. It is usually the maximum single watering required, and other waterings are done at usual intervals.

The total depth of water required to raise a crop over a unit area of land is usually called ***delta***. Some typical values of delta for common crops in some regions of India are as follows:

Rice

- 1000mm to 1500mm for heavy soils or high water table
- 1500mm to 2000mm for medium soils
- 2000 to 2500 for light soils or deep water table
- 1600mm for upland conditions

Wheat

- 250mm to 400mm in northern India

- 500mm to 600mm in Central India
- Barley: 450mm

Maize

- 100mm during rainy season
- 500mm during winter season
- 900mm during summer season
- Cotton: 400 – 500mm

Sugarcane

- 1400mm to 1500mm in Bihar
- 1600mm to 1700mm in Andhra Pradesh
- 1700mm to 1800mm in Punjab
- 2200mm to 2400mm in Madhya Pradesh
- 2800mm to 3000mm in Maharashtra

This information has been gathered from the ***Handbook of Agriculture*** (fifth edition, 2000) published by the Indian Council of Agricultural Research.

3.3.2 Duty of water

The term ***duty*** means the area of land that can be irrigated with unit volume of irrigation water. Quantitatively, duty is defined as the area of land expressed in hectares that can be irrigated with unit discharge, that is, 1 cumec flowing throughout the base period, expressed in days.

Imagine a field growing a single crop having a base period B days and a Delta Δ mm which is being supplied by a source located at the head (uppermost point) of the field, as shown in Figures 2 and 3.

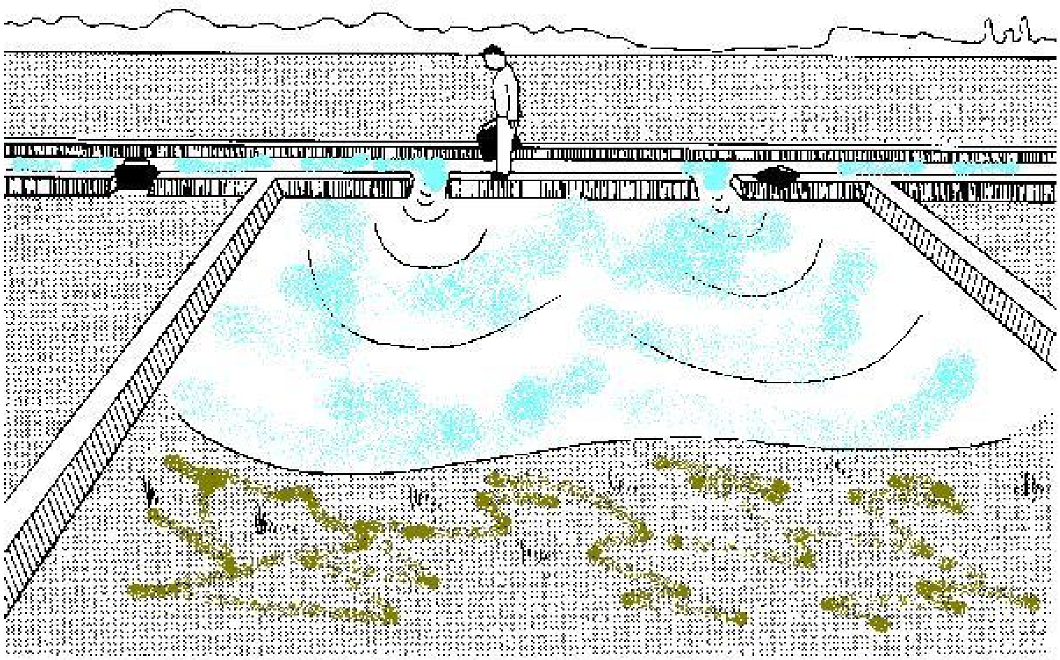


FIGURE 2. Border irrigation method of applying water at the head of a field
(Image courtesy: Food and Agriculture Organisation, FAO)

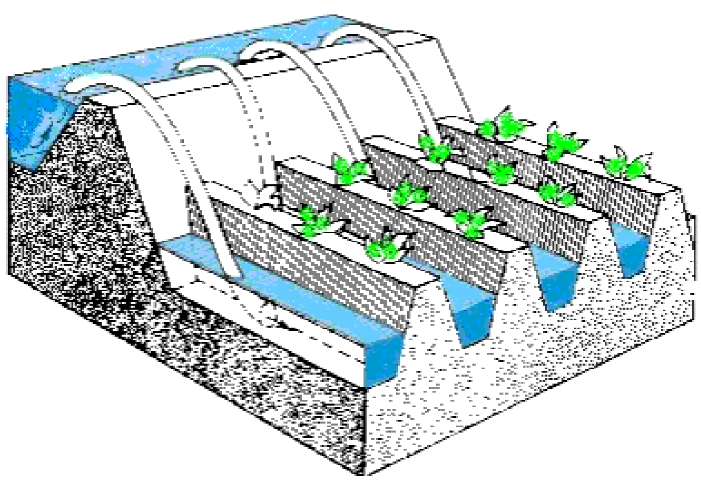


FIGURE 3. Furow irrigation method of applying water to a field

(Image courtesy: Food and Agriculture Organisation, FAO)

The water being supplied may be through the diversion of river water through a canal, or it could be using ground water by pumping (Figure 4).

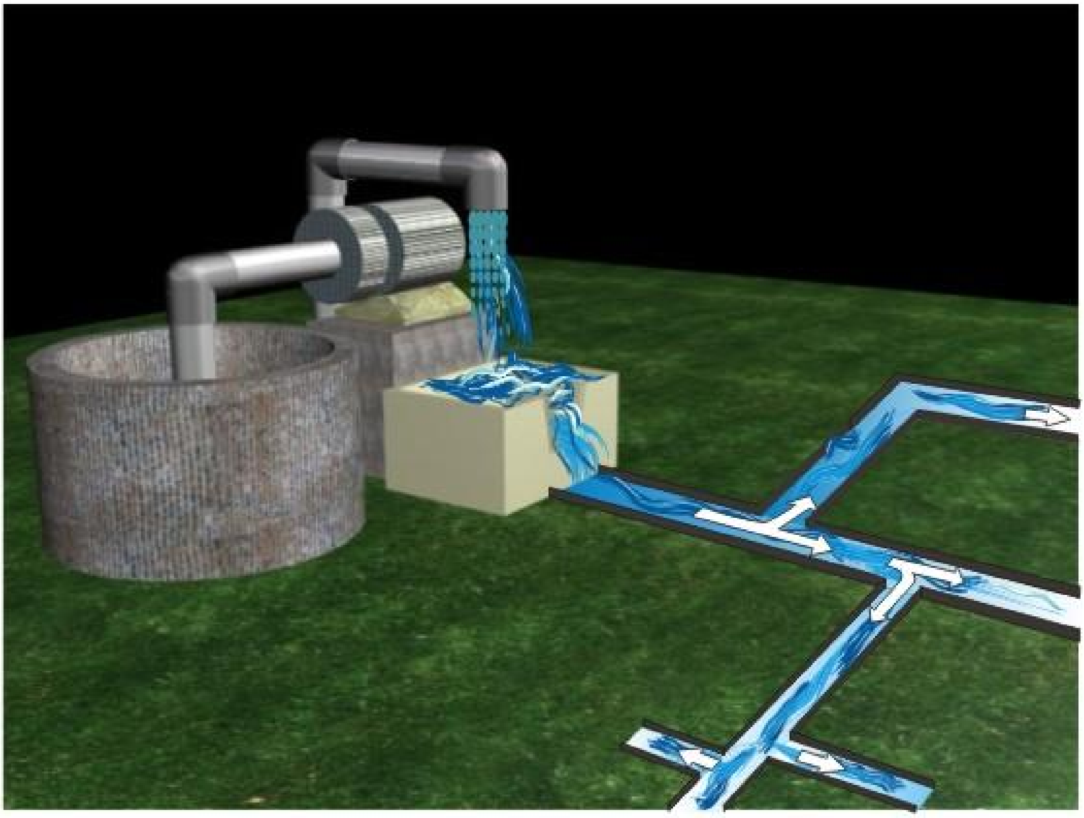


FIGURE 4. Water applied to field by pumping ground water

If the water supplied is just enough to raise the crop within D hectares of the field, then a relationship may be found out amongst all the variables as:

Volume of water supplied = B*60*60*24 m^3
Area of crop irrigated = D*10^4 m^2
Volume of water supplied per unit area = $\frac{86400}{10000D} = \frac{8.64B}{D}$

Hence, knowing two of the three variables B, D and Δ the third party may be found out.

The duty of irrigation water depends upon a number of factors; some of the important ones are as follows:

- **Type of crop:** As different crops require different amount of water for maturity, duties are also required. The duty would vary inversely as the water requirement of crop.

- **Climate season and type of soil:** Some water applied to the field is expected to be lost through evaporation and deep percolation.

Evaporation loss has a direct bearing on the prevalent climate and percolation may be during drier seasons when the water table is low and soil is also dry. Percolation loss would be more for sandy soils than silty or clayey soils.

- **Efficiency of cultivation methods:** If the tillage and methods of water application are faulty and less efficient, then the amount of water actually reaching the plant roots would be less. Hence, for proper crop growth more water would be required than an equivalent efficient system. Also, if the water is conveyed over long distances through field channels before being finally applied to the field, then also the duty will rise due to the losses taking place in the channels.

3.3.3 Crop growing seasons in India

Each crop has its own sowing and harvesting seasons and it is important to have a knowledge of this which may help to decide the total water demand in a field having mixed crops.

In India, the northern and north eastern regions have two distinct cropping seasons. The first coinciding mostly with the South western monsoon is called ***kharif*** , which spans mostly from July to October. The other, called ***rabi***, spans generally over October to March. The summer season crops are planted sometime between April and June. In southern part of India, there is no such distinct season, but each region has its own classification of seasons.

Generally, the kharif is characterized by a gradual fall in temperature, more numerous cloudy days, low intensity, high relative humidity and cyclonic weather. During Rabi, there is a gradual rise in temperature, bright sunshine, near absence of cloud days, and a lower relative humidity.

The following table indicates some the regional cropping calendars in India.

State	Season	Local name	Growing month
Andhra Pradesh	Kharif	Serva or Abi	July – December
	Rabi	Dalwa or Tabi	December – April
	Summer	In limited areas	March/April – June
Assam	Pre-monsoon	Ahu	Mar/April– June/july
	-	Sali	June/July- Nov/Dec
	-	Boro	Nov - May
Bihar	Summer	-	March – July/Aug
	Autumn	-	May/June–

			Sept/Oct
	Winter	-	June – Nov/Dec
Gujarat	Kharif	Chomasu Dangar	June/July-Oct/Nov
	-	Unala Dangar	Dec – June
Haryana	Kharif	-	May/June–Sept/Oct
Himachal Pradesh	Kharif	-	June/July-Sept/Oct
Jammu & Kashmir	-	-	Jammu: June-Nov Kashmir: Last week of April - October
Karnataka	Kharif	-	June – Dec
	Summer	-	Jan-May/June
Kerala	first crop	Virippu	April-May/Sept-Oct
	Second crop	Mundakan	Sept-Oct/Dec-Jan
	Third crop	Punja	Dec/Jan-Mar/April
Madhya Pradesh	Kharif	-	June/July-Dec
Maharashtra	Kharif	-	June/July-Dec
Manipur	Kharif	-	Mar/June-Sept/Oct
Meghalaya	Kharif	-	May/June-Aug/Sept
	Rabi	-	----------------
Nagaland	Kharif	-	May/June-Nov/Dec
	Rabi	-	Feb - May
Orissa	-	Sarad	June-Dec
	-	Dalua	Dec-April
	-	Beali (short Duration)	April/May –Sept (Only in uplands)
Punjab	Kharif	-	May – Nov
Rajasthan	Kharif	-	June/July-Sept/oct
Tamil Nadu	-	Navarai	Jan-April
	-	Sornavari	April – July
	-	Kar or Kuruvai	June – August
	-	Samba	June/July-Nov/Dec
	-	Thaladi or Pishanam	Sept/Oct-Feb/March
Uttar Pradesh	Kharif	-	June – Oct
West Bengal	Pre-Kharif	Aus	April-Sept
	Kharif	Aman	June-Dec
	Summer	Boro	End Nov-Mid June

3.3.4 Variations in the country's irrigation demands

It may be appreciated that in India there is a large variation of rainfall, which is the primary source of irrigation in most parts of the country. In fact, the crops grown in various regions have been adapted according to the local rainfall availability. Water resources engineers are therefore concerned with arranging supplementary water to support the crops for seasonal variations of rainfall in order to ensure an assured crop harvest.

Further, due to variation in the type of soil over different regions of the country, the types of crop grown also varies- thus dictating the water requirement at different regions during different times. Hence, the country has been broadly classified into eight agro climatic zones, a list of which is given.

3.3.5 Cropping patterns

Planning of an irrigation project requires estimation of water demand of a cultivated area. Naturally, this would depend upon the type of crop grown. Since irrigation water may have to be supplied to one field growing a combination of crops or to many fields growing different crops, it is important to understand certain cropping practices which would be helpful in estimating the irrigation demand. Some of the prevalent practices are as follows:

- Crops grown solely or mixed: Mixed cropping
- Crops grown in a definite sequence: Rotational cropping
- Land occupied by one crop during one season: Mono cropping
- Land occupied by two crops: double cropping
- Land sowed with more than one crop in a year: multiple cropping

3.3.6 Irrigation water need

For raising a field crop effectively, it is essential to supply water through artificial irrigation supplementing the rain falling over the plot of land and raising the soil moisture. Irrigation requirement for a typical crop and an assumed rainfall pattern may be illustrated as in Figure 5.

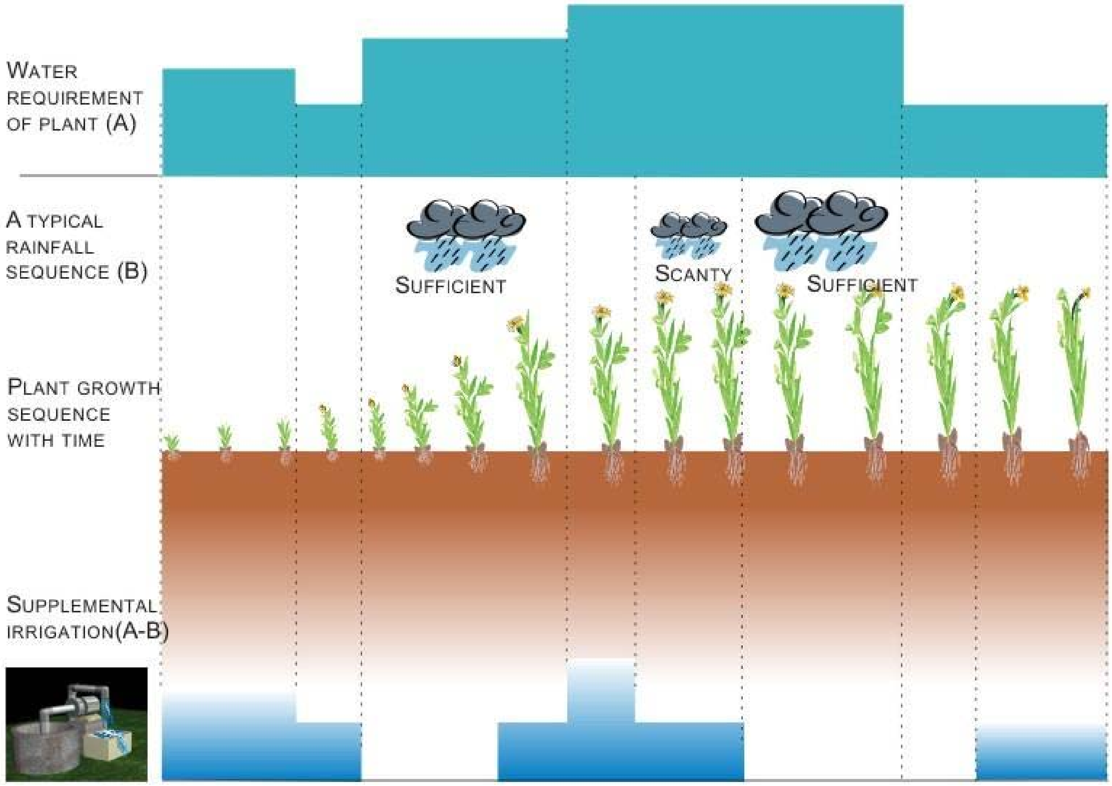


FIGURE 5 . Typical irrigation requirement of a crop and water provided naturally by rain or artificially by pumping

Hence, it may be seen that irrigation water requirement is rather a dynamic one. Also, the crop water requirement is shown with slight variation, it actually shows more variation, depending on the type of crop and the prevalent climate. Though farmers may be tempted to allow more water to the plants through supplemental irrigation, it must be remembered that there is an optimum water requirement schedule of each crop depending upon its stage of growth. It has been proved that at times application of more water may cause reduction in yield.

3.3.7 Variation of crop water requirement

The total water need for various plants, known as delta, has been discussed earlier. However, in planning the supply of irrigation water to a field crop, it is essential to estimate the water requirement of each plot of land growing a crop or crops at any point of time. This may be done by studying the dynamic interaction between a crop and the prevalent climate and the consequent water requirement. The demand would, naturally be also dependant on the type of crop and its stage of growth.

Plant roots extract water from the soil. Most of this water doesn’t remain in the plant, but escapes to the atmosphere as vapour through the plants leaves and stems, a process which is called ***transpiration*** and occurs mostly during daytime. The water on the soil surface as well as the water attaching to the leaves and stem of a plant during a rainfall also is lost to the atmosphere by evaporation. Hence, the water need of a crop consists of transpiration plus evaporation, together called ***evapotranspiration***.

The effect of the major climatic factors on crop water needs may be summarized as follows:

- Sunshine
- Temperature
- Humidity
- Wind speed

The variation of evapotranspiration upon these factors is illustrated in Figure 6.

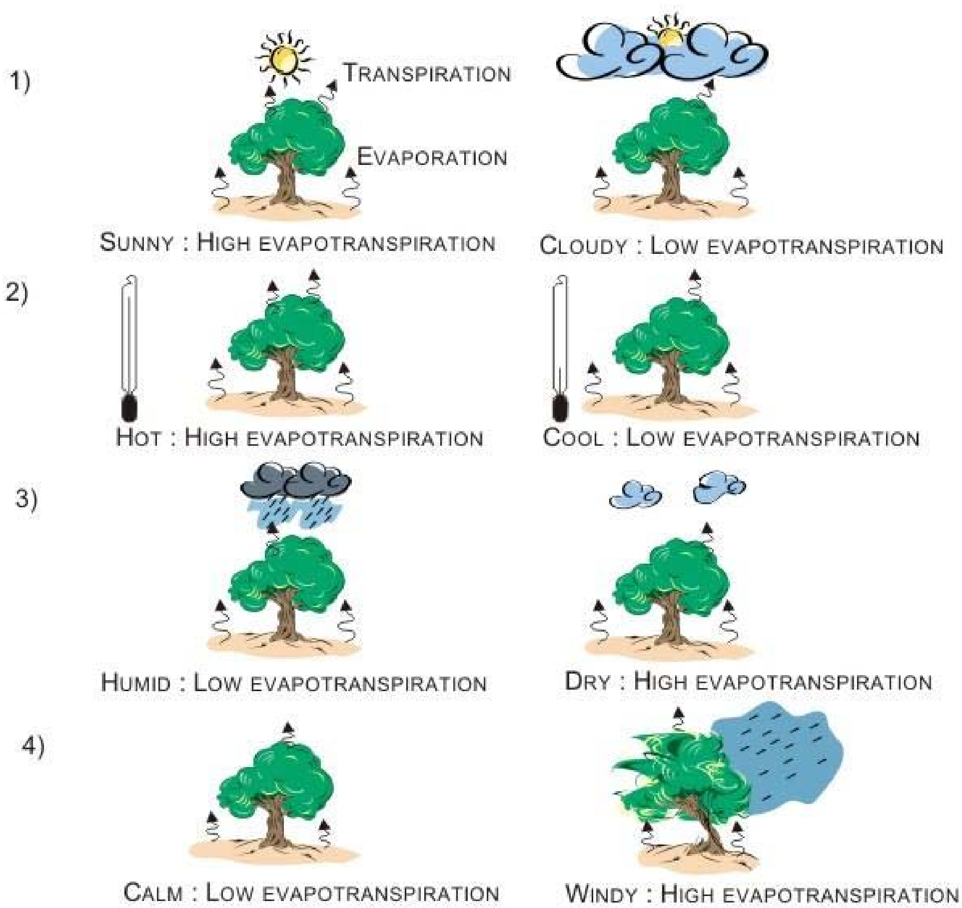


FIGURE 6. Dependence of evapotranspiration upon different climatological factors

Since the same crop grown in different climatic variations have different water needs, it has been accepted to evaluate the evapotranspiration rate for a standard or reference crop and find out that of all other crops in terms of this reference. Grass has been chosen as standard reference for this purpose. The evapotranspiration rate of this standard grass is, therefore, called the ***reference crop evapotranspiration*** and is denoted as ET_O, which is of course, the function of the climatic variables. Training Manual 3: Irrigation Water Needs published by the Food and Agricultural Organisation, (FAO) and available on-line through the under-mentioned web-site gives an idea about the variation of ET_O under different climatic conditions and is reproduced in the table below.

http://www.fao.org/ag/agL/public.stm#iwmtm

Table showing the daily variation of water needs of standard grass (in mm) under different climatic patterns (ET_O)

Climatic Zone	Mean daily Temperature		
	Low (<15^0C)	Medium (15^0-25^0C)	High (>25^0C)
Desert/Arid	4-6	7-8	9-10
Semi-arid	4-5	6-7	8-9
Sub-humid	3-4	5-6	7-8
humid	1-2	3-4	5-6

Other methods have been devised to calculate ET_O for given values of climatic parameters. These are discussed in the next section. In this section, we proceed on to discuss, how to find crop water need, if ET_O is known.

Agricultural scientists have evaluated a factor called ***crop factor*** and denoted it by K_C, to evaluate specific crop water needs. Naturally, Kc would be different for different crops and would not be the same throughout the growth season of one type of crop. Thus, the crop evapotranspiration, denoted by ET_C is to be evaluated as under:

$$ET_O = K_C * ET_C \tag{1}$$

Both ET_O and ET_C should be in the same units and generally, mm/day is used as a standard all over the world.

In order to simply the calculations, the factor K_C has been evaluated for 4 stages of a crop growth usually denoted as

1. Initial stage
2. Crop development stage
3. Mid-season stage
4. Late season stage

The FAO Training Manual 3 gives the growth stage periods and the corresponding K_C values for some typical crops. In the table below, that for rice is presented.

Rice	Climate			
	Little wind		Strong wind	
Growth stage	Dry	Humid	Dry	Humid
0-60 days	1.1	1.1	1.1	1.1
Mid season	1.2	1.05	1.35	1.3
Last 30 days before harvest	1.0	1.0	1.0	1.0

It may be mentioned that any crop doesn't have a fixed total growth period, which is the summation of growth stage periods given above. There is usually a range, depending upon the variety of the crop and the condition in which it is cultivated.

The values of K_C also depend upon the climate and particularly on humidity and wind speed, as shown for rice in the above table. In general, the values of K_C should be reduced by 0.05 if the relative humidity is high (>80%) and the wind speed is low (<2m/s). Likewise, the values should be increased by 0.05 if the relative humidity is low (<50%) and the wind speed is high (>5m/s).

For full details, the FAO training manual 3 may be consulted as K_C values for other crops are evaluated in different manners. For some of the crops, the following table provides information:

Crop	Variety	Crop growth stage				Total growth period
Cabbage/Carrot	Short duration	20 days	25 days	60 days	15 days	120 days
	Long duration	25 days	30 days	65 days	20 days	140 days
	K_C	0.45	0.75	1.05	0.9	
Cotton/Fiax	Short duration	30 days	50 days	55 days	45 days	180
	Long duration	30 days	50 days	65 days	50 days	195
	Kc	0.45	0.75	1.15	0.75	
Lentil/Pulses	Short duration	20 days	30 days	60 days	40 days	150
	Long duration	25 days	35 days	70 days	40 days	170
	K_C	0.45	0.75	1.1	0.5	
	Short	20	25	25	10	80

	duration					
Maize	Long duration	20	30	50	10	110
	K_C	0.4	0.8	1.15	1.0	
Onion (dry)	Short duration	15	25	70	40	150
	Long duration	20	35	110	45	210
	K_C	0.5	0.75	1.05	0.85	
Potato	Short duration	25	30	30	20	105
	Long duration	30	35	50	30	145
	K_C	0.45	0.75	1.15	0.85	

3.3.8 Estimation of reference crop ET_O

Of the many methods available, the commonly used ones are two:

i. Experimental methods, using the experimentation data from evaporation pan.
ii. Theoretical methods using empirical formulae, that take into account, climatic parameters.

3.3.8.1 *Experimental method*

Estimation of ET_0 can be made using the formula

$$ET_O = Kpan \times Epan \qquad (2)$$

Where ET_O is the ***reference crop evapotranspiration*** in mm/day, Kpan is a coefficient called ***pan coefficient*** and Epan is the ***evaporation*** in mm/day from the pan.

The factor Kpan varies with the position of the equipment (say, whether placed in a fallow area or a cropped area), humidity and wind speed. Generally, the details are supplied by the manufacturers of the pan. For the ***US Class A evaporation pan***, which is also used in India, Kpan varies between 0.35 and 0.85, with an average value of 0.7.

It may be noticed that finding out ET_C would involve the following expression

$$ET_C = Kcrop \times ET_O = Kc \times Epan \times Kpan \qquad (3)$$

K_C has been discussed in the previous section. If instead, Kcrop x Kpan is taken as a single factor, say K, then ET_C may directly be found from Epan as under:

ET_C =K x Epan, where K may be called the crop factor (4)

The water management division of the Department of Agriculture, Government of India has published a list of factors for common crops and depending upon the stage of growth, which have to be multiplied with the evaporation values of the USWB Class A evaporation pan.

3.3.8.2 *Theoretical methods*

The important methods that have been proposed over the years take into account, various climatic parameters. Of these, only the following would be discussed, as they are the most commonly used.

3.3.8.2.1 Blanney-Criddle formula:

This formula gives an estimate of the mean monthly values of ET_O, which is stated as

$$ET_O = p\,(0.46\,T_{mean} + 8.13) \qquad (5)$$

Where p is the mean daily percentage of annual day time hours and has been estimated according to latitude; Tmean is the mean monthly temperature in degrees Centigrade and may be taken as ½ x (Tmax + Tmin) for a particular month. Thus using the Equation (1), one may evaluate ET_C for each month of the growing season, from which the total water need for the full growing season of the crop may be found out.

3.3.8.2.2 Penman-Monteith method:

This method suggests that the value of ET_O may be evaluated by the following formula:

$$ET_O = \frac{0.408\,\Delta(R_n - G) + \gamma \dfrac{900}{T+273} u_2 (e_s - e_a)}{\Delta + \gamma(1 + 0.34 u_2)} \qquad (6)$$

Where the variables have the following meanings:

ET_o reference evapotranspiration [mm day^{-1}],
R_n net radiation at the crop surface [MJ m^{-2} day^{-1}],
G soil heat flux density [MJ m^{-2} day^{-1}],
T mean daily air temperature at 2 m height [°C],
u_2 wind speed at 2 m height [m s^{-1}],
e_s saturation vapour pressure [kPa],
e_a actual vapour pressure [kPa],
e_s - e_a saturation vapour pressure deficit [kPa],
Δ slope vapour pressure curve [kPa °C^{-1}],
g psychrometric constant [kPa °C^{-1}].

3.3.9 Application interval of irrigation water

The water need of a crop is usually expressed as mm/day, mm/month or mm/season, where season means the crop growing period. Whatever be the water need, it need not be applied each day. A larger amount of water may be applied once in a few days and it gets stored in the crop root zone, from where the plant keeps on extracting water.

Soon after irrigation, when the soil is saturated, up to the field capacity, the extraction of water from the soil by the plants is at the peak. This rate of water withdrawal decreases as the soil moisture depletes (Figure 7).

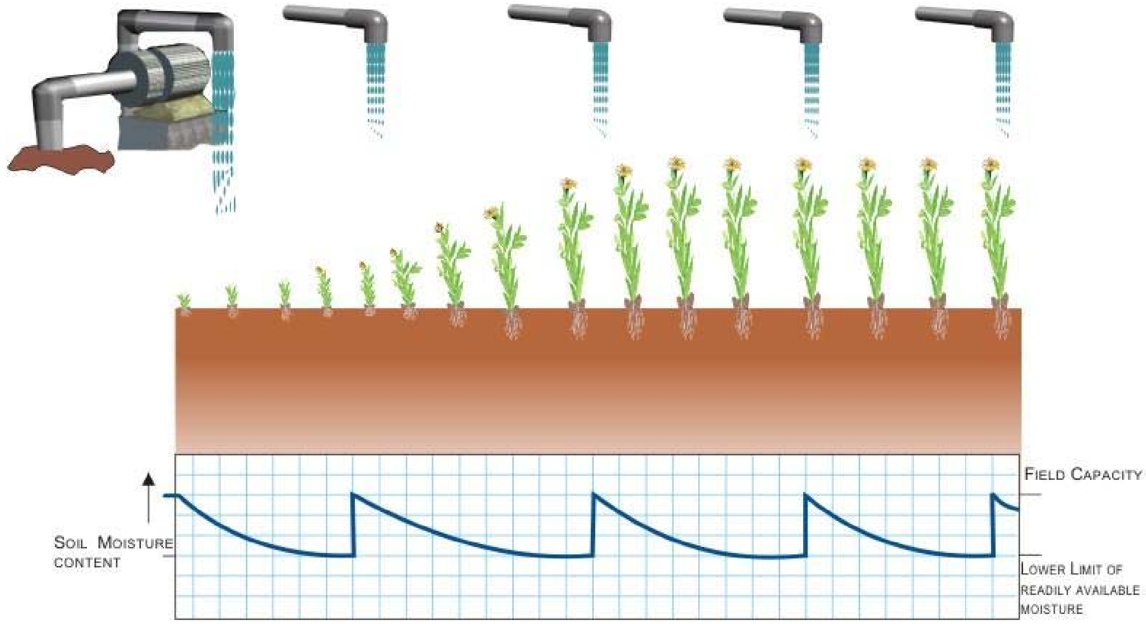


FIGURE 7. Rise and fall of soil moisture content due to irrigation and evapotranspiration

A stage is reached, in the moisture content of the soil, below which the plant is stressed to extract and unless the soil moisture is increased by application of water, the plant production would decrease. The difference of moisture content between field capacity (the maximum content of available water) and the lowest allowable moisture content is called the optimum soil water.

The optimum soil moisture range for some common crops is required from which the interval period of irrigation water may be estimated as follows:

$$\text{Irrigation period (days)} = \frac{\text{Net depth of soil depletion in the crop area just before irrigation (mm)}}{\text{ETc (mm/day)}} \qquad \ldots\ldots\ldots \quad (7)$$

Where the crop evapotranspiration rate (ET_C) may be determined according to the crop type, growth stage and prevailing climate as mentioned in the previous sections.

The irrigation period, as calculated above, has not taken the soil retention characteristics. Naturally, a soil with greater water retentive capacity serves as a bigger water reservoir for crops and supply of irrigation can be delayed. Consequently, frequency of irrigation is lower and interval of irrigation is longer in heavier soils and in soils with good organic content and low content of soluble salts.

Further, the calculation of ET_C as presented earlier and employed in the equation above to calculate irrigation period, what is called, the ***potential evapotranspiration*** (PET). This is the highest rate of water with drawl by an actively growing crop with abundant water supply. However as the soil moisture depletes, the ***actual evapotranspiration*** (AET) also decreases, as evident from the decrease in the gradient of the soil moisture curve with time in Figure 7. The AET would also be different from the PET if the climatic conditions like humidity temperature etc vary from the ones assumed when calculating PET. Nevertheless since PET is easier to estimate and since it would also be higher than AET, it is rational to consider PET, while designing the water requirement for a field of crop.

3.3.10 Total water requirement in growing a crop

The water that is required to irrigate a field or plot of land growing the particular crop not only has to satisfy the evapotranspiration needs for growing the crop, but would also include the following:

- Losses in the form of deep percolation while conveying water from the inlet of the field upto its last or tail end as the water gets distributed within the field
- Water requirement for special operations like land preparation, transplanting, leaching of salts, etc.

Further, the evapotranspiration requirement of crops (ET) really doesn't include the water required by crops for building up plant tissues, which is rather negligible compared to the evaporation needs. Hence ET_C is often equivalently taken as the ***consumptive irrigation requirement*** (CIR).

The ***net irrigation requirement*** (NIR) is defined as the amount of irrigation water required to be delivered in the field to meet the consumptive requirement of crop as well as other needs such as leaching, ***pre-sowing*** and ***nursery water requirement*** (if any). Thus,

$$NIR = CIR + LR + PSR + NWR \qquad (8)$$

Where

LR = Leaching requirement
PSR = Pre-sowing requirement
NWR = Nursery water requirement

Field Irrigation Requirement (FIR) is defined as the amount of water required to meet the net irrigation requirements plus the amount of water lost as surface runoff and through deep percolation. Considering a factor η_a called the water application efficiency or the field application efficiency which accounts for the loss of irrigation water during its application over the field, we have

$$\text{FIR} = \frac{\text{NIR}}{\eta_a} \qquad (9)$$

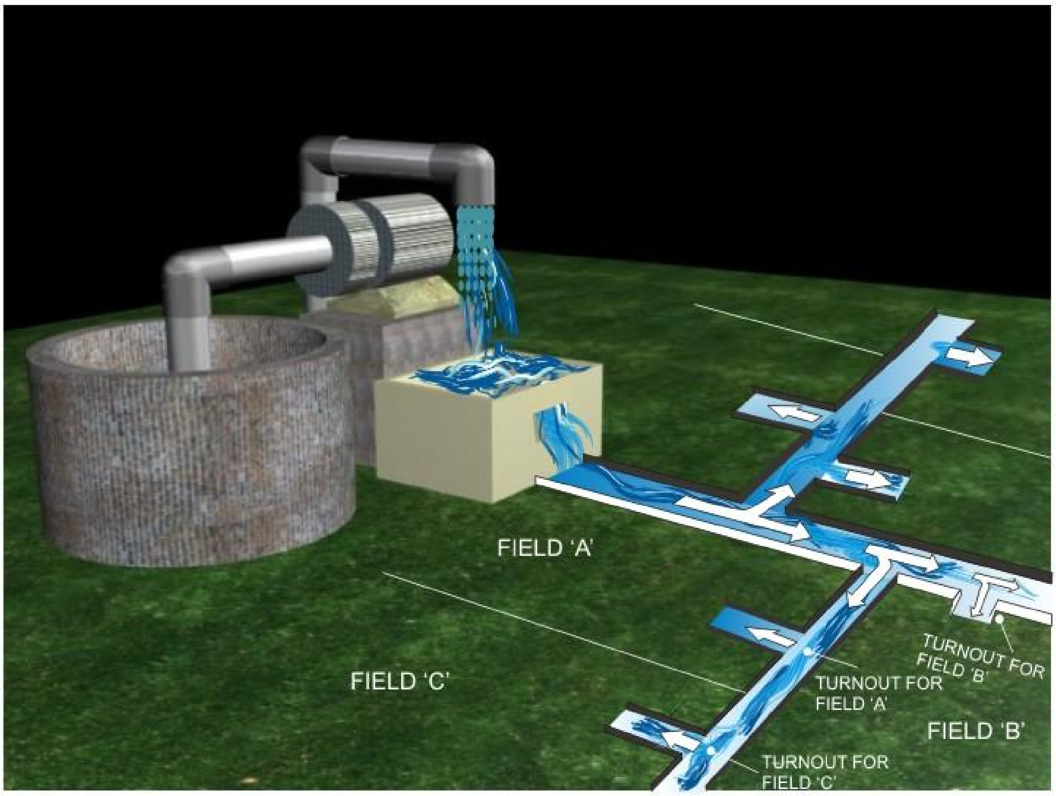


FIGURE 8. A typical ground water source irrigating a number of fields

Now, consider an irrigated area where there is a single source of water (say, a ground water pump) is supplying water to a number of fields and water is applied to each field by rotation (Figure 8). Naturally, some water is lost through the respective turnouts. Hence, the source must supply a larger amount of water than that required at any point of time by adding up the flows to the fields turnouts that are open at that point of time. Thus, the capacity of the water supply source may be termed as the ***gross irrigation requirement*** (GIR), defined as:

$$\text{GIR} = \frac{\text{FIR}}{\eta_C} \qquad (10)$$

In the above equation, η_c is the ***water conveyance efficiency***.

Figure 9 shows the factors that decide the overall irrigation efficiency.

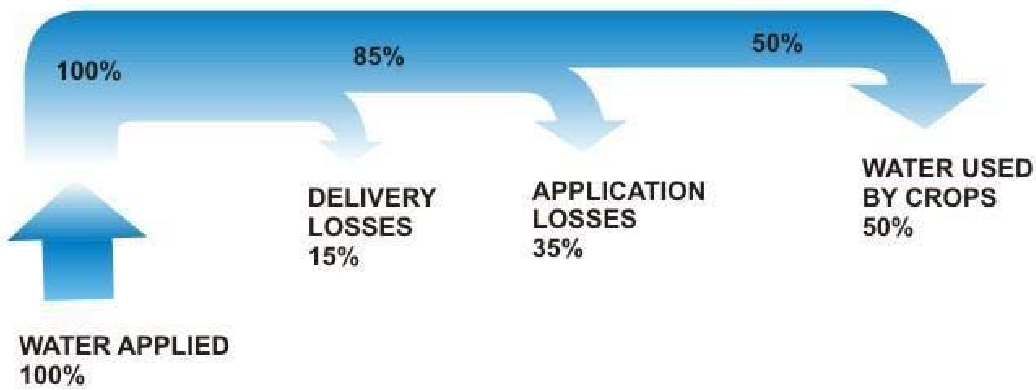


FIGURE 9. Typical values of losses in an irrigation system: factors affecting irrigation efficiency

3.3.11 Important terms related to crop water requirements and irrigation

Paleo irrigation

Sometimes, in the initial stages before the crop is sown, the land is very dry. This happens usually at the time of sowing Rabi crops because of hot September, when the soil may be too dry to be sown easily. In such a case, the soil is first moistured with water to help to sowing of seeds, and the water application for this purpose is known as Paleo Irrigation.

Kor watering

The total quality of water required by a crop is applied through a number of waterings at certain intervals throughout the base period of the crop. However, the quantity of water required to be applied during each of these waterings is not the same. In general, for all crops during the first watering after the plants have grown a few centimeters high, the quantity of water required is more than that during subsequent waterings. The first watering after the plants have grown a few centimeters high is known as Kor watering and the depth of water applied during watering is known as Kor depth. The watering must be done in a limited period which is known as the Kor period.

Outlet factor

The duty of water at the outlet that is at the turnout leading from the water courses and field channels on the field is known as the outlet factor.

Overlap allowance

It might happen that the crop of one season may sometimes overlap the next crop season for some period. During such a period of overlapping, irrigation water is required to be supplied simultaneously to the crops of both the seasons. Thus there is extra demand of water during this period and thus the water supply must be increased by some amount. The extra discharge that has to be supplied for this purpose is known as Overlap allowance.

The Evaporation Pan

A shallow edged container used to hold water during observations for the determination of the quantity of evaporation at a given location. The U.S. Weather Bureau Class A pan is 4 feet in diameter, 10 inches deep, set up on a timber grillage so that the top rim is about 16 inches from the ground. The water level in the pan during the course of observation is maintained between 2 and 3 inches below the rim.

Module 3

Irrigation Engineering Principles

Lesson 4

Types of Irrigation Schemes and Methods of Field Water Application

Instructional objectives

On completion of this lesson, the students come to know of the following:

1. The classification of irrigation projects in India
2. Direct and Storage methods of irrigation
3. Structures necessary for implementing irrigation projects
4. The methods employed for application of water to irrigate fields
5. Surface and subsurface methods of irrigation
6. Drip and Sprinkler irrigation systems

3.4.0 Introduction

The last lesson (3.3) discussed about the water requirement of crops as well as the total water required at the entrance to a field of crops. It also gives an idea of the water that is required to be supplied at the head of a channel that is conveying water to more than one field. Infact, water supply to such a cluster of fields with perhaps many sowing different crops is what is termed as an “irrigation scheme”. In the first part of this lesson, we shall be discussing about the ways in which various water sources may be utilized to meet the demand of irrigation or the gross irrigation requirement.

In the later portion of this section, different procedures adopted for applying the water to the field of crops is discussed.

3.4.1 Classification of irrigation projects

Irrigation projects are classified in different ways, however, in Indian context it is usually classified as follows:

- **Major project:** This type of project consists of huge surface water, storage reservoirs and flow diversion structures. The area envisaged to be covered under irrigation is of the order over 10000 hectare.

- **Medium project:** These are also surface water projects but with medium size storage and diversion structures with the area under irrigation between 10000 hectare and 2000 hectare.

- **Minor project:** The area proposed under irrigation for these schemes is below 2000Ha and the source of water is either ground water or from wells or tube wells or surface water lifted by pumps or by gravity flow from tanks. It could also be irrigated from through water from tanks.

The major and medium irrigation projects are further classified as

- Direct irrigation method
- Storage irrigation method.

Each of the two classifications is explained in subsequent sections. But before that, it may be worthwhile to discuss here a few terms related to irrigation projects which may also be called irrigation schemes.

Commanded area (CA): is defined as the area that can be irrigated by a canal system, the CA may further be classified as under:

Gross command area (GCA): This is defined as total area that can be irrigated by a canal system on the perception that unlimited quantity of water is available. It is the total area that may theoretically be served by the irrigation system. But this may include inhibited areas, roads, ponds, uncultivable areas etc which would not be irrigated.

Culturable command area (CCA): This is the actually irrigated area within the GCA. However, the entire CCA is never put under cultivation during any crop season due to the following reasons:

- The required quantity of water, fertilizer, etc. may not be available to cultivate the entire CCA at a particular point of time. Thus, this is a physical constraint.
- The land may be kept fallow that is without cultivation for one or more crop seasons to increase the fertility of the soil. This is a cultural decision.
- Due to high water table in some areas of the CCA irrigated water may not be applied as the crops get enough water from the saturation provide to the surface water table.

During any crop season, only a part of the CCA is put under cultivation and this area is termed as ***culturable cultivated area***. The remaining area which is not cultivated during a crop season is conversely termed as ***culturable uncultivated area***.

Intensity of irrigation is defined as the percentage of the irrigation proposed to be irrigated annually. Usually the areas irrigated during each crop season (Rabi, Kharif, etc) is expressed as a percentage of the CCA which represents the intensity of irrigation for the crop season. By adding the intensities of irrigation for all crop seasons the yearly intensity of irrigation to be obtained.

As such, the projects with a CCA of more than 2000 hectare are grouped as ***major*** and ***medium*** irrigation projects. The ultimate irrigation potential of our country from major and medium projects has been assessed as 58.46 M-hectare.

As per the report of the Ministry of Water Resources, Government of India the plan wise progress of irrigation of creation of irrigation potential through major and medium projects is as follows.

The following table (from the Planning Commission's report) provides a list of the Major and Medium irrigation state-wise creation and utilisation of irrigation potential at the end of VIII plan (1992-97). It may be noted that the information contained in the table does not separate out the newly formed states which were created after 1997.

Areas given in thousand hectares

Sl. No.	State Name	Potential created upto the end of VIII Plan (1992-97) *	Potential utilised upto the end of VIII Plan (1992-97) *
1.	Andhra Pradesh	3045.10	2883.80
2.	Arunachal Pradesh	-	-
3.	Assam	196.67	138.17
4.	Bihar	2802.50	2324.20
5.	Goa	13.02	12.07
6.	Gujarat	1350.00	1200.00
7.	Haryana	2078.79	1833.62
8.	Himachal Pradesh	10.55	5.59
9.	Jammu & Kashmir	173.70	147.57
10.	Karnataka	1666.02	1471.70
11.	Kerala	513.31	464.31
12.	Madhya Pradesh	2317.60	1620.95
13.	Maharashtra	2313.00	1287.70
14.	Manipur	63.00	52.00
15.	Meghalaya	-	-
16.	Mizoram	-	-
17.	Nagaland	-	-
18.	Orissa	1557.75	1442.66
19.	Punjab	2512.85	2452.34
20.	Rajasthan	2273.88	2088.39
21.	Sikkim	-	-
22.	Tamil Nadu	1545.51	1545.49
23.	Tripura	2.30	2.30
24.	Uttar Pradesh	7059.00	6126.00
25.	West Bengal	1444.08	1332.52
Total - States		32938.63	28431.38
Total - UT		18.51	9.29
Grand Total		32957.14	28440.67

* Provisional

As for the minor irrigation schemes mostly using ground water sources are primarily developed through individual and cooperative efforts of the farmers with the help of institutional finance and their own savings. Surface water minor irrigation schemes (like lifting water by pumps from rivers) are generally funded from the public sector outlay. The ultimate irrigation potential from minor irrigation schemes has been assessed as 81.43hectare. The development of minor irrigation should receive greater attention because of the several advantages they possess like small investments, simpler components has also being labour incentive quick money and most of all farmers friendly.

The importance of irrigation in the Indian agricultural economy be appreciated at a glance of the following table showing state wise details of net sown areas and the area that is irrigated (net irrigated areas) for states like Punjab the area irrigated is more than 90 percent followed by Haryana (77 percent) and Uttar Pradesh (67 percent). The national average is low, at around 38 percent.

Areas given in thousand hectares

Sl. No.	State Name	Net Sown Area (NSA)	Net Irrigated Area (NIA)	% of NIA to NSA
1.	Andhra Pradesh	10637	4123	38.76
2.	Arunachal Pradesh	185	36	19.46
3.	Assam	2780	572	20.57
4.	Bihar	7321	3680	50.27
5.	Goa	139	23	16.55
6.	Gujarat	9609	3002	31.24
7.	Haryana	2586	2761	76.99
8.	Himachal Pradesh	568	101	17.78
9.	Jammu & Kashmir	734	386	52.59
10.	Karnataka	10420	2302	22.09
11.	Kerala	2265	342	15.10
12.	Madhya Pradesh	19752	5928	30.01
13.	Maharashtra	17911	2567	14.33
14.	Manipur	140	65	46.43
15.	Meghalaya	206	45	21.84
16.	Mizoram	109	7	6.42
17.	Nagaland	211	62	29.38

18.	Orissa	6210	2090	33.65
19.	Punjab	4139	3847	92.94
20.	Rajasthan	16575	5232	31.56
21.	Sikkim	95	16	16.84
22.	Tamil Nadu	5342	2625	49.14
23.	Tripura	277	35	12.64
24.	Uttar Pradesh	17399	11675	67.10
25.	West Bengal	5462	1911	34.99
Total - States		142072	53433	37.61
Total - UT		143	75	52.45
Grand Total		142215	53508	37.62

3.4.2 Direct and Indirect (Or Storage) Irrigation Methods

The major and medium surface water schemes are usually classified as either direct or indirect irrigation projects and these are defined as follows:

3.4.2.1 *Direct Irrigation method*

In this project water is directly diverted from the river into the canal by constructing a diversion structure like weir or barrage across the river with some pondage to take care of diurnal variations. It also effects in raising the river water level which is then able to flow into the offtaking channel by gravity. The flow in the channel is usually controlled by a gated structure and this in combination with the diversion structure is also sometimes called the headworks.

If the water from such headworks is available throughout the period of growth of crops irrigated by it, it is called a perennial irrigation scheme. In this type of projects, the water in the offtaking channels from the river carries water through out the year. It may not be necessary, however, to provide irrigation water to the fields during monsoon. In some places local rainfall would be sufficient to meet the plant water needs. In case of a non-perennial river the offtaking channel would be carrying water only for certain period in a year depending upon the availability of supply from the source.

Another form of direct irrigation is the inundation irrigation which may be called river-canal irrigation. In this type of irrigation there is no irrigation work across the river to control the level of water in the river. Inundation canal off-taking from a river is a seasonal canal which conveys water as and when available in the river. This type of direct irrigation is usually practiced in deltaic tract that is, in areas having even and plane topography. It is feasible when the normal flow of river or stream throughout the period of growth of crop irrigated, is never less than the requirements of the irrigated crops at any time of the base period. A direct irrigation scheme of irrigation using river water diversion head works typically be laid out as in Figure 1.

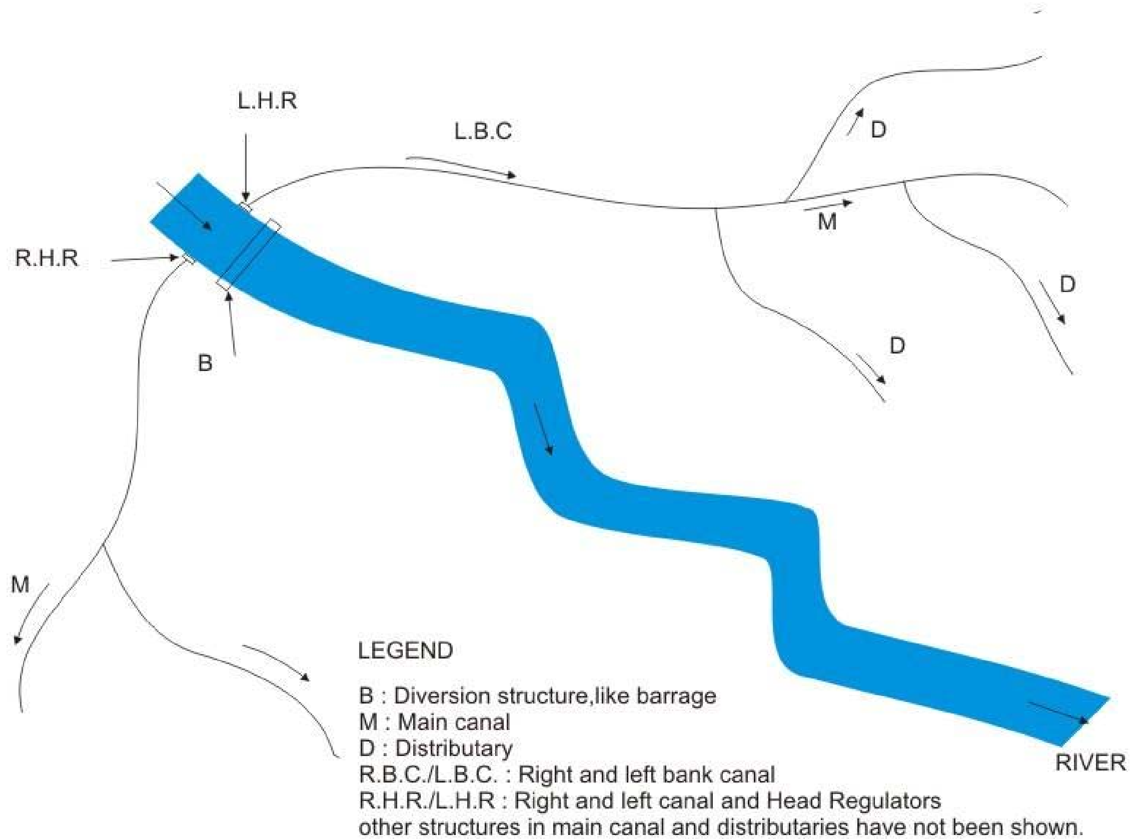


FIGURE 1. An example of a Direct Irrigation scheme

Though the diversion structure raises the river water level and is just sufficient to force some water into the channel, the stored water in the pond created behind doesn't have sufficient storage volume it may however be able to take care of any diurnal variation in the river water. An example of this scheme is the DVC irrigation project on the Damodar river with the barrage located at Durgapur.

3.4.2.2 *Storage Irrigation Method*

For this type of irrigation schemes part of the excess water of a river during monsoon which other wise would have passed down the river as a flood is stored in a reservoir or tank found at the upstream of a dam constructed across a river or stream. This stored water is then used for irrigation is adopted when the flow of river or stream is in excess of the requirements of irrigated crops during a certain part of the year but falls below requirements or is not available at all in the river during remaining part of the year. Since the construction site of a storage reservoir is possible in regions of undulating topography, it is usually practiced in non deltaic areas. A general layout of this irrigation scheme may typically be laid out as shown in Figure 2.

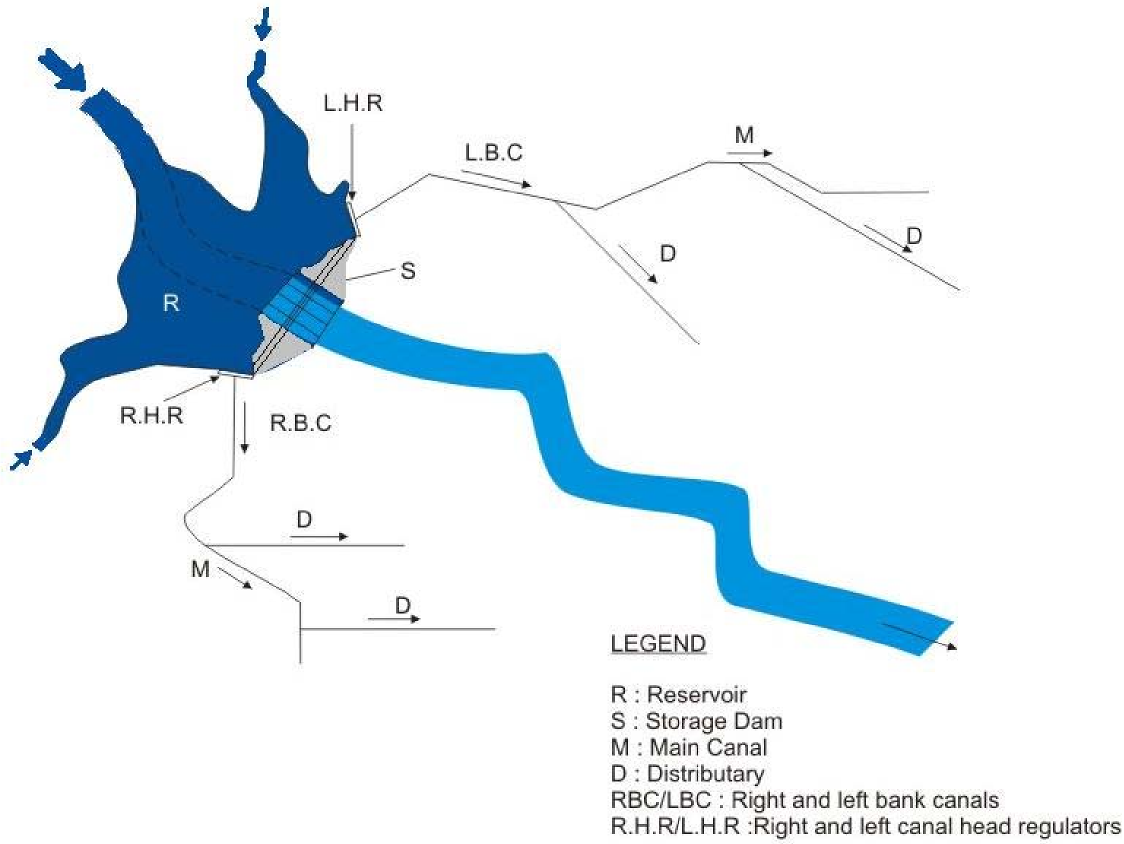


FIGURE 2. A typical layout for a storage irrigation scheme incorporating a dam

In third type of scheme the storage head works or the dams has to be equipped with ancillary structure like outlet, sluice, spillway, log chutes, etc. The storage created by the dam behind the reservoir is substantial compared to that behind a barrage and may inundate a large tract of land, depending on the topography. The capacity of the reservoir is generally determined systematically by knowing possible withdrawal demands (in this case for irrigation) over the weeks and months of a year and corresponding expected inflows.

An example for this type of scheme is the Indira Sagar project on the Narmada River. Of course, apart from serving irrigation demand the project also generates electricity. Hence it is actually a multi purpose project.

Another type of storage irrigation method envisages the storage of water at some place in the hilly terrain of the river where the construction of the dam is possible. A barrage is constructed at some downstream location, where the terrain is flatter and canals take off as in a usual direct irrigation method. A general layout of such scheme could be represented as in Figure 3.

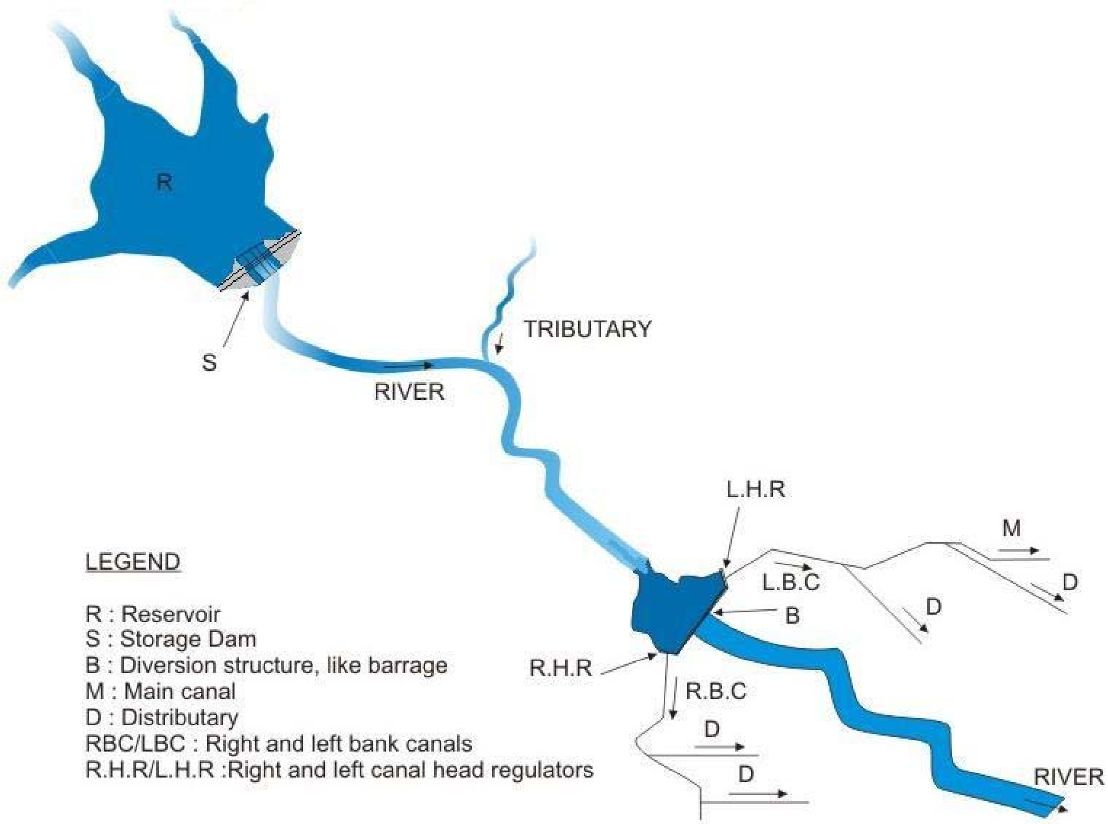


FIGURE 3. A typical layout of a storage irrigation scheme incorporating a dam with a barrage on its downstream

An example for this type of scheme is the Bhakra-dam Nangal-barrage combination on the river Sutlej.

3.4.3 Irrigation Project Structures

As might have been noticed from the irrigation scheme plans in the previous section, a number of structures are required for the successful implementation of a project. Some of these are:

Storage structure and appurtenant works

- ***Dams***
- ***Spillways and energy dissipators***
- ***Sluices and outlets***

Diversion structure and appurtenant works

- ***Barrage*** (weirs are not commonly used these days for sizeable projects)

- ***Canal head regulator***
- ***River training works***

Canal water conveyance structures

- ***Canal sections and layout***
- ***Cross regulators***
- ***Drops***
- ***Turnouts***

Also, for an irrigation scheme to be successful, it is important that only the right amount of water be applied to the fields at any point of time. Hence, excess water carried by the canals, as during rainy season, needs to be removed through a drainage network of channels and returned back to the river. Construction of such drainage channels form an important part of a properly designed irrigation network.

All the above structures are discussed in detail in the subsequent lessons. However, ultimately aim of irrigation planning is to distribute the water meant for irrigation that is stored in storage reservoirs, diverted through diversion structures, conveyed through canal network and discharged to the fields through turnouts to be properly distributed throughout the field. Depending on the type of crop to be grown, the terrain topography and soil characteristics, climate and other local factors different ways and means have been evolved for field water application. These methods of irrigation are discussed in subsequent sections.

3.4.4 Methods of Field Water Application

Irrigation water conveyed to the head or upstream point of a field must be applied efficiently on the whole area such that the crops growing in the either fields gets water more or less uniformly.

Naturally it may be observed that a lot depends on the topography of the land since a large area with uneven topography would result in the water spreading to the low lying areas. The type of crop grown also immensely matter as some like rice, require standing water depths at almost all stages of its growth. Some, like potato, on the other hand, suffer under excess water conditions and require only the right amount of water to be applied at the right time. Another important factor determining the way water is to apply in the fields is the quantity of water available at any point of time. If water is scarce, as what is actually happening in many parts of the country, then it is to be applied through carefully controlled methods with minimum amount of wastage. Usually these methods employ pressurized flow through pipes which is either sprinkled over the crop or applied carefully near the plant root. On the other hand when water is rather unlimited during the crop growing season as in deltaic regions, the river flood water is allowed to inundate as much area as possible as long the excess water is available. Another important parameter dictating the choice of the irrigation method is the type of soil. Sometimes water is applied not on the surface of the field but is used to moist the root zone of the plants from beneath the soil surface. Thus, in effective the type of irrigation methods can be broadly divided as under:

- Surface irrigation method
- Subsurface irrigation method
- Sprinkler irrigation system
- Drip irrigation system

Each of these methods has been discussed in the subsequent section of this lesson.

3.4.5 Surface Irrigation Methods

In this system of field water application the water is applied directly to the soil from a channel located at the upper reach of the field. It is essential in these methods to construct designed water distribution systems to provide adequate control of water to the fields and proper land preparation to permit uniform distribution of water over the field.

One of the surface irrigation method is ***flooding method*** where the water is allowed to cover the surface of land in a continuous sheet of water with the depth of applied water just sufficient to allow the field to absorb the right amount of water needed to raise the soil moisture up to field capacity,. A properly designed size of irrigation stream aims at proper balance against the intake rate of soil, the total depth of water to be stored in the root zone and the area to be covered giving a reasonably uniform saturation of soil over the entire field.

Flooding method has been used in India for generations without any control what so ever and is called uncontrolled flooding. The water is made to enter the fields bordering rivers during folds. When the flood water inundates the flood plane areas, the water distribution is quite uneven, hence not very efficient, as a lot of water is likely to be wasted as well as soils of excessive slopes are prone to erosion. However the adaptation of this method doesn't cost much.

The flooding method applied in a controlled way is used in two types of irrigation methods as under:

- ***Border irrigation method***
- ***Basin irrigation method***

As the names suggest the water applied to the fields by this inundates or floods the land, even if temporarily. On the other hand there are many crops which would try better if water is applied only near their root zone instead of inundating.

Such an irrigation method is called the Furrow irrigation method. All these methods are discussed in the subsequent sections.

3.4.5 Border irrigation

Borders are usually long uniformly graded strips of land separated by earth bunds (low ridges) as shown in Figure 4.

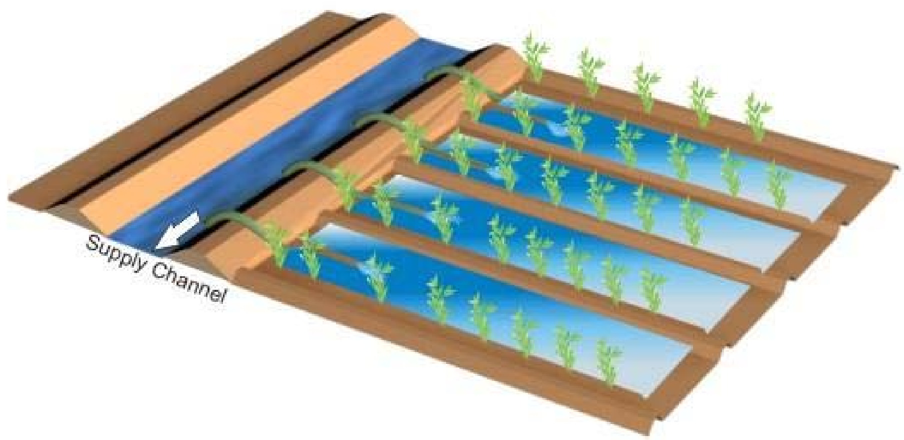


FIGURE 4. Border irrigation with water being applied to the borders with the help of flexible pipes, acting as siphons

The essential feature of the border irrigation is to provide an even surface over which the water can flow down the slope with a nearly uniform depth. Each strip is irrigated independently by turning in a stream of water at the upper end as shown in Figure 5.

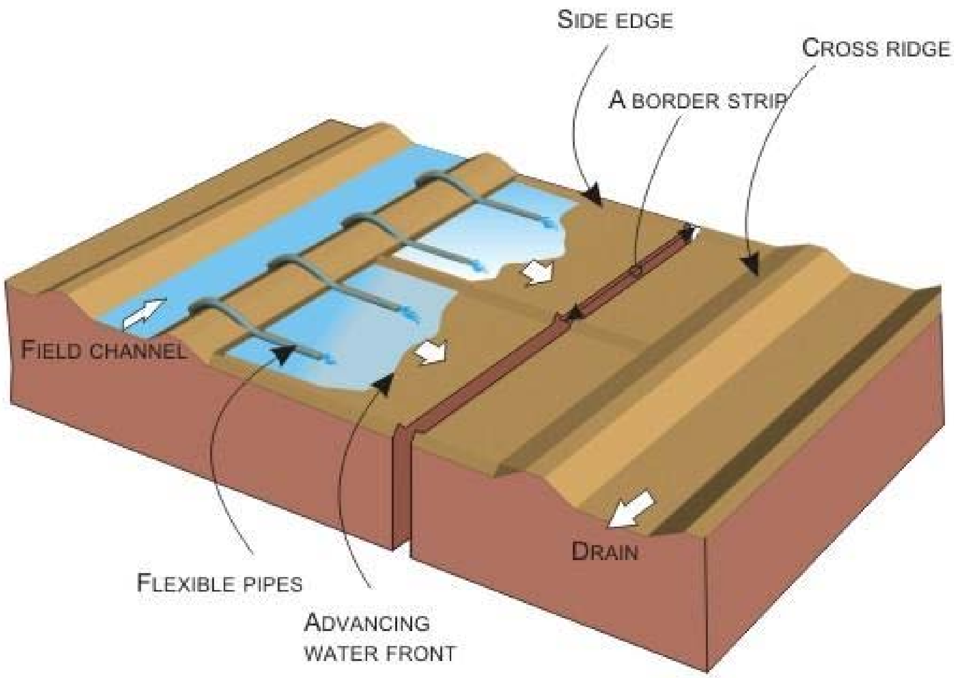


FIGURE 5. Water entering each border strip independently

The water spreads and flow down the strip in a sheet confined by border ridges. When the advancing water reaches the lower end of the border, the stream is turned off.

For uniform advancement of water front the borders must be properly leveled. The border shown in the figures above are called ***straight borders***, in which the border strips are laid along the direction of general slope of the field. The borders are sometimes laid along the elevation contours of the topography when the land slope is excessive. Thos method of border is called ***contour border method*** of irrigation (Figure 6).

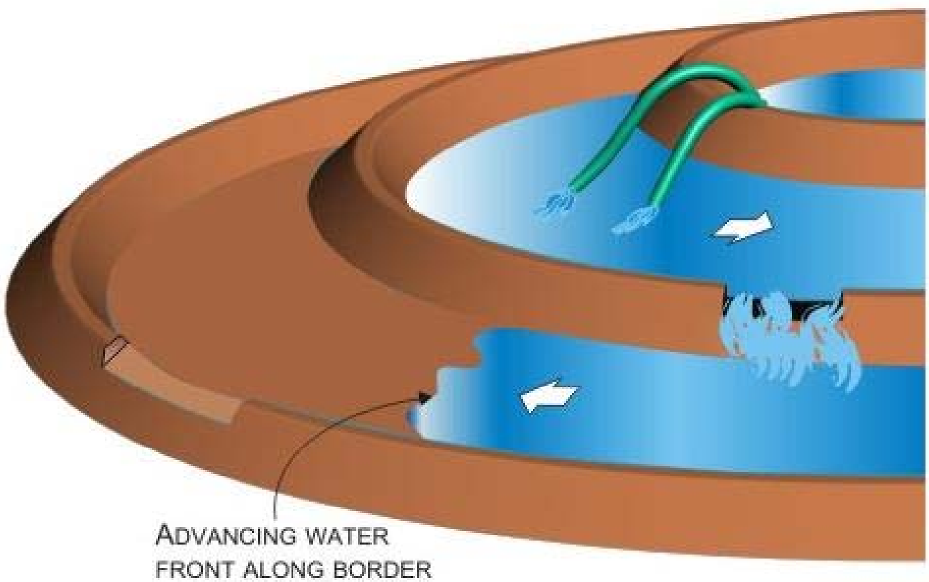


FIGURE 6. Contour border method of irrigation

The straight border irrigation is generally suited to the larger mechanized farms as it is designed to produce long uninterrupted field lengths for ease of machine operations. Borders can be 800m or more in length and 3 – 30 m wide depending on variety of factors. It is less suited to small scale farms involving hand labour or animal powered cultivation methods.

Generally, border slopes should be uniform, with a minimum slope of 0.05% to provide adequate drainage and a maximum slope of 2% to limit problems of soil erosion.

As for the type of soil suitable for border irrigation, deep homogeneous loam or clay soils with medium infiltration rates are preferred. Heavy, clay soils can be difficult to irrigate with border irrigation because of the time needed to infiltrate sufficient water into the soil. Basin irrigation is preferable in such circumstances.

3.4.6 Basin Irrigation

Basins are flat areas of land surrounded by low bunds. The bunds prevent the water from flowing to the adjacent fields. The basins are filled to desired depth and the water is retained until it infiltrates into the soil. Water may be maintained for considerable periods of time.

Basin method of irrigation can be formally divided into two, viz; the c***heck basin method*** and the ***ring basin method***. The check basin method is the most common method of irrigation used in India. In this method, the land to be irrigated is divided into small plots or basins surrounded by checks, levees (low bunds); as shown in Figure 7.

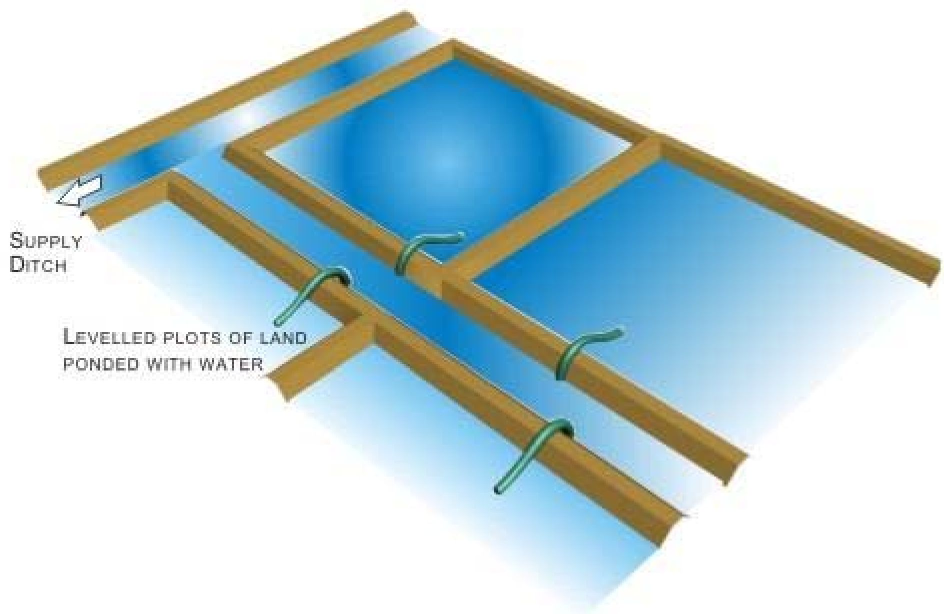


FIGURE 7. Check basin method of irrigation

Each plot or basin has a nearly level surface. The irrigation water is applied by filling the plots with water up to the desired depth without overtopping the levees and the water retained there is allowed to infiltrate into the soil. The levees may be constructed for temporary use or may be semi permanent for repeated use as for paddy cultivation. The size of the levees depends on the depths of water to be impounded as on the stability of the soil when wet.

Water is conveyed to the cluster of check basins by a system of supply channels and lateral field channels or ditches. The supply channel is aligned on the upper side (at a higher elevation) of the field for every two rows of plot as shown in the figure.

The size of basins depends not only on the slope but also on the soil type and the available water flow to the basins. Generally, it is found that the following holds good for basin sizes.

Basin size should be small if the

1. Slope of the land is steep.
2. Soil is sandy.
3. Stream size to basin is small.
4. Required depth of irrigation application is small.
5. Field preparation is done by hand or animal traction

Basin size can be large if the

1. Slope of the land is flat
2. Soil is clay.
3. Stream size to the basin is large
4. Required depth of the irrigation is large.
5. Field preparation is mechanized.

Basin irrigation is suitable for many field crops. Paddy rice grows best when its roots are submerged in water and so basin irrigation is the best method for use with the crop.

The other form of basin irrigation is the ring basin method which is used for growing trees in orchards. In this method, generally for each tree, a separate basin is made which is usually circular in shape, as shown in Figure 8.

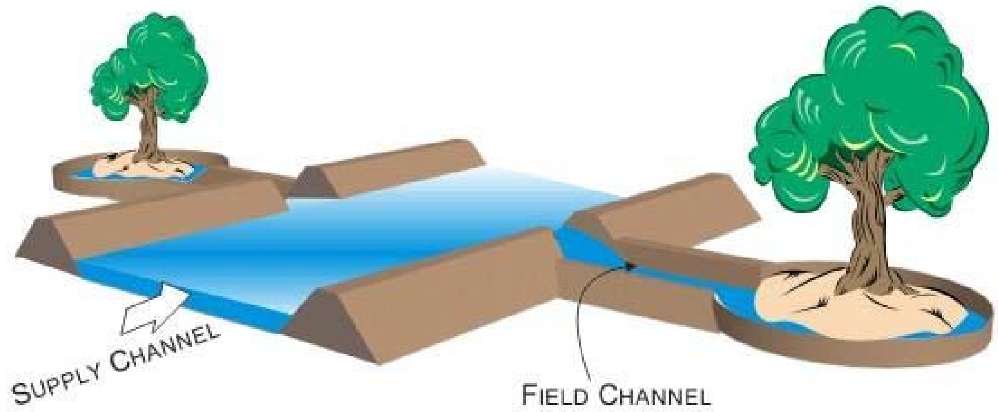


FIGURE 8. Ring basin method of irrigation

Sometimes, basin sizes are made larger to include two more trees in one basin. Water to the basins is supplied from a supply channel through small field channels conveyed the basins with the supply channel.

Trees which can be irrigated successfully using the ring basin method include citrus and banana.

Basins can also be constructed on hillside. Here, the ridges of the basins are constructed as in contour border method thus making the only difference between the two is in the application of water. In the border method, the water is applied once during an irrigation cycle and is allowed to flow along the field and as the water infiltrates, till the supply is cutoff. In the basin method, as in a rice field the water is higher at a desired level on the basin. Basin irrigation is suitable for many field crops. Paddy rice grows best when its roots are submerged in water and so basin irrigation is the best method for use with this crop.

3.4.7 Furrow Irrigation

Furrows are small channels, which carry water down the land slope between the crop rows. Water infiltrates into the soil as it moves along the slope. The crop is usually grown on ridges between the furrows, as shown in Figure 9. This method is suitable for all row crops and for crops that cannot stand water for long periods, like 12 to 24 hours, as is generally encountered in the border or basin methods of irrigation.

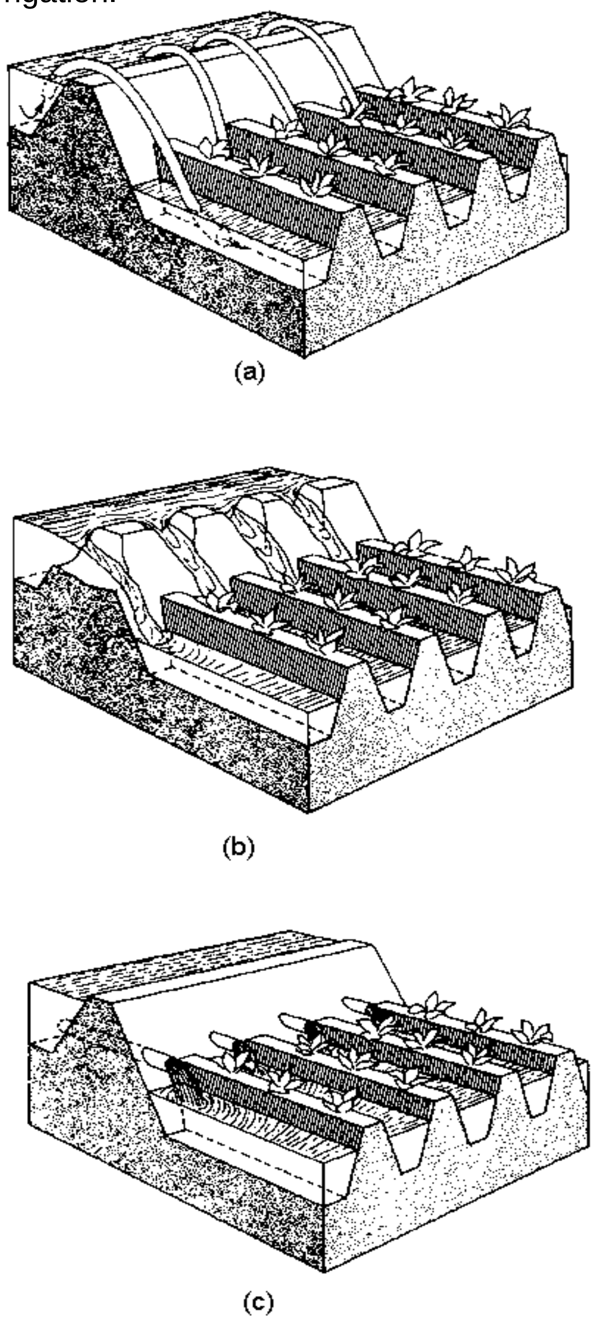


FIGURE 9. Furow irrigation method of applying water to a field
(a) Using flexible pipes to siphon out water from field channel
(b) Using the breach method to apply water to the furrows
(c) Pipe outlets to deliver water to the furrows

(Image courtesy: Food and Agriculture Organisation, FAO)

Water is applied to the furrows by letting in water from the supply channel, either by pipe siphons or by making temporary breaches in the supply channel embankment. The length of time the water is to flow in the furrows depends on the amount of water required to replenish the root zone and the infiltration rate of the soil and the rate of lateral spread of water in the soil.

Furrow irrigation is suitable to most soils except sandy soils that have very high infiltration water and provide poor lateral distribution water between furrows. As compared to the other methods of surface irrigation, the furrow method is advantageous as:

- Water in the furrows contacts only one half to one-fifth of the land surface, thus reducing puddling and clustering of soils and excessive evaporation of water.
- Earlier cultivation is possible

Furrows may be straight laid along the land slope, if the slope of the land is small (about 5 percent) for lands with larger slopes, the furrows can be laid along the contours.

3.4.8 Subsurface irrigation methods

As suggested by the name, the application of water to fields in this type of irrigation system is below the ground surface so that it is supplied directly to the root zone of the plants. The main advantages of these types of irrigation is reduction of evaporation losses and less hindrance to cultivation works which takes place on the surface.

There may be two ways by which irrigation water may be applied below ground and these are termed as:

- Natural sub-surface irrigation method
- Artificial sub-surface irrigation method

These methods are discussed further below

3.4.8.1 Natural Sub-surface irrigation method

Under favorable conditions of topography and soil conditions, the water table may be close enough to the root zone of the field of crops which gets its moisture due to the upward capillary movement of water from the water table. The natural presence of the water table may not be able to supply the requisite water throughout the crop growing season. However, it may be done artificially by constructing deep channels in the field which may be filled with water at all times to ensure the presence of water table at a desired elevation below the root zone depth. Though this method of irrigation is excellent from both water distribution and labour saving points of view, it is favorable mostly for the following

- The soil in the root zone should be quite permeable
- There should be an impermeable substratum below the water table to prevent deep percolation of water.
- There must be abundant supply of quality water that is one which is salt free, otherwise there are chances of upward movement of these salts along with the moisture likely to lead the conditions of salt incrustation on the surface.

3.4.8.2 Artificial subsurface irrigation method

The concept of maintaining a suitable water table just below the root zone is obtained by providing perforated pipes laid in a network pattern below the soil surface at a desired depth. This method of irrigation will function only if the soil in the root zone has high horizontal permeability to permit free lateral movement of water and low vertical permeability to prevent deep percolation of water. For uniform distribution of water percolating into the soil, the pipes are required to be very closely spaced, say at about 0.5m. Further, in order to avoid interference with cultivation the pipes have to be buried not less than about 0.4m below the ground surface. This method of irrigation is not very popular because of the high expenses involved, unsuitable distribution of subsurface moisture in may cases, and possibility of clogging of the perforation of the pipes.

3.4.9 Sprinkler Irrigation System

Sprinkler irrigation is a method of applying water which is similar to natural rainfall but spread uniformly over the land surface just when needed and at a rate less than the infiltration rate of the soil so as to avoid surface runoff from irrigation. This is achieved by distributing water through a system of pipes usually by pumping which is then sprayed into the air through sprinklers so that it breaks up into small water drops which fall to the ground. The system of irrigation is suitable for undulating lands, with poor water availability, sandy or shallow soils, or where uniform application of water is desired. No land leveling is required as with the surface irrigation methods. Sprinklers are, however, not suitable for soils which easily form a crust. The water that is pumped through the pump pipe sprinkler system must be free of suspended sediments. As otherwise there would be chances of blockage of the sprinkler nozzles.

A typical sprinkler irrigation system consists of the following components:

- ***Pump unit***
- ***Mainline and sometimes sub mainlines***
- ***Laterals***

- ***Sprinklers***

Figure 10 shows a typical layout of a sprinkler irrigation system.

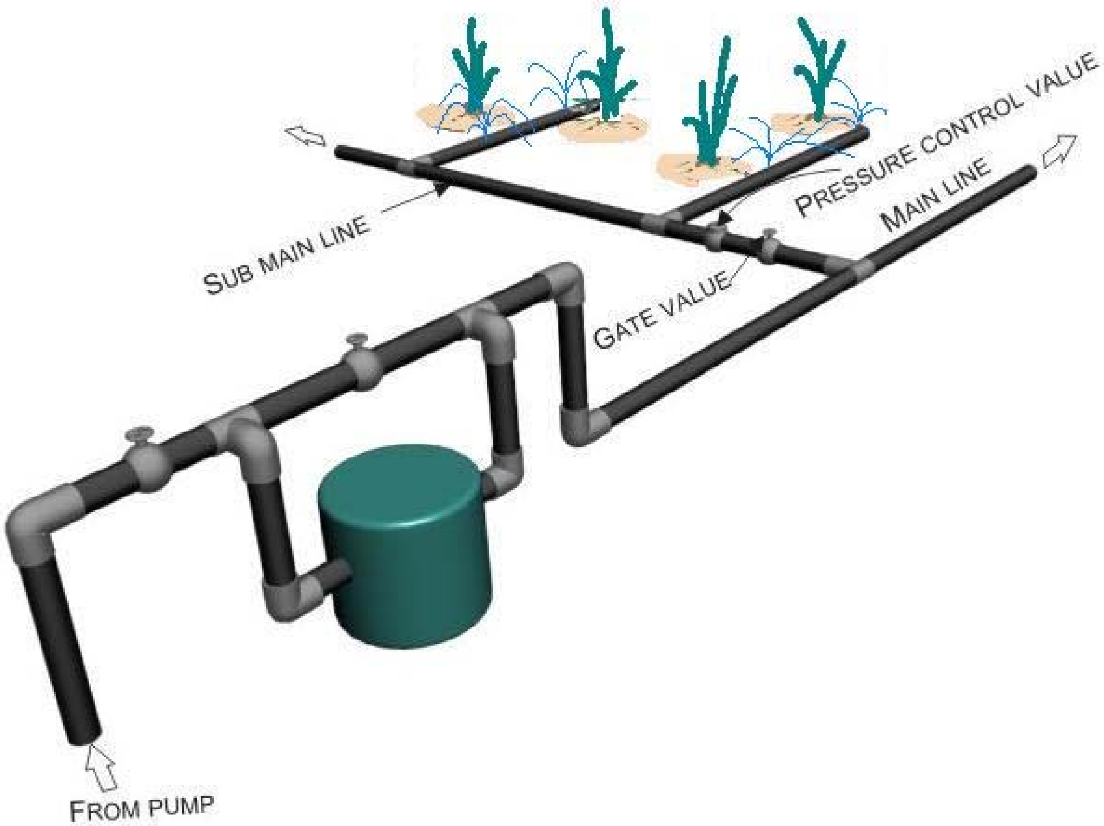


FIGURE 10. The sprinkler irrigation system

The pump unit is usually a centrifugal pump which takes water from a source and provides adequate pressure for delivery into the pipe system.

The mainline and sub mainlines are pipes which deliver water from the pump to the laterals. In some cases, these pipelines are permanent and are laid on the soil surface or buried below ground. In other cases, they are temporary, and can be moved from field to field. The main pipe materials include asbestos cement, plastic or aluminum alloy.

The laterals deliver water from the mainlines or sub mainlines to the sprinklers. They can be permanent but more often they are portable and made of aluminium alloy or plastic so that they can be moved easily.

The most common types of sprinklers that are used are:

- **Perforated pipe system:** This consists of holes perforated in the lateral irrigation pipes in specially designed pattern to distribute water fairly uniformly (Figure 11). The sprays emanating from the perforations are directed in both sided of the pipe and can cover a strip of land 6 m to 15m wide.

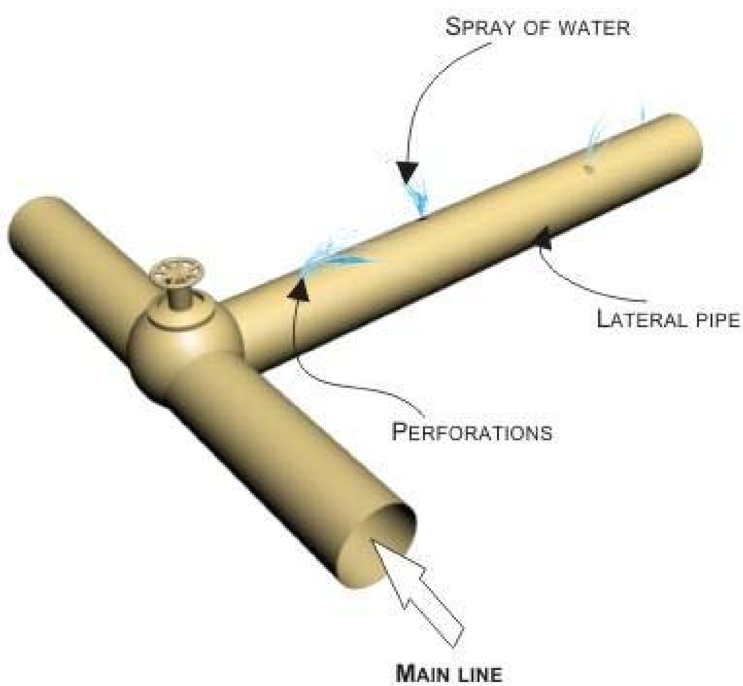


FIGURE 11. Perforated pipe type of sprinkler system

- **Rotating head system:** Here small sized nozzles are placed on riser pipes fixed at uniform intervals along the length of the lateral pipe (Figure 12). The lateral pipes are usually laid on the ground surface. The nozzle of the sprinkler rotates due to a small mechanical arrangement which utilizes the thrust of the issuing water.

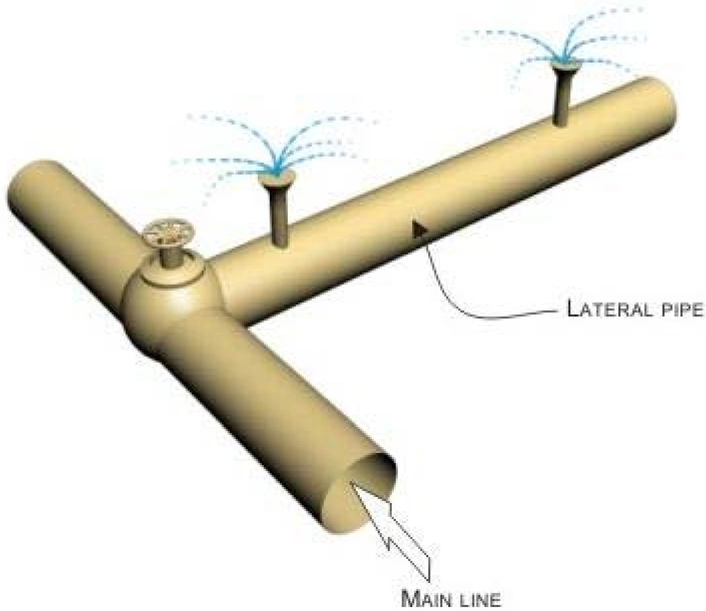


FIGURE 12. Rotating head system of sprinkler irrigation

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

Sprinkler irrigation has high efficiency. It however, varies according to climatic conditions; 60% in warm climate; 70% in moderate climate and 80% in humid or cool climate.

Sprinkler irrigation was not widely used in India before the 1980. Although no statistics are available on the total area under sprinkler irrigation, more than 200000 sprinkler sets were sold between 1985 and 1996(with 65000 for 1995-96) according to the National Committee on the use of plastics in agriculture. The average growth rate of sprinkler irrigated area in India is about 25 percent. The cost of installation of sprinkler irrigation depends on a number of factors such as type of crop, the distance from water source.

3.4.10 Drip Irrigation System

Drip Irrigation system is sometimes called trickle irrigation and involves dripping water onto the soil at very low rates (2-20 litres per hour) from a system of small diameter plastic pipes filled with outlets called emitters or drippers. Water is applied close to the plants so that only part of the soil in which the roots grow is wetted, unlike surface and sprinkler irrigation, which involves wetting the whole soil profile. With drip irrigation water, applications are more frequent than with other methods and this provides a very favourable high moisture level in the soil in which plants can flourish. Figure 13 shows a typical layout of the drip irrigation system.

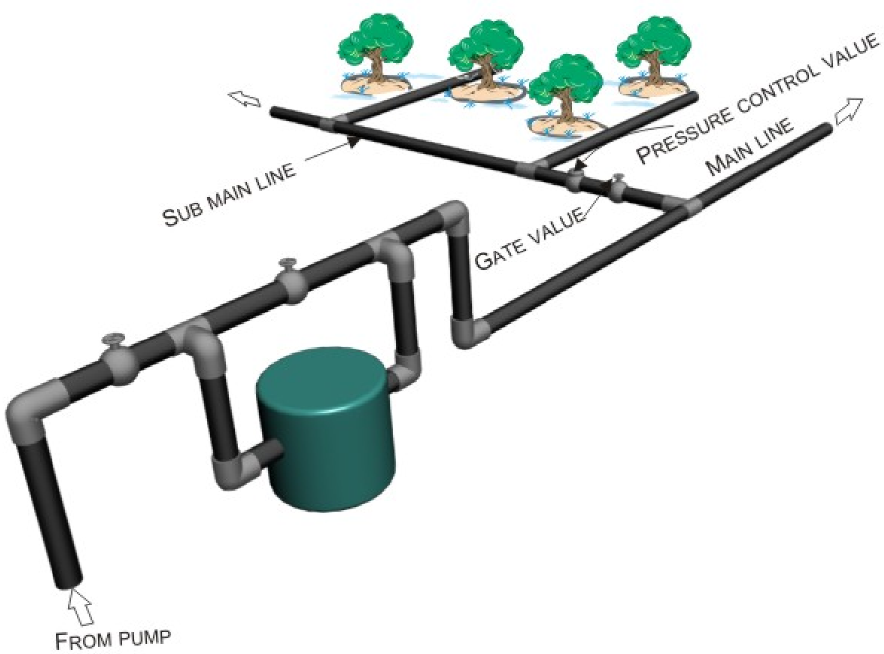


FIGURE 13. The typical layout of the drip irrigation system

A typical drip irrigation system consists of the following components:

- ***Pump unit***
- ***Control Head***
- ***Main and sub main lines***
- ***Laterals***
- ***Emitters and drippers***

The drip irrigation system is particularly suited to areas where water quality is marginal, land is steeply sloping or undulating and of poor quality, where water or labour are expensive, or where high value crops require frequent water applications. It is more economical for orchard crops than for other crops and vegetables since in the orchards plants as well as rows are widely spaced. Drip irrigation limits the water supplied for consumptive use of plants. By maintaining a minimum soil moisture in the root zone, thereby maximizing the water saving. A unique feature of drip irrigation is its excellent adaptability to saline water. Since the frequency of irrigation is quite high, the plant base always remains wet which keeps the salt concentration in the plant zone below the critical. Irrigation efficiency of a drip irrigation system is more than 90 percent.

Drip irrigation usage in India is expanding rapidly. There is even some Government subsidy to encourage its use. From about 1000 hectare in 1985, the area under drip irrigation increased to 70860 hectare in 1991, with the maximum developments taking place in the following states:

- Maharashtra (32924 hectare)
- Andhra Pradesh (11585 hectare)
- Karnataka (11412 hectare)

The drip irrigated crops are mainly used to irrigate orchards of which the following crops are important ones (according to a 1991 survey):

- Grapes (12000 hectare)
- Bananas (6500 hectare)
- Pomegranates (5440 hectare)
- Mangoes

Drip irrigation was also used to irrigate sugarcane (3900 hectare) and coconut (2600 hectare).

3.4.11 Important terms and definitions

Dam: A dam is a hydraulic structure constructed across a river to store water on its upstream side. It is an impervious or fairly impervious barrier put across a natural stream so that a reservoir is formed.

Spillways and energy dissipators: Spillway is a channel that carries excess water over or around a dam or other obstruction. An energy dissipator is a device that is used to convert concentrated storm water runoff to sheet flow and is constructed at the end of all storm sewers or channels that outfall into a buffer.

Sluice and Outlet: A sluice is an artificial channel for conducting water, with a valve or gate to regulate the flow. An outlet is a small structure which admits water from the distributing channel to a water course of field channel. Thus an outlet is a sort of head regulator for the field channel delivering water to the irrigation fields.

Barrage: An artificial obstruction placed in a river or water course to increase the depth of water.

Canal Head Regulator: Any structure constructed to regulate the discharge, full supply level or velocity in a canal is known as a regulator work. This is necessary for the efficient working and safety of an irrigation channel. A canal head regulator regulates the supplies of the offtaking channel and the present channel respectively. The head regulator is provided at the head of the distributary and controls the supply entering the distributary.

River Training Works: Various measures adopted on a river to direct and guide the river flow, to train and regulate the river bed or to increase the low water depth are called River Training works. The purpose of the river training is to stabilize the channel alomg a certain alignment.

Cross regulator: A regulator provided on the main channel at the downstream of the offtake to head up the water level and to enable the off-taking channel to draw the required supply is called a Cross Regulator.

Pump Unit: This takes water from the source and provides the right pressure for delivering into the pipe system.

The control head: This consists of valves to control the discharge and pressure in the entire system. It may also have filters to clear the water. Common types of filters include screen filters and graded some filters which remove fine material suspended in the water. Some control head units contain a fertilizer or nutrient tank. These slowly add a measured dose of fertilizer into the water during irrigation.

Mainlines, sub mains and laterals: These components supply water from the control head into the fields. They are normally made from PVC or Polyethylene Glucose and should be buried below ground because they easily degrade when exposed to direct solar radiation.

Emitters or drippers: These are devices used to control the discharge water from the lateral to the plants. They are usually spaced more than 1 meter apart with one or more emitters used for a single plant such as a tree. For row crops more closely spaced emitters may be used to wet a strip of soil. The aspect of an emitter that should be kept in mind to decide its efficiency is that it should provide a specified constant discharge which doesn't vary much with pressure change and doesn't block easily.

Module 3

Irrigation Engineering Principles

Lesson 5

Traditional Water Systems and Minor Irrigation Schemes

Instructional objectives

On completion of this lesson, the student shall learn about:

1. The indigenous systems of water utilisation in India
2. Region-specific traditional water storage systems in India

3.5.0 Introduction

Water is essential for the existence and survival of any life form. Hence human settlements the world over have thrived so long they had enough water for consumption and growing crops that yield food for community. The Indian subcontinent has witnessed many such early settlements that devised their own systems of water harvesting, which could be either from surface water or ground water. In the medieval age areas dominated by feudal system, some of these systems took more scientific form and surprisingly, many of these water extraction storage and conveyance structures are still in use in some parts of the country. In other parts of the country, especially in the hilly and mountainous terrains, the communities have devised certain means of water tapping and transfer systems that have been used for generations to provide water for drinking and agriculture. In fact, even today most of these hilly tracts do not enjoy the benefit of modern irrigation systems and still depend on their age old practices, as may be seen from the table below.

Net area irrigated by sources in Himalayan states (1988-89)
(All figures in thousand hectares)

States	Canals			Tanks (‘000 ha)	Wells (Including Tubewells)	Other Sources	Total
	Govern-ment	Private	Total				
Arunachal Pradesh	-	-	-	-	-	32	32
Assam	71	291	362	-	-	210	572
Himachal Pradesh	-	-	-	-	11	88	99
Jammu and Kashmir	130	159	289	3	3	15	310
Manipur							
Meghalaya	-	-	-	-	-	65	65
Mizoram	-	-	-	-	-	50	50
Nagaland	-	-	-	-	-	8	8
Sikkim	-	-	-	-	-	56	56
	-	-	-	-	-	16	16
Total	201	450	651	3	14	540	1,208

Source: “Dying Wisdom”: A publication by Centre for Science and Environment, 1997

As may be observed from the data in the table, the role of traditional irrigation systems, managed by the farmers themselves is important in the remote and hilly areas. In this lesson, we look more closely to these traditional approaches of irrigation prevalent in different regions of our country and discuss ways by which these may be integrated with modern engineering practices.

Irrigation schemes less than 2000 hectare are designed as minor irrigation schemes. Some examples of such schemes have also been discussed in this lesson.

3.5.1 Traditional systems in different regions of India

In India, one may find regions varying in temperature, elevation and rainfall. As a result, the climate is drastically varying from one place to another for example, when that of the northern Himalayan regions is compared with that of the southern coastal regions or the dry western and central regions are compared with the west eastern and north eastern regions. Hence, the methods of traditional systems of water utilization have evolved very differently over the years in different places. The main sources of water have generally been:

- Utilization of stream and river water
- Direct tapping of rain water (rain water harvesting)
- Ground water

Stream/river/surface water

Wherever these sources exist, especially in the hill and mountain regions of India, people developed techniques to divert its water with the help of simple engineering structures, into artificial channels that would take the water directly to the agricultural fields where streams joined to form rivers, even some larger and complex engineering was used to divert the river water, for example the ***Grand Anicut*** on the river Cauvery.

In the arid and semi arid regions, where water in the streams was more seasonal and not available round the year, the diversion channels were first directed into a storage structure-so that water could be used in the ensuing dry period for human and animal consumption, and for agriculture. However not all storage structures were riverfed or streamfed. Many of them simply collected water running off a catchment area to be stored for later use.

In the flood plains, people developed ingenious techniques to use the menacing flood water, not just to irrigate their fields, but also to fertilize their fields with the silt deposited by flood waters and control diseases like malaria by making use of the fish in the flood waters to eat away the mosquito larvae.

Rain water utilization

There is a great potential of rain water harvesting and this has been used at many places of the country, especially in the drier areas. For example, even in the drier areas of the Thar Desert of Rajasthan, people devised water harvesting structures called the ***kundi***. In certain areas of Rajasthan and Gujarat, people even devised and used techniques to tap the scarce rain water falling on roof tops.

Ground water

Traditionally, dug-wells have been constructed, the water from which was lifted using traditional technology like the Persian wheel or simply pulley arrangement for use in agriculture or drinking water purposes. Wells were, and still are, important source of irrigation in the ground water rich region of the Indo-Gangetic plains. In the dry areas of Rajasthan, people built structures like step wells and wells below tanks and other types of water storage structures. These served both as ground water collection points as well as storage structures for rain water harvesting. In some hilly regions, for instance in the hills and mountains of eastern ghats, people have long since used the Middle Eastern technology of ***qanats*** to build subterranean structures, which are horizontal wells with a mild slope and called ***surangams*** locally, the top the water seeping down the hill sides for use as drinking water.

In summary, the various types of traditional water harvesting system practiced in India may be presented as given in the table below. It must be remembered that all these methods are ingenious forms of water resources engineering and a modern engineer may not only learn from these age old techniques but also suggest new ways to improve them to make these more efficient.

Types of Indian Traditional water harvesting systems

Ecological Regions	***System for Irrigation***	***Systems for Drinking water***
1. Hill and Mountain regions	a) Diversion channels leading directly to agricultural fields (eg. *Guhls* and *kuhls* of western Himalaya) b) Ocassionally, the channels first lead into a storage structure so that water can be used in the subsequent dry period, too (eg. Zings of Ladakh).	a) Natural springs were often harvested. b) Rainwater harvesting from rooftops. c) In the Northeast, spring water is often carried over long distances with the help of bamboo pipes.
2. Arid and Semi-arid regions	a) Rainfed storage structures which provided water for a command area downstream (eg. Tanks)	a) Groundwater harvesting structures like wells and stepwells were built to tap

	b) Stream or riverfed storage structures, sometimes built in a series, with overflow from one becoming runoff for the subsequent one (eg. System tanks of tamilnadu, bandharas of Maharashtra, keres of Karnataka) c) Rainfed storage structures, which allow runoff to stand over and moisten the fertile soil-bed of the storage structure itself, which is later used for growing crops (eg. Khadins of the Jaisalmer district and Johads of the alwar district in Rajasthan).	groundwater aquifers (eg. Bavdis of Rajasthan). b) Groundwater harvesting structures like wells and stepwells were invariably built wherever they were possible, especially below storage structures like tanks to collect clean seepage for use as drinking water (eg. Several such structures can be found in the forts of chittor and Ranthambhore). c) Rainwater harvesting from rooftops (eg. Tankas of Pali). d) Rainwater harvesting using artificially created catchments which drain water into an artificial well – just about any land can be used to create such a water harvesting structure (eg. Kunds of Rajasthan) e) Special rainwater harvesting structures which help to keep sweet rainwater from mixing with saline groundwater and, thus, providing a layer of potable water (eg. *Virdas* of kutch) f) Horizontal wells similar to the middle east to harvest seepage down hill slopes (eg. *Surangams* of Kerala).
3. Plains and Flood plains	a) In the flood plains of major rivers, people built inundation channels which allowed floodwaters to be diverted to agricultural lands (eg. Flood irrigation system of west Bengal) b) In specific types of soil and cropping regions, people also store rainwater in the agricultural fields by bunding them (eg. Haveli system of Madhya Pradesh).	a) Dugwells

4. Coastal Areas	a) Regulatory systems to control ingress of saline riverwaters, especially during coastal tides, and thus maintain crop productivity in the coastal plains (eg. Khazana lands of Goa).	a) Dugwells

The following sections mention briefly the ingenious ways of local people in different parts of India in the field of water tapping, storage and utilization for irrigation as well as drinking purposes.

3.5.2 Western Himalayan region

This is the portion to the north of India spanning from Jammu and Kashmir valleys and going in a south-eastern direction through Himachal Pradesh and ending in Uttaranchal. This region forms the upper catchments of the river Indus and its five tributaries, viz, Ravi, Beas, Sutlej, Jhelum and Chenab in addition to those of the rivers Yamuna and Ganga. Generally, terraced agriculture is commonly practiced on the slopes, and paddy cultivation in the valleys that separate the sub-himalayan ranges from the middle mountains.

Here has been an extensive system of water harvesting in the western Himalayas. Farmers have had a major tradition of building canals aligned roughly with contours to draw water from hill streams or springs. These canals are known as ***kuhls***, which vary from 1 to 15 kms. Generally, a kuhl would have a trapezoidal cross section, 0.1 to 0.2 square meters in area, and usually conveys a discharge of around 15 to 100 litres per second. Many kuhls collect rainwater and snow melt running of the slopes above them, and as a result, ocassionly it is possible to find a kuhl whose discharge increase along its length. The discharge can vary along with the season. A single kuhl can irrigate an area of 80 to 400 ha through distributaries or by flooding. The irrigated land, being situated on hill slopes, is terraced. The system is common at altitudes from 350 to 3000 m in the outer and middle Himalaya where there is a significant drop in elevation in the path of a kuhl, the fall is utilized to drive a simple machinery, like flour mills. Apart from these water channels, another water harvesting structure-the ponds is fairly common in the Jammu region. However, these ponds now suffer from growing encroachment of rising population at water inlets and from siltation.

It is worth noting that utilization of the waters of the mountain streams is need in many villages to drive grinders for grain crushing. In fact, it is the energy of the water due to elevation difference of the water flowing from one point to another. Recently, some villages have taken the initiative to utilize this traditional system of water-energy tapping to drive small turbine connected to generators. The traditional grinders, working with rather very low efficiencies is also being replaced by modern high-efficiency runners which enable higher quantities of grains to the grinded per unit time compared to the traditional areas.

It may be mentioned that though not a traditional system of water utilizing method, the ***hydraulic ram (***or ***hydram*** for short) is needed at many hilly streams to lift water to fields at higher elevations.

In the Nainital and Dehradun districts, channels called *Guhls* are used extensively. Here streams are dammed by temporary diagonal obstructions made of trees and branches, which are used to direct the water into contour channels along the hillside.

3.5.3 Eastern Himalayan Region

The Indian portion of the eastern Himalayan regions consists of the states of Sikkim and Arunachal Pradesh Darjeeling district of West Bengal.

Artificial irrigation is common in the terai region of Darjeeling district. The slope of the land and numerous small streams that exist there, make it easy to utilize the water. The hill springs, called ***jhoras***, are the only dependable source of water in most places.

In Sikkim, local people have evolved efficient water harvesting systems together with land management systems. Irrigation is mostly confined to rice fields and cardamom plantations. In rice fields, irrigation is done in bench terraces. In the case of cardamom, irrigation water is allowed to flow without proper distribution channels. Construction of water channels, regulation of water flow, and drawing of drinking water were traditionally done through community participation. The common sources of drinking water are streams and ***kholas*** (tanks).

There are two important traditional irrigation systems in Arunachal Pradesh-the irrigation of rice terraces with bamboo pipes, and the ***Apatani*** system of wet rice irrigation. In the former method, water is transported through an intricate system of Bamboo pipes to agriculture fields. But this system is now becoming obsolete and is being replaced by iron pipes and channel irrigation.

The apatanis have evolved a very scientific system of field irrigation. The striking features are partially flooded rice fields, and the intricate design of the ‘contour dams dividing the plots. The valley floor has a gentle gradient, and the terraced holdings are laid out along the general slope. The plots are divided by about half a meter high earthen dams supported by bamboo frames. All holdings have an inlet facing the water head and an outlet at the opposite side. The inlets for the low lying plots act as outlets for the higher level plots. A deeper conduit channel connects the inlet point with the outlet point. When a terrace is to be filled with water, the outlet is blocked. By opening and blocking the connecting ducts, any field can be flooded or drained as required.

3.5.4 Northeastern hill ranges

This region stretches over six states Assam, Nagaland, Manipur, Mizoram, Tripura and Meghalaya. The water resources potential of the region is the largest in the entire country. Consequently, there is an abundant groundwater resource. Maximum scope for groundwater exists in Assam, Tripura and Arunachal Pradesh. The available surface water resources have hardly been tapped because of the rugged nature of the terrain. Hence, cultivation in the region is largely rainfed and ***jhum*** cultivation (shifting cultivation) has been widely adopted.

In Nagaland, the cultivations of the southern districts mainly practice net terraced rice cultivation. The proportion of Jhum cultivation in these areas is only 20 percent of the total cultivated area whereas, in Tnensong and Mokokchung districts, it is as much as 70 to 80 percent.

Water harvesting practices in Meghalaya take the form of wet rice cultivation in parts of the Jaintia hills, and of bamboo drip irrigation in other regions. This ingenious system of tapping of stream and spring water by using bamboo pipes to irrigate plantations is widely prevalent and so performed that about 18 to 20 litres of water entering the bamboo pipe system per minute gets transported over several hundred metres and finally gets reduced to 20 to 80 drops per minute at the site of the plant. The bamboo drip irrigation system is normally used to irrigate the betel leaf or black pepper crops planted in arecanut (betelnut) orchards or in mixed orchards. Bamboo pipes are used to divert perennial springs on the hilltops to the lower reaches by gravity.

In Manipur, the jhum method of cultivation is still the prevalent mode of cultivation. However, there has been some tradition of water harvesting amonst the Nagas of Manipur.

In Mizoram, the high rainfall (about 2500mm over about eight months a year) has had ill effects on hillslopes as deforestation has resulted in soil erosion and reduction in water retention capacity of the soil. The traditional sources of water were the numerous springs in the hills, known locally as ***tuikhur***, but many have been drying up recently. For domestic water consumption, therefore the method of rooftop rainwater harvesting has become a widely prevalent practice in Mizoram. A common method of storing rainwater is to place horizontal rain gutters along the sides of the sloping roof, which is normally made of corrugated iron sheets. Rainwater pours into the pipe connected to the tank.

3.5.5 Teesta, Brahmaputra and Barak valleys

These are the plains of the North Bengal and Assam which drain the adjacent hilly and mountainous terrains.
In the North Bengal, there was an old tradition of harvesting streams in the plains abutting the hills. It has been reported that cultivators cut small irrigated channels,

locally called ***jampoi***, from streams which seemed suitable. As such, this method was once the principle source of irrigation in the Jalpaguri district.

In the Brahmaputra and Barak valleys of Assam, too, there had been a long tradition of artificial irrigation, by the means of artificial channels in the sub-mountainous tracts. In certain parts of the state, particularly in Golaghat, Sibsagar, and Jorhat areas, there was also a tradition to dig ponds which were mostly used for drinking water.

3.5.6 Indo-Gangetic plains

This is a zone of high human concentration and is a vast enclosed basin of numerous small and large rivers, separated by alluvial divides. This region comprises of Punjab, Harayana, Chandigarh, Delhi and Northern Uttar Pradesh in the upper Ganga plains. The Middle Ganga plain is the transitional zone between upper and lower ganga plains, located in Eastern Uttar Pradesh. The lower ganga plains extends over Bihar and West Bengal.

Punjab, the land of five rivers, has been rich in cultivation all through its history and traditionally, irrigation was done with the help of canals can be found from the Mughal times when much larger and longer canals started being constructed. For example the Badshahi Nahar was constructed in 1633 A.D by the famous engineer and architect Ali Mardam Khan under the orders of the Mughal emperor Shahjahan. In other places, the Persial wheel was used extensively to lift water from wells. Among other canals, were the Hasli canal originally built by Mughals and later revived by the sikh to feed the sacred tank at Amritsar. In British rule, efforts to renovate the canal were made, but finally it was scrapped and the canal in the upper Bari Doab was constructed. Another medieval canal, the Shah-Nahar, was an inundated cut branching off from the beas. Records show that in the Beas-Sutlej doab (the elevated region between two rivers), irrigation was almost entirely done through wells. A normal well could draw about two thirds of an acre. Irrigation in some areas, like Kapurthala was done by flood waters of rivers. During the British rule, the Sirhind canal was opened which had a large extent of the ***distributary*** channels to bring water within the boundaries of each village.

In Harayana, too there was development of canals during the Mughal period, though prior to that the most prevalent way of irrigation was through wells. Ali Mardam Khan begun the Rohtak canal in 1643 AD by diverting water to Delhi from the old channel, constructed for irrigation. This was gradually extended as far as Gohana rule. The British remodeled these canals in 1870 by putting up a weir on Yamuna at Tajewala, which is the place where the river leaves the hills and joins the plains. The western Yamuna canal still serves to bring water through the plains of Harayana to Delhi, irrigating many hectares of land on the way. In the areas south of Delhi, irrigation was done by the Agra canal which took off from a weir at Okhla on the Yamuna constructed by the British in 1875 this canal still also is under operation.

Since Delhi was not as populated as it is today till some years bak, many parts of the region were used for cultivation. The rulers of Delhi had taken keen interest in providing artificial irrigation water and a nuer of interesting engineering feats of water storage and conveyance were constructed. Though none of these are probably used today for the purpose they were made and even some of these have dried up, it is interesting to note the high level of engineering skill that was existing since about a thousand years in this region.

The Gngetic plains of Uttar Pradesh were always irrigated by wells because the ground water level was very high, and hence boring of wells was easy and cheap in most districts except in the sandy lowlands along the rivers. In these tracts, the wells, unless made of masonry, broke down during the rains. Tanks were also built, which varied in sized, generally covering less than a hectare and not more than 5 to 6 meters deep. Usually, a ***kuchcha*** (unlined) well would be dug at the centre to reach the ground water table. Other sources of irrigated were natural jheels, swamps and small water courses. During British rule, irrigation canals were built in western Utter Pradesh like the Ganga canal designed by P B Cautley in 1846, which is still working today. In Bihar, the large number of Himalayan rivers join the Ganga from the north-Ghagra, Gandak and Kosi, among others. Of the traditional water utilization system, the method of ***ahar-pyne*** was particularly interesting, which probably dated for over two thousand years. The pynes were canals that bring water into ***ahars***, which were tanks. The ahar recharged the ground water and thus the nearby wells generally could draw good amount of water most parts of the year. In recent years, there have been attempts to revive this system, especially where modern irrigation system has not spread. The pyne-ahar system also helped to divert some part of the rain water falling with in the catchment thus reducing flood potential of the larger rivers at lower positions of the terrain.

In Bengal, one of the traditional systems of irrigation had been the inundation canals, which is almost totally abandoned today. In this method, the high flood waters of the rivers were allowed to inundate the adjacent flood plains, thus bringing with it rich silt over which good amount of crops were grown after the passage of flood. However, from the times of the British rule, especially in the early twentieth century and even after independence almost all rivers were embanked, leaving no room such traditional practice of agriculture. As a result, the rivers have to flow through a much constricted cross-sectional area during floods, with a corresponding rise in flood water levels. The falling floods have also deposited the silt on the constricted riverbed, thus raising it dangerously high at many places. At times, the embankments breach during floods, thus inundate the human settlements that have unauthorizedly encroached upon behind these embankments in the flood plains.

3.5.7 The Desert Areas

This is the Thar desert area to the west of Rajasthan and covering parts of Gujarat, Punjab and Harayana. Being a generally water scarce area, these regions developed judicious methods to conserve water. Though many of these systems have been in use for centuries, at many places they are still being used. In the towns, however, new methods of water supply have put some of these in disuse but very recently, with growing awareness about scarcity of water, many such systems are being revived.

The wells have been the most important source of water both for irrigation and drinking purposes in the Jodhpur district. The land irrigated by wells was referred to as ***chahi***. The Luni is an important river in Jodhpur. During rare floods, river water overflowed and crops of wheat and barley were grown on the saturated soil. Tanks have also been used at many places. The land irrigated by canals emanating from large tanks was known as canals ***nahri***. Villagers also collected rainwater in covered pits called ***kund*** or in simple excavations called ***sor***, where the ground was hard. The practice of water conservation was also very strong in this region. In some households in Jaisalmer district, even today people bath on the wooden platform under which a vessel is kept. Water that drains into this vessel is used for animal consumption. Similarly, in parts of Jaisalmer, Pokhran and Phalodi, people bath on a stone block outside the house, from which water drains into an animal watering tank.

The cities in Rajasthan were built around areas that had some sort of water source. All the forts had tanks and wells. Rooftop rainwater harvesting was common across towns and villages. Rainwater that falls on the sloping roofs of houses is taken through a pipe into an underground tank built in the main house or courtyard. The first spell of rain would not be collected as this would clean the roof and the pipes. Subsequent rainwater would be collected in underground tanks, which would as large as a big room. It is reported that though this method is gradually giving way to modern methods liked piped municipal water supply, many houses in Phalodi, still about 2000 still maintain their tanks.

It is, however, unfortunate that many traditional water harvesting structures have been put to disuse, for example, that of Baiji-ka-talab of Jodhpur which was huge reservoir collecting rain water for use in drier months. Many ***bavdis***, or step wells, of these may be revived as the rising population would generate much pressure on the existing water supply system of the town in the near future. Of course, there one other reservoir like the Ranisar Lake situated inside the Mehrangarh fort, which is still use today. The spilled over water of this lake is again collected in another, called the Padansar lake. The water from these lakes not only provides direct supply, but also recharges the ground which keeps the wells and bavdis down the hill full of water throughout the year.

In Gujarat, thar desert extends over a large area. Here too, very similar methods of water harvesting, like tanks wells and stepwells have been in use, perhaps from time as old as the 8^{th} century AD. Interestingly, for the nomadic Madhari tribesmen of Gujarat, ***virdas*** are the principal means of water harvesting. Virdas are shallow wells dug in low depressions and collect enough rain water to ensure the availability of freshwater throughout the year.

3.5.8 Central Highlands

This region comprises of the semi arid uplands of eastern Rajasthan, the Aravali range and the uplands of Banas-chambal basins. In Rajasthan, the Jhansi-Mirzapur uplands of Uttar Pradesh; the eastern hilly regions of Dangs and Panchnabal in Gujarat; northern Madhya Pradesh uplands; the Sagar, Bhopal and the Ratlam plateaus of central Madhya Pradesh; and the Narmada region, include the flanks of the Vindhya and Satpura ranges of southern Madhya Pradesh.

In these areas generally, there is little cultivation on the hillsides. The cultivated area is generally confined to the valleys and the low ground between the hills, where the soil is alluvial. In many of these regions, wells were the chief irrigation source. Tanks were also constructed in and around many forts that are scattered in the region. In some parts of Madhya Pradesh, especially in the upper part of Narmada valley, a unique cultivation system based on water harvesting and runoff farming exists, which is known locally as the ***haveli*** system. Rain water is held in embankment fields, enclosed on four sides until sowing time. Water is let out as soon as the land is dry and the fields are sown. After this, no watering is needed for the crops. The haveli system is location specific, like other ingenious runoff farming practices of the country. The soil, where it is practiced, is heavy black clay, which holds large amount of water. The soil is unfit for kharif crops like paddy or cotton but it is excellent for wheat. An important reason for the development of the haveli system was the need to control weeds.

It may be worth noting that there is a grand example of mughal water engineering works for water supply to the city of Berhampur, by the side of Tapi river. The system, which is still active today, consists of ***bhandaras*** or storage tanks which collect ground water from the underground springs flowing down from the Satpura hills towards the Tapi. The groundwater is intercepted at four places northwest of Berhampur. Water is carried through subterranean conduits with a number of connected wells to a collection chamber, called ***jail karanj**,* and from there to the town.

Though not in function any more, there are three ancient water tanks on the hills of Sanchi, a city renowned for its Buddhist past, which are constructed at about the third century BC.

3.5.9 Eastern highlands

This area extends from Chhatisgarh, Jharkhand, eastern Orissa up to Chhota Nagpur plateau. There is evidence of rich tradition of water harvesting systems of this region. In most of these areas, the undulating terrain has been utilized by constructing small bunds either locally by erstwhile Zamindars, British administrative officials or by the respective state governments, usually under minor irrigation schemes, which is mentioned later in this chapter.

3.5.10 Deccan Plateau

The Deccan plateau consists of the whole of the south Indian table land. The plateau occupies large parts of Maharashtra and Karnataka and the eastern half of Andhra Pradesh.

In Maharashtra, the well irrigation was fairly extensively used and was the principal form of irrigation in the central deccan region. Numerous dams, known locally as ***bandharas***, as a permanent or temporary nature built either of earthern or masonry, were built across rivers and streams in western deccan. These structures would either raise the water level to enable water to form large reservoirs. Where a bandhara was built across a small stream, the water supply would usually last for a few months. When the water level dropped below the level of a channel, a wooden shovel or scoop was used to lift the water. The system of bandhara making is still prevalent in many places and people use modern day ingenious methods like used cement gunny bags filled with soil piled up to form a dam like structure across small rivers. In some places, the system was extended to what is known as ***phad*** system, which is the division of the command area into smaller zones called phads. In fact the bandhara based water diversion through canal is very similar to any modern day irrigation system. The cropping pattern in the phads follows local wisdom. In a year of plentiful water, the community may decide to grow sugarcane in most of the phads, whereas in drier years, sugarcane may be cultivated in few phads, with driver crops being planted on others and even leaving a few phads follow, if needed.

Traditional water harvesting for irrigation in Karnataka used a number of systems: water was supplied directly from river channels; from tanks supplied by river channels; from a series of tanks situated in valleys of rivers and streams, and, by wells and springs locally called ***talpariges***. At the beginning of the twentieth century, channel irrigation was initiated by the British administration but restricted largely to the south of the erstwhile Mysore state. Irrigation was also obtained in the Mysore state from river-fed tanks using the waters of the Cauvery, Hemavati, Yegachi, Lakshman Tirtha, Kabini, Swarnavati, Nagu and Shimsha. In north Karnataka, the Tungabhadra ancients (dykes across rivers, or bunds) and channels partially irrigated the Bellary, Dharwar, and Bijapur districts of the former Bombay presidency. Tank irrigation was also extremely important in Karnataka and it was found predominantly in Bijapur, Shimoga, Kadar, Bellary, Hasson, Tumkur, Kolar and Banglore districts. Even though they were not the primary irrigation source in the mysore state, tanks were common in many regions. Many of them are still active but others have become defunct or the water quality has become degraded, or for the famous tank Hussainsagar in Hyderabad, which was built in the sixteenth century by the Nizam rulers of the princely state. In fact, two other tanks were built later for supplying water to the city of Hyderabad-Himayat Nagar and Osaman Nagar. However, over the recent past these tanks have not filled up during rains probably because of extensive water tapping in the contributing catchment.

The tradition of tanks, some of which have been built as far back as the fourteenth century, remained the largest source of irrigation in Andhra Pradesh until mid-1960s.

The number of tanks in operation over a span of thirty years, as shown in the following table, may give an idea of the relative declination of the dependence on tanks. However, wells for irrigation have increased lately. This might be a reason for lowering of ground water table in many regions, with a resulting cutoff in recharge to tanks from ground water and consequent drying up of some of the surface tanks. There are about 50,000 tanks in Andhra Pradesh.

Decline of tank Irrigation in Andhra Pradesh

Districts	1955-56				1986-89			
	Number of Tanks			Area1 (ha)	Number of Tanks			Area3 (ha)
	Irrigating less than 40 ha	Irrigating more than 40 ha	Total		Irrigating less than 40 ha	Irrigating more than 40 ha	Total	
Drought Prone Districts								
Prakasam	-	-	-	-	584	200	784	38000
Kurnool	426	92	518	24472	301	88	389	15000
Anantapur	1497	740	2237	37254	931	274	1205	19714
Cuddapah	1029	155	1184	29806	1219	163	1382	15333
Chittoor	5290	1336	6626	72024	7152	465	7617	57000
Mahbubnagar	3847	437	4284	68149	5736	499	6235	28667
Nalgonda	3625	329	3954	45078	5061	447	5508	31333
Rangareddy	-	-	-	-	1706	168	1874	8333
Total for drought-prone districts	15714	3089	18803	276783	22690	2304	24994	213380
Non-drought-prone districts	33997	5728	39715	800211	46230	5439	51669	776286
Total Andhra Pradesh	49711	8817	58518	1076994	68920	7743	76663	989666

Source: "Dying Wisdom": A publication by Centre for Science and Environment, 1997

Area under tanks and wells, and rainfall in drought-prone areas of Andhra Pradesh

District	Area under Tanks		Area under wells		Rainfall	
	1959-62 (ha)	1986-89 (ha)	1959-62 (ha)	1986-89 (ha)	1959-62 (mm)	1986-89 (mm)
Prakasam	NA	38000	NA	60000	802	841
Kurnool	28453	15000	7833	35667	587	760
Anantapur	33757	19714	31439	81333	565	633
Cuddapah	27697	15333	36108	80000	435	732
Chittoor	80980	57000	37799	112333	458	896
Rangareddy	NA	8333	NA	46333	713	988
Mahbubnagar	66355	28667	20747	78000	921	753
Nalgonda	57545	31333	27089	81667	859	759
Total	294789	213380	160925	575333	-	-

Source: "Dying Wisdom": A publication by Centre for Science and Environment, 1997

3.5.11 Western Ghats

This constitutes a narrow long range of hills running from north to south along the western coast of India and spreads over parts of the states of Gujarat, Maharashtra, Goa, Karnataka and Kerala. Though there has been very little documented evidence about the traditional water harvesting practices of the people living in these hill ranges, it is gathered that ponds and tanks for water storage and diversion channels from streams has been practiced for many years for paddy cultivation and orchards.

Wells are common in the coastal belt aquifers of Kerala, while medium to large diameter open dug wells and bore wells are common in the midlands. In the highlands, springs, borewells, dugwells, and medium-sized surface tanks are common water harvesting structures. However, the Kasaragad district of northern Malabar has a special harvesting structure called ***surangam***, which is actually adapted from the ***qanats*** that existed in Mesopotania and Babylon at around 700BC. It has been found out by the Centre for Water Resources Development and Management in Kochikode that there has been a rise in the number of surangams in the recent years. A survey has also indicated that about sixty percent of the surangams have perennial water supply, though the summer discharge varied from 2 to 60 litres per minute. Surangams are found to be used for both domestic water supplies as well as for irrigation purposes.

3.5.12 Western coastal plains

This region stretches southwards from the hot and near-arid Kathiawar peninsula of Gujarat to the humid Malabar Coast of Kerala.

The Kathiawar peninsula and the Gujarat plains lie midway between the barren Thar desert and the humid Konkan region. Here, water is found close to the surface and numerous wells, called ***vavs*** or ***bavdis*** are used as sources of water for drinking. In some low-lying districts to the north and northwest of the peninsula, the ground water is saline, and as a result almost every village has a pond or reservoir. In Surat, Bharuch and Ahmedabad districts and the erstwhile Baroda state, wells, lakes and tanks are important sources of water, both for irrigation and domestic use. In Ahmedabad, a number of lakes and reservoirs were built to store water and the Kankariya Lake was built in 1451 AD.

The Konkan region of Maharashtra is a narrow belt of land which lies to the west of Sahyadri range, and is characterized by high humidity and heavy rainfall. Fertile alluvial soils in the lowland regions are used to cultivate rice. These soils are free of salt and the fields are often embanked (***bandhini***) and flooded during the monsoon. Wells, lakes and reservoirs also were important source for irrigation and drinking. The larger rivers are saline ingressed due to their proximity to the sea and the waters are only periodically sweet. Though there are no large canals, often water from small streams are diverted through narrow water courses (***pats***) to irrigate rice fields. Field embankments (***shilotris***) were an interesting feature of tillage in Colaba and Thana districts. These embankments were made to keep out the tidal waters.

The Khazana lands of Goa plays a crucial rola in the states' coastal ecology and cover more than 18,000 ha. The term Khazana is said to be derived from the portugese casino, which means a big rice field, and are the low lying agricultural fields irrigated with water from rivers, which is controlled through sluice gates' two major rivers, the Zuari and the Mandovi, form the food bowl of Goa. Paddy is grown in the Khazana fields during monsoon when rain water reduces the salt in the water to its lowest level, bunds or embankments made of locally available materials like mud, straw, bamboo, twigs and branches from mangrove, laterite stones etc. These bunds protect the fields from being flooded with undesirable brackish water.

In the coastal regions of Karnataka, cultivators have used springs and streams skillfully by diverting numerous channels and constructing temporary dams. In some places, like uttara Kamada, the lowland coastal belt is narrower and the Sahyadris rise to the east. Its lowland and upland valleys have basins crowded with spice and betal garden. The water to these gardens are brought from springs which abound at the head of every valley, and generally collected in small ponds or reservoir and from there channeled to the gardens. Water is also brought by channels from small rivulets which abound the area. In the dry season, dams of earth, stones and tree branches are made across streams and the land in the vicinity watered. The dam is removed at the close of the dry season or left to be swept away later by floods.
Irrigation in Kerala in the past was restricted to the southern regions of the erstwhile Travancore state and to irrigate rice and garden crops. It has been observed that it is an age-old custom in the coastal regions of Kerala to dam small streams and divert water by gravity flow for irrigation of low-lying lands around. The cultivators themselves used

to temporary bunds put up during seasons of cultivation which were washed off in subsequent floods.

3.5.13 Eastern Ghats

This is the rugged, hilly terrain running almost parallel to the eastern coast of India, covering parts of the states of Orissa, Andhra Pradesh, and TamilNadu. In Orissa region of the Eastern Ghats, traditionally, people depended on tanks for irrigation and terracing for cultivation. In Andhra Pradesh region, channel irrigation was traditionally common though tanks were also dug.

The high mountains of the Nilgiri in TamilNadu rise at the southern-most end of the eastern ghats and mark the meeting point of the eastern and western ghats. The continuous slope from the mountains to the sea was made use of by ancient rulers of Madurai to establish a system of irrigation. Whenever the surface dipped a little on either side of the Vaigai River, a strong curved embankment was raised with its concave side facing a river and a channel branching off near the top of the embankment. During floods, water would run through the channel to low lying lands and fill the tanks. Though no artificial irrigation has been practiced in the Nilgiri district, in the Madurai and Coimbatore districts, channel tank, and well-irrigation is widespread.

3.5.14 Eastern Coastal Plains

This is a wide and long stretch of land that lies between the Eastern Ghats and the Bay of Bengal and also covers parts of Orissa, Andhra Pradesh and Tamilnadu.

The coastal plains of Orissa, comprises of three district geographic divisions-the salt tract; the arable tract, which is the main rice producing zone of the state, and the submountainous tract, where the delta meets the mountainous tract. Generally, the mainstay of irrigation sources in the hilly areas had been the tanks. Some tanks were excavated and fed by natural streams, others were made by constructing embankments across drains. In the plains, the chief source of irrigation was flood water which was let out into cultivated areas through embankment sluices. Earthen dams were also constructed across minor drainage channels to raise the level of water to irrigate adjacent fields.

The coastal plains of Andhra Pradesh comprise a belt of varying width which extends from Visakhapatnam in the north to Nellore in the south. The two main river systems of this region are the Krishna and Godavari. Numerous small streams and rivers flow down the hills which have traditionally been used to irrigate crops either through cuts in embankments or by tanks fed by these streams. In the Visakhapatnam district, the Eastern Ghats give rise to numerous streams that flow directly into the sea or to the Godavari. It has been recorded that traditionally weed and brushwood dams were built across the streams and flood waters were diverted directly into fields or indirectly through storage tanks. The main rivers that were used for irrigation in these plains were the Varaha, Sarada, Nagavali and the Suvarnamukhi. There were a number of ancient dams, called the ***anicuts***, on these rivers. In the Krishna and Godavari deltas, irrigation largely depends on channels drawn from the rivers. Hence, on the whole, irrigation in the coastal plains of Andhra Pradesh was dependent on tanks and anicuts, many of which are still in use today.

The region between the Nilgiri hills and the coast of Tamilnadu covers a large tract of land that suffers from rather scanty rainfall. Hence, artificial irrigation has been practiced for centuries. The various sources of irrigation in the erstwhile Madras presidency were rivers, reservoirs, tanks and wells. Anicuts (diversion weirs) on rivers like Tambrapami, Noel, Tungabhadra, Amaravati, Vaigai and Pennar were constructed by local rulers, much before the British rule, to irrigate the adjacent areas through systems of irrigation channels. It is indeed marvelous that many of these systems are still in use today. For example, the grand anicut on the river cauvery was build by the Chola king, Kari, in the second century AD, and is still in use in slightly remodeled form. Apart from anicuts, there has also been a strong tradition of irrigation through tanks, called eris. The eris have played several important roles in maintaining ecological harmony as flood coastal systems, preventing soil erosion and wastage of runoff during periods of heavy rainfall, and recharging the ground water in the surrounding areas. It may be noted that the tanks of south India are mostly found in the red soil regions. Though they supply many villages with drinking water, their primary purpose is irrigation, especially of paddy fields. It has been estimated that in Tamilnadu, there are about 30,000 tanks or ***eris*** which account for an estimated 50,000 Km long embankments and about 300,000 masonry structures. The eris usually have embankments on three sides and one side is left open for water to flow in. The water stored in the eris is released through sluices. Eris have two sluices-the ***mettu madagu***, which is on a higher elevation, is usually opened to release water, and the ***palla madagu***, which is at a lower level, is opened once the water stops flowing out of the metta madugu. Tanks also have an outlet (***kalanga***) at the full tank level.

3.5.15 The Islands

There are two sets of islands in India the Andaman and Nicobar Islands in the Bay of Bengal, and the Lakshadweep islands in the Arabian Sea.

The Shompen and Jarawa tribals of Andaman make extensive use of split bamboos in their water harvesting systems. A full length bamboo is cut longitudinally, and placed along a gentle slope with the lower end leading into a shallow pit. These serve as conduits for rainwater which is painstakingly collected, drop by drop, in pits called ***jackwells***. Among the ones, buckets made of logs and sometimes of giant bamboo, are often found suspended from the roofs of the huts along with nets and baskets to trap rain water.

In Car Nicobar the tribals, mainly Nicobarese and Jarawas, use circular dugwells ranging from 2 to 20 metre in diameter, since the water table is just about 2 to 3 meters from ground.

In the Lakshdweep islands, the residents usually have wells for meeting their drinking water requirements. Many of these wells are stepwells and almost every house hold has a dugwell for domestic purposes. In Kavaratti, there are about 800 dugwells while in Amini there are more than 650. Presently, however, rainwater harvesting (from roofs) structures are being constructed all over the islands.

3.5.16 Minor Irrigation

Minor irrigation schemes have probably evolved as a systematic development only after the country's independence and with the initiation of the Five-Year Plan. These include all ground water and surface water irrigation projects having culturable command area of up to 2000 Ha. Naturally, the groundwater schemes require pumping up of water from tube wells, whereas surface water schemes may include those which involve flow due to gravity as well as lift by pumping from surface water flow irrigation projects include storage and diversion works and are the only means of irrigation in several areas, like the undulating terrains of Central India and the ghats or the hilly tracts of north and north east India. Such projects offer considerable opportunity for rural employment and also help in hard rock areas. When available surface water cannot be used for irrigation through construction of flow irrigation schemes due to topographical limitations, surface water lift irrigation schemes provide the solution.

As such, ground water development is a major activity of the minor irrigation programme. It is mainly a cultivators' own programme implemented primarily through institutional sources. Adequate energy for pumping ground water is essential for near normal production of crops when there is severe drought. Hence, energy management is also essential.

Under surface flow minor irrigation schemes, water tanks are created by constructing bunds across depressions in undulating terrains. There should be a properly designed spillway for overflowing excess rain water during monsoons and sluice gates for releasing controlled quantities of water to canals. It is obvious that the amount of water expected in the reservoir would be proportional to the catchment area draining into the reservoir and the rainfall in the catchment.

For lift irrigation schemes, pumps have to be installed on the river or canal banks with the suction pipe long enough to be immersed into the flowing water body and should also ensure sufficient draft. At times, floating barges moored to the banks, may have to be installed to accommodate the pumps. Here, the suction pipe would be less but the lift pipe would be longer.

Module 3

Irrigation Engineering Principles

Lesson 6

Canal Systems for Major and Medium Irrigation Schemes

Instructional objectives

On completion of this lesson, the student shall learn about the following:

1. The canal system of irrigation under major and minor irrigation schemes
2. Components of a canal irrigation system
3. Layout of a canal irrigation system
4. Lining methods in canal irrigation
5. Negative impact of canal irrigation

3.6.0 Introduction

As noted in earlier chapters, major irrigation schemes are those which have culturable command area (C.C.A) more than 10,000 ha. Irrigation schemes having C.C.A between 2000 and 10000 ha are called medium irrigation schemes. All these schemes are flow types of irrigation systems and water from a river is diverted to flow through a canal by constructing diversion structure across the river. The main canal further divides into branches and distributaries water from the distributaries is let off through gated outlets into the fields with the help of water courses.

It is important to note that the canal method of water conveyance and distribution is a dynamic system with variation in demand occurring according to the crops planted in the command area. Also, the source of water, usually a river, may not be able to supply sufficient amount of water all times. Nevertheless, the canal system has to be planned and designed for the maximum expected demand. The layout of the canal is also important as it should ensure smooth flow by gravity in each channel. Wrong alignment may lead to possible stagnation of water at some places or too fast moving that may damage the canal itself. This lesson discusses about various aspects of irrigation canal system layout and design. Other canals like those for navigation and power generation are discussed separately.

3.6.1 Canal Components

At the diversion structure, a headwork regulates the flow into a canal. This canal, which takes its supplies directly from the river, is called the main canal and usually direct irrigation from the waters of this canal is not carried out. This acts as a feeder channel to the branch canals, or branches. Branch canals generally carry a discharge higher than 5 m^3/s and acts as feeder channel for major distributaries which, in turn carry 0.25 to 5 m^3/s of discharge. The major distributaries either feed the water courses or the minor distributaries, which generally carry discharge less than 0.25 m^3/s.

A typical layout of an irrigation canal system may be seen in Figure 1.

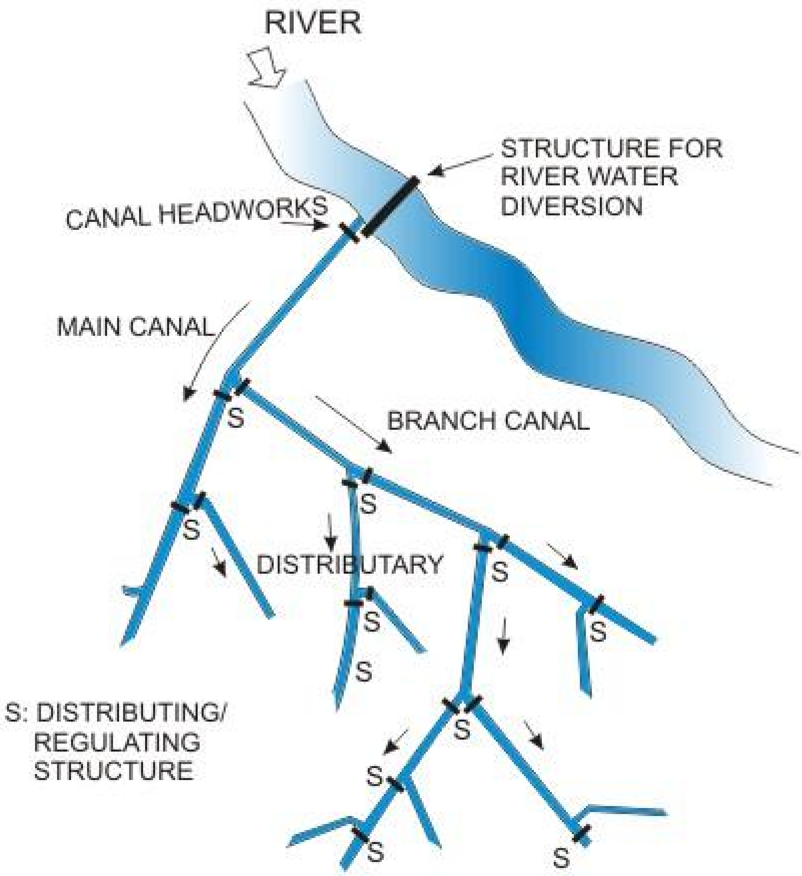


FIGURE 1 : Typical layout of an irrigation canal system

It may be observed that whenever a channel off-takes from the corresponding main branch there has to be gated structures just downstream of the bifurcation, for controlling the water level upstream of the point as well as to control the amount of water going into the off take and the main branches.

3.6.2 Irrigation canal layout

Usually it is desirable that a canal off taking from a river should be able to irrigate as much an area as possible. The general layout concept can be explained when studied in respect of the off take point of the canal and the surrounding contours, as shown in Figure 2.

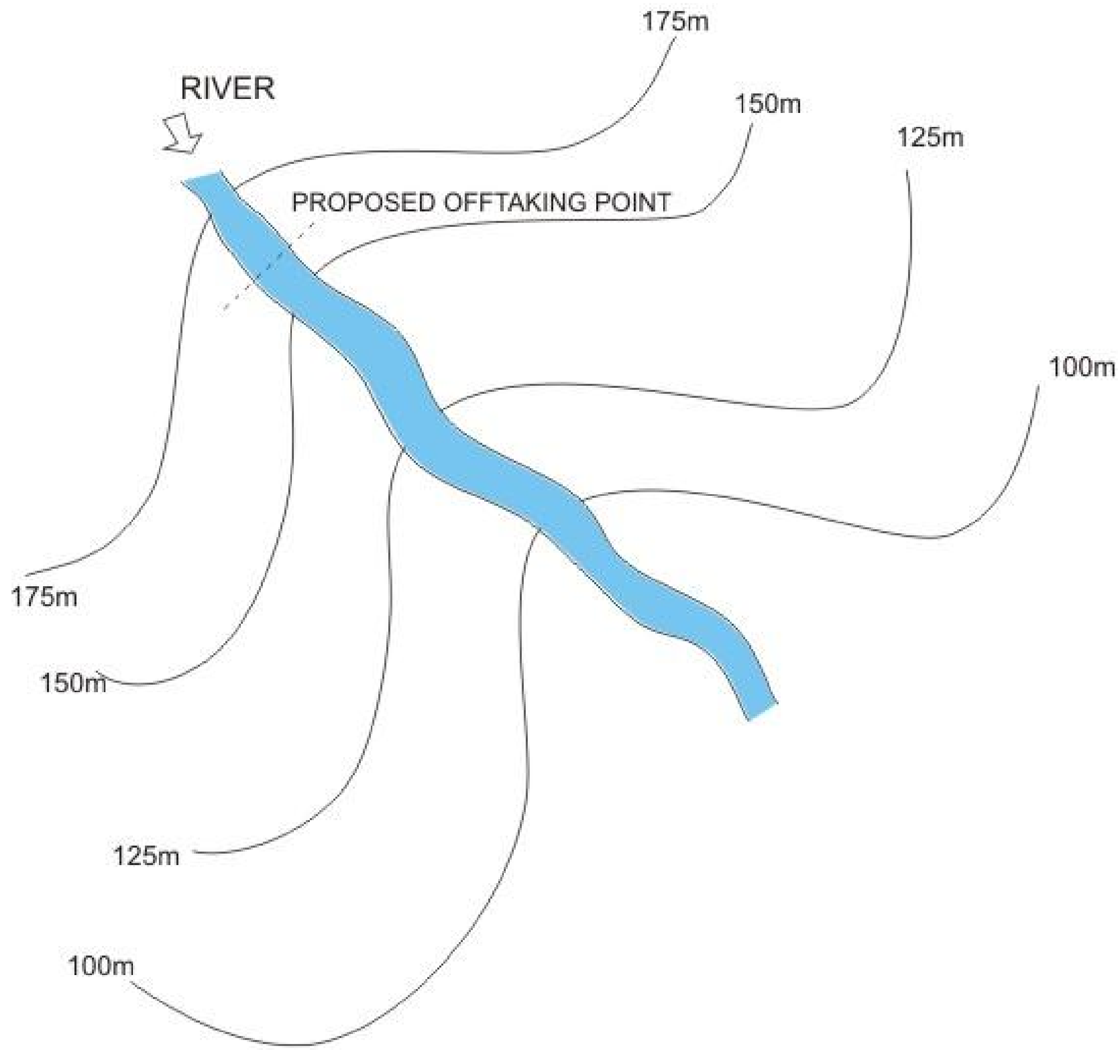


FIGURE 2 : Location of an off taking point for an irrigation system showing a river and hypothetical elevation contours

Now, if a canal were to be laid from the off taking point, then it should lie between the two extreme lines, as explained below.

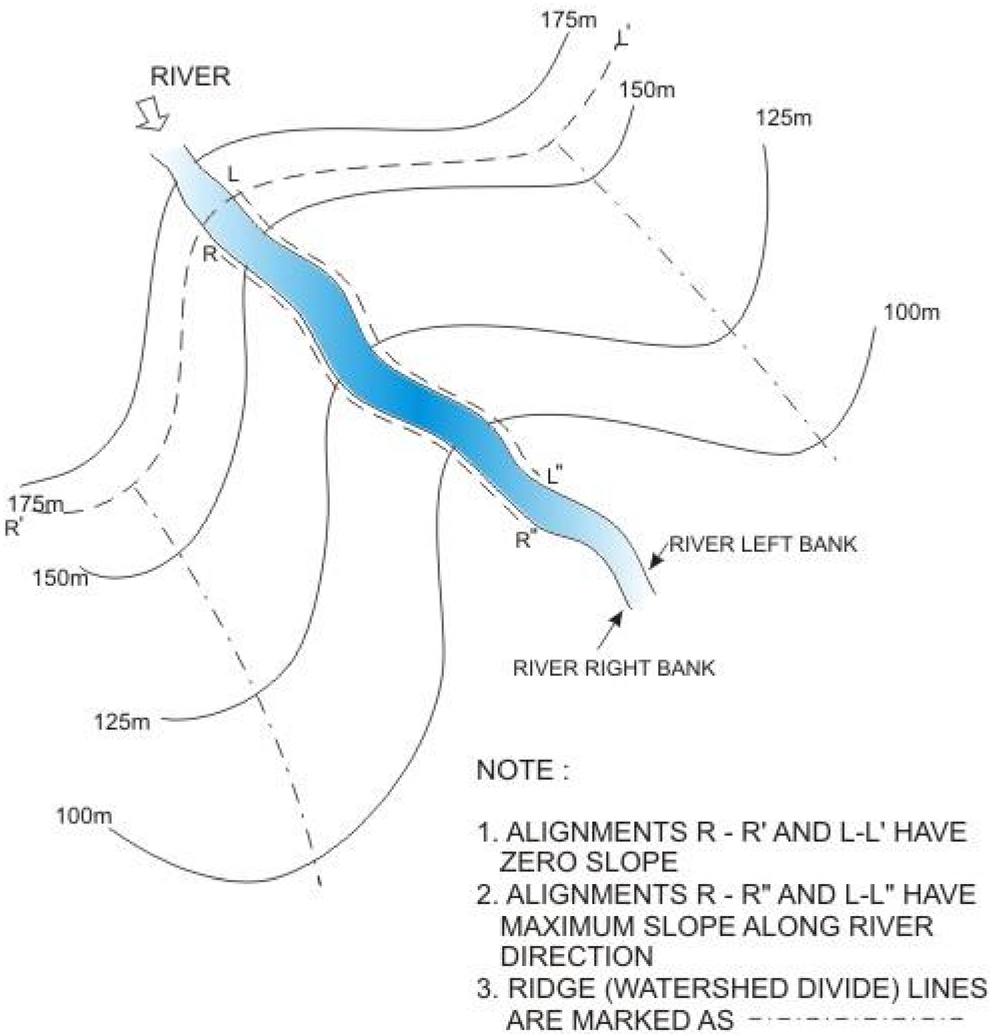


FIGURE 3 : Possible alignments based on which a practical alignment of canal has to be chosen

Consider the possible alignments of canals as shown in Figure 3. A shown in the figure, the ***right bank canal*** (generally termed RBC) and off taking from a point R on the river bank may be aligned somewhere in the region bounded by R-R', the contour line at the elevation of R or the right riverbank R-R''. It is not possible to align the canal along R-R', as there would not be any slope, whereas an alignment along R-R'' would mean zero command area for the canal. Hence, a suitable slope of the canal that is neither too flat nor too steep (as discussed later in this lesson) would be the most appropriate.

Based on the above logic, possible alignments of the right and left bank canals (RBC and LBC) have been shown in Figure 4.

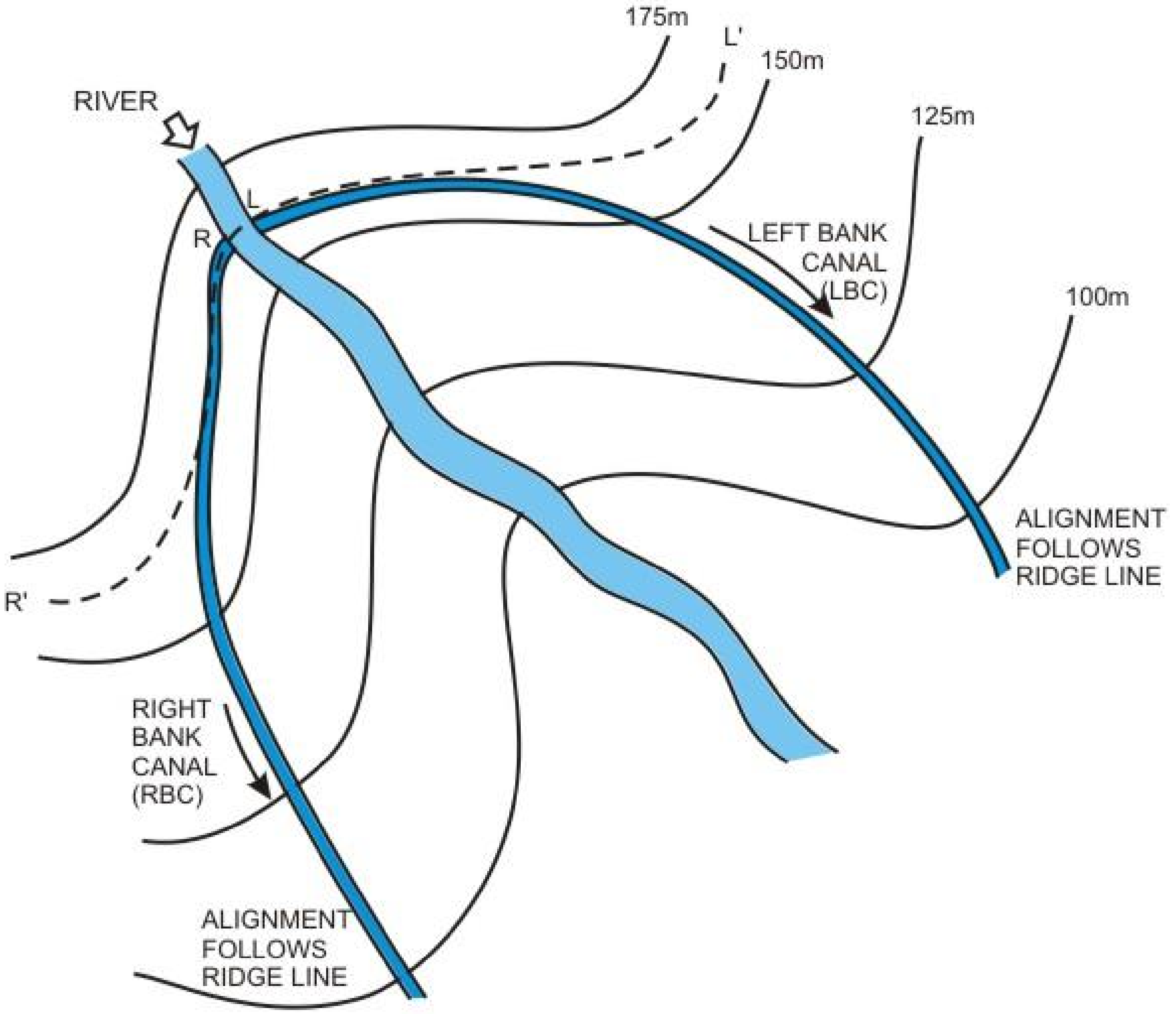


FIGURE 4 : Practical alignments of canals taking off from right and left banks of the river at location L-R

In order to demonstrate the effect of the adjacent valley slopes on the canal layout, the right and left bank contours have been chosen in such a way that the slope of the right bank valley is flatter than that of the left bank. Hence, as may be observed from Figure 5, for the same canal slope on both the banks, the right bank canal covers a larger command area (the area between the canal and the river).

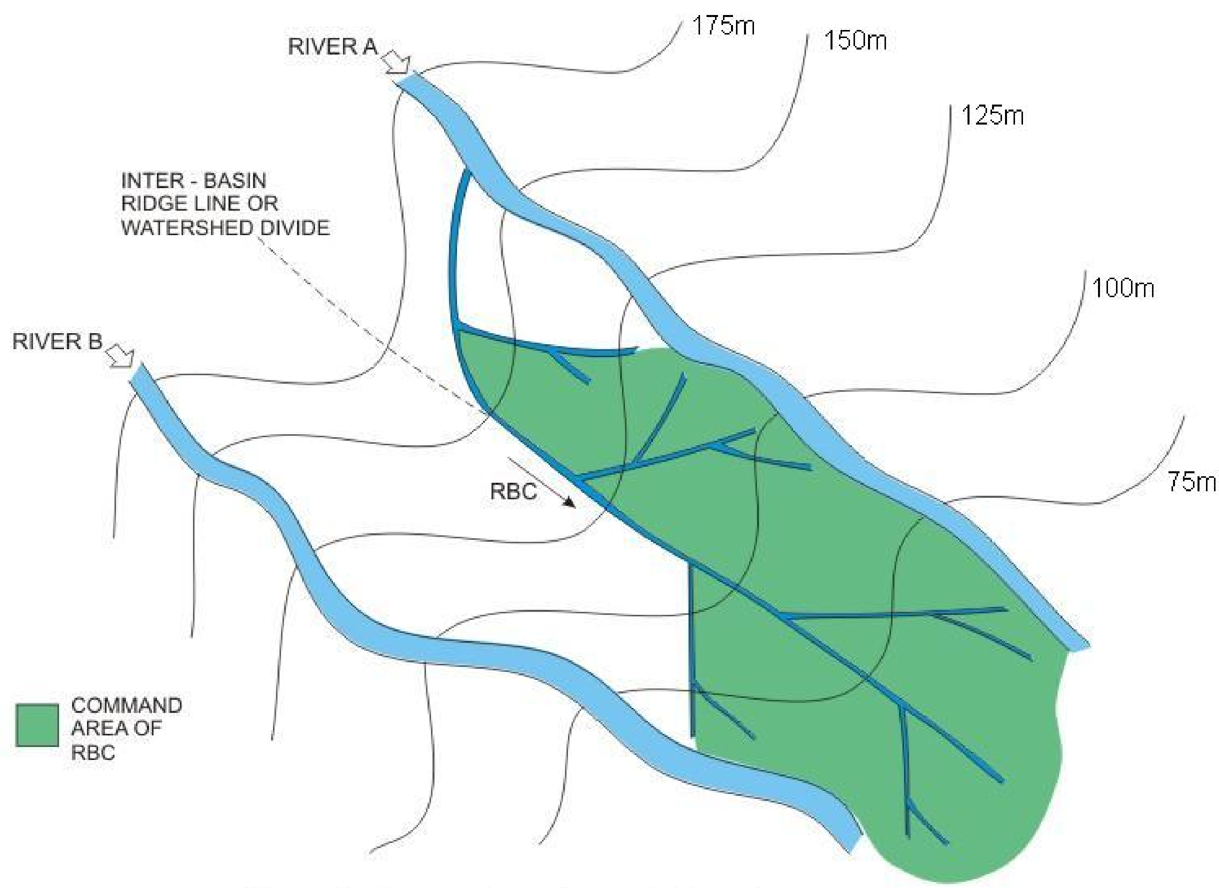


FIGURE 5 : Command area for a typical canal system

The alignment of a canal can be done in such a way that it is laid up to a ridge line between two valleys, which would allow a larger command area for the same canal, as shown in the figure, which shows possible contour lines between two rivers that the canal off taking from one river is able to irrigate areas between the river of the adjacent valley, too.

There is one more advantage of leading a canal up to the watershed divide line: the number of small streams to be crossed by the canal would be a minimum, though once a canal may be aligned along the watershed divide line, generally, it may be necessary to provide a shorter path if the divide line is tortuous.

A few more points may be noted on the layout of a canal may be noted, as mentioned below.

- As far as possible, curves should be avoided in the alignment of canals because the curves lead to disturbance of flow and a tendency to silt on the inner bend and scour the toe of the outer (concave) bend. However, if curves have to be provided; they should be as gentle as possible. Further, the permissible minimum radius of curvature for a channel curve is shorter for lined canals than unlined ones and is shorter for small cross sections than for large cross sections of canals. According to the Bureau of Indian Standard code IS: 5968-1970 “Guide for planning and layout of canal system for irrigation”, the radii of curves should usually be 3 to 7 times the water surface width subject to the minimum values as given in the following table.

Type of canal	Capacity of canal (m^3/s)	Minimum radius (m)
Unlined canals	80 and above	1500
	30 to 80	1000
	15 to 30	600
	3 to 15	300
	0.3 to 3	150
	Less than 3	90
Lined canals	280 and above	900
	200 to 280	750
	140 to 200	600
	70 to 140	450
	40 to 70	300
	10 to 40	200
	3 to 10	150
	0.3 to 3	100
	Less than 0.3	50

- The alignment should be such that the cutting and filling of earth or rock should be balanced, as far as possible.

- The alignment should be such that the canal crosses the natural stream at its narrowest point in the vicinity.

In order to finalize the layout of canal network for an irrigation project, the alignment of channels should be marked on topo-sheets, until an optimum is reached. This alignment is then transferred to the field by fixing marking posts along the centerline of the canal.

Formal guidelines for canal layout may be had from Bureau of Indian Standard IS: 5968-1987 “Guide for planning and layout of canal system for irrigation”.

3.6.3 Lining of Irrigation canals

Though irrigation canals may be constructed in natural or compacted earth, these suffer from certain disadvantages, like the following

- Maximum velocity limited to prevent erosion
- Seepage of water into the ground
- Possibility of vegetation growth in banks, leading to increased friction
- Possibility of bank failure, either due to erosion or activities of burrowing animals

All these reasons lead to adoption of lining of canals, though the cost may be prohibitive. Hence, before suggesting a possible lining for a canal, it is necessary to evaluate the cost vis-à-vis the savings due to reduction in water loss through seepage.

Apart from avoiding all the disadvantages of an unlined canal, a lined canal also has the advantage of giving low resistance and thus reducing the frictional loss and maintaining the energy and water surface slopes for the canal as less as possible. This is advantageous as it means that the canal slope may also be smaller, to maintain the same discharge than for a canal with higher friction loss. A smaller canal slope means a larger command area.

3.6.4 Types of canal lining

Different types of canal linings are possible, and the bureau of Indian standards code IS: 5331-1969 "Guide for selection of type of linings for canals" may be consulted for details. In general, the following types of linings are generally used.

3.6.4.1 Concrete lining

Cement concrete lining made from selected aggregate gives very satisfactory service. Despite the fact that they are frequently high in their initial cost, their long life and minimum maintenance make them economical. Cement concrete lining are best suited for main canals which carry large quantities of water at high velocities. However, a firm foundation is essential for avoiding any possibility of cracking due to foundation settlement. Expansive clay soils should be avoided and proper moisture and density control of the sub grade soil should be maintained while lining. In areas where the ground water table is likely to rise above the invert level of the lining and cause undue uplift pressure, drains are laid below the lining to release the water and relieve the pressure, generally, a thickness of about 5 to 12 cm is generally adopted for larger canals and stable side slopes are considered to be between 1.5H: 1V to 1.25H: 1V. Reinforcement to the extent of 0.1 to 0.4 percent of the area in longitudinal direction and 0.1 to 0.2 percent of the area in the transverse direction reduces width of the shrinkage cracks, thereby reducing seepage. Further details regarding cement concrete linings may be had from Bureau of Indian Standards code IS: 3873-1987 "Code of practice for laying in-situ cement concrete lining on canals", since there would be construction/contraction joints in the lining, it is essential to plug the joints, for which the following code may be referred IS: 13143-1991 “Joints in concrete lining of canals-sealing compound”.

3.6.4.2 Shotcrete lining

Shotcrete, that is, cement mortar in the ratio of 1 cement to 4 sand proportions is through a pump-pipe-nozzle system on the surface of the channel. Wire mesh reinforcement is generally, though not necessarily, is clamped to the channel surface (as for a rocky excavation) before applying shot Crete. Equipment units used for shot Crete construction are relatively small and easily moved. They are convenient for lining small sections, for repair of old linings, and for placing linings around curves or structures. Shot Crete linings are generally laid in a thickness of about 3.5cm, but many standard code IS: 9012-1978 “recommended practice for shotcreting” (Reaffirmed in 1992) may be consulted for details.

3.6.4.3 Brick or burnt clay tile lining precast concrete tile lining

This type of lining is popular because of certain advantages like non-requirement of skilled mason or rigid quality control. Further, since it is more labour intensive, it generates employment potential. Brick tiles can be plastered to increase the carrying capacity of canal with same section and help in increasing the life span of the lining. Sometimes a layer of tiles is laid over a layer of brick masonry. The top layer is generally laid in 1:3 cement mortal over 15mm thick layer of plaster in 1:3 cement plaster. The size of tiles is generally restricted to 30mm x 150mm x 53m. This type of lining is stable even if there is settlement of foundation, since the mortar joint between bricks or tiles provides for numerous cracks so fine that seepage is insignificant. Further details may be had from the following Bureau of Indian Standard codes:

- IS: 3860-1966 “Specifications for precast cement concrete canal linings”.
- IS: 3872-1966 “Code of practice for lining of canals with burnt clay tiles”.
- IS: 10646-1991 “Canal linings-cement concrete tiles”

3.6.4.4 Boulder Lining

Also called dry stone lining or stone pitching, is used for lining the earthen canal cross section, by proper placement and packing of stones, either after laying a filter layer over the soil surface or without any such filter, depending upon the site requirement. To reduce the resistance to flow, a 20 to 25mm thick cement plaster is provided as a finishing surface. Stones are generally placed on leveled sub-grade, and hand packed. This type of lining is of course suitable where stones of required specification are available in abundance locally. For details of this type of lining, one may refer to the Bureau of Indian standard code IS: 4515-1976 “Code of practice for boulder lining of canals”. One advantage of this type of lining is allowing free flow of water from the submerged or saturated sub-grade into the canal. Hence, this type of lining does not need any drainage arrangement or pressure relief values, etc. which may be required for concrete or brick lining.

3.6.4.5 Low density polyethylene lining (from *IS: 9698-1980/1991*)

3.6.4.6 Hot bitumen/Bituminous felt lining (from *IS:9097-1979*) (Reaffirmed 1990)

3.6.4.7 Earth linings

The different types of earth linings that are used in canals include the following:

i. Stabilized earth linings: Here, the sub-grade is stabilized using either clay for granular sub-grade or by adding chemicals that compact the soil.

ii. Loose earth blankets: Fine grained soil is laid on the sub-grade and evenly spread. However, this type of lining is prone to erosion, and requires a flatter side slopes of canal.

iii. Compacted earth linings: Here the graded soil containing about 15 percent clay is spread over the sub-grade and compacted.

iv. Buried ***bentonite*** membrames: Bentonite is a special type of clay soil, found naturally, which swell considerably when wetted. Buried bentonite linings for canals are constructed by spreading soil-bentonite mixtures over the sub-grade and covering it with gravel or compacted earth.

v. Soil-cement lining: Here, cement and sandy soil are mixed and then compacted at optimum moisture content or cement and soil is machine mixed with water and then laid. The Bureau of Indian Standards code IS: 7113-1973 “Code of Practice for soil cement lining for canals” (Reaffirmed in 1990) may be controlled for details regarding this type of lining.

3.6.5 Ill-effects of canal irrigation: Water logging

Canal is an artificial channel for conveying water through lands that was perhaps naturally devoid of sustained water flow. Hence, water seeping from canals down to the soil below may, at times, raise the ground water very close to the ground level. This may result in blocking all the voids in the soil and obstructing the plant roots to breathe. It has been observed that water logging conditions adversely affects crop production as it is reduced drastically. Apart from seepage water of canals, excessive and unplanned irrigation also caused water logging conditions. This happens because the farmers at the head reaches of canals draw undue share of canal water in the false hope of producing larger agricultural outputs.

Apart from ill aeration of plants, other problems created by water logging are as follows:

- Normal cultivation operations, such as tilling, ploughing, etc. cannot be easily carried out in wet soils. In extreme cases, the free water may rise above the ground level making agricultural operations impossible.

- Certain water loving plants like grasses, weeds, etc. grow profusely and luxuriantly in water-logged lands, thus affecting and interfering with the growth of the crops.
- Water logging also leads to a condition called salinity, which is caused when the capillary fringe of the elevated water table rises within the root zone of plants. Since the roots of the plants continuously draw water from this zone, there is a steady upward movement of water which causes rise of salts, especially alkali salts, to come up to the ground surface. This situation is termed as salinity.

In order to avoid water-logging condition to occur for canal irrigation system, certain steps may be taken as follows:

- Canals and water courses may be lined. Also if possible, the full supply level of canal may be reduced.
- Intensity of irrigation may be reduced and farmers advised to apply water judiciously to their fields and not over-irrigate.
- Provide an efficient drainage system to drain away excess irrigation water.
- Introduce more tubewells for irrigation which shall lower the water table
- Cropping pattern may be suitably modified such that only low water requiring crops are planted instead of those requiring heavy irrigation
- Natural drainage of the soil may be improved such that less of excess surface water percolates and mostly drains off through natural drains.

Since water loss by seepage from canals is an important factor through which they pass, it is essential to scientifically monitor when they pass and devise a suitable seepage prevention strategy. This may be done by certain methods, which have been elaborated in the Bureau of Indian Standards code IS: 9452 “Code of practice for measurement of seepage losses from canals”, with the following parts:

- IS: 9452 Part 1-1980 “Ponding method” (Reaffirmed 1991)
- IS: 9452 Part 2-1980 “Inflow-outflow method” (Reaffirmed 1990)
- IS: 9452 Part 3-1988 “Seepage meter method” (Reaffirmed 1990)

Another method, the analytical method, detailed in IS: 9447-1980 “Guidelines for assessment of seepage losses from canals by analytical method” to adopt.

3.6.6 Maintenance of Canals

It is often seen that the conditions under which a canal system is designed is not maintained during the years of its system is operation, Physical damage resulting from erosion/deposition of sediments, ground water, soil subsidence, human activity, weed growth, etc. cause much changes from the ideal conditions. Hence, it is not only essential to design a canal system well; it should also be maintained well. The Bureau of Indian standards has published the code IS: 4839 “Code of practice for maintenance of canals” under the following volumes for the purpose.

- IS: 4839 Part 1-1992 “Unlined canals” (Second revision)
- IS: 4839 Part 2-1992 “Lined canals” (Second revision)

3.6.7 Drainage of land for canal Irrigation Commands

A proper design of canal irrigation system also consist provision of a suitable drainage system for removal of excess water. Of course, this may not be required all over the command area of the canal, but may be necessary in areas of high water table and in river deltas. The drainage system may also help to drain out storm water as well, and thus to prevent its percolation and to ensure easy disposal. There are two types of drainages that may be provided, which are either of surface-type or of sub-surface types. These are briefly described in the following paragraphs.

3.6.7.1 Surface Drainage

These constitute open ditches, field drains, proper land grading and related structures. The open drains which are broad and shallow are called shallow surface drains and carry the runoff to the outlet drains. The outlet drains are termed as the deep surface drains. Land grading, or properly sloping the land towards the field drains, is an important method for effecting surface drainage. The Bureau of Indian Standards code IS: 8835-1975 “Guidelines for planning and design of surface drains” (Reaffirmed in 1990) may be referred to for further details.

3.6.7.2 Sub-Surface Drainage

These are installed to lower the water table and consists of underground pipes which collect water and remove it through a network of such pipes. These pipes are usually made of porous earthenware and circular in section and the diameter varies from 10 to 30 cm. for installation of these drains, trenches are dug in the ground and these pipe sections are butted against each other with open joints which help in allowing ground water to enter into the pipes. The trenches are then backfilled with sand and excavated material. The water drained by the tile drain is discharged into a bigger drain or into a deep surface drain. If the tile drain network is buried quite deep into the ground, it may be necessary to discharge the water of the drains into an underground sump and remove the water from the sump by pumping. Generally an area is under laid with a network of tile drains, it is essential to calculate the spacing of these drains based on the depth through which the water table in the region is to be lowered.

Module 3

Irrigation Engineering Principles

Lesson 7

Design of Irrigation Canals

Instructional Objectives

On completion of this lesson, the student shall learn about:

1. The basics of irrigation canals design
2. The procedures followed to design unlined and lined canals
3. Methods for subsurface drainage of lined canals

3.7.0 Introduction

The entire water conveyance system for irrigation, comprising of the main canal, branch canals, major and minor distributaries, field channels and water courses have to be properly designed. The design process comprises of finding out the longitudinal slope of the channels and fixing the cross sections. The channels themselves may be made up of different construction materials. For example, the main and branch canals may be lined and the smaller ones unlined. Even for the unlined canals, there could be some passing through soils which are erodible due to high water velocity, while some others may pass through stiff soils or rock, which may be relatively less prone to erosion. Further, the bank slopes of canals would be different for canals passing through loose or stiff soils or rock. In this lesson, we discuss the general procedures for designing canal sections, based on different practical considerations.

3.7.1 Design of lined channels

The Bureau of Indian Standards code IS: 10430 -1982 “Criteria for design of lined canals and guidelines for selection of type of lining” (Reaffirmed in 1991) recommend trapezoidal sections with rounded corners for all channels-small or large. However, in India, the earlier practice had been to provide triangular channel sections with rounded bottom for smaller discharges. The geometric elements of these two types of channels are given below:

Triangular section

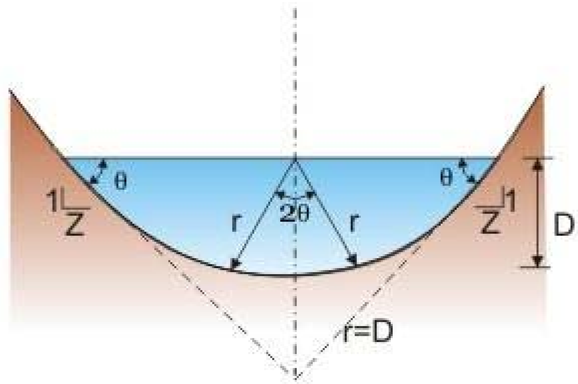


FIGURE 1 . Triangular channel section

For triangular section, the following expressions may be derived

$$A = D^2 (\square \cot \square) \quad (1)$$

$$P = 2 D (\square \cot \square) \quad (2)$$

$$R = D / 2 \quad (3)$$

The above expressions for cross sectional area (A), wetted perimeter (P) and hydraulic radius (R) for a triangular section may be verified by the reader.

Trapezoidal section

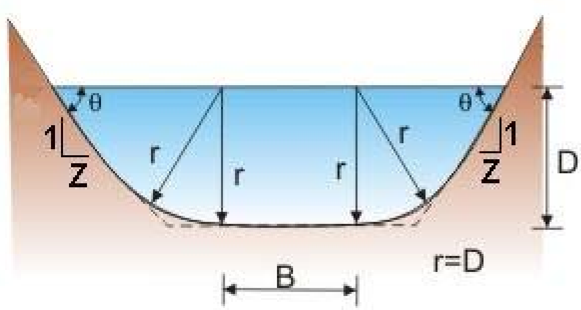


FIGURE 2. **Trapezoidal** channel section

For the Trapezoidal channel section, the corresponding expressions are:

$$A = B D + D^2 (\square \cot \square) \quad (4)$$

$$P = B + 2D(\theta + \cot\theta) \qquad (5)$$

$$R = A / P$$

The expressions for A and P may, again, be verified by the reader. In all the above expressions, the value of θ is in radians.

The steps to be followed for selecting appropriate design parameters of a lined irrigation channel, according to IS: 10430 may be summarized as follows:

1. Select a suitable slope for the channel banks. These should be nearly equal to the angle of repose of the natural soil in the subgrade so that no earth pressure is exerted from behind on the lining. For example, for canals passing through sandy soil, the slope may be kept as 2H: 1V whereas canals in firm clay may have bank slopes as 1.5H: 1V canals cut in rock may have almost vertical slopes, but slopes like 0.25 to 0.75H: 1V is preferred from practical considerations.

2. Decide on the freeboard, which is the depth allowance by which the banks are raised above the full supply level (FSL) of a canal. For channels of different discharge carrying capacities, the values recommended for freeboard are given in the following table:

Type of Channel	Discharge (m^3/s)	Freeboard (m)
Main and branch canals	> 10	0.75
Branch canals and major distributaries	5 – 10	0.6
Major distributaries	1 – 5	0.50
Minor distributaries	< 1	0.30
Water courses	< 0.06	0.1 – 0.15

3. Berms or horizontal strips of land provided at canal banks in deep cutting, have to be incorporated in the section, as shown in Figure 3.

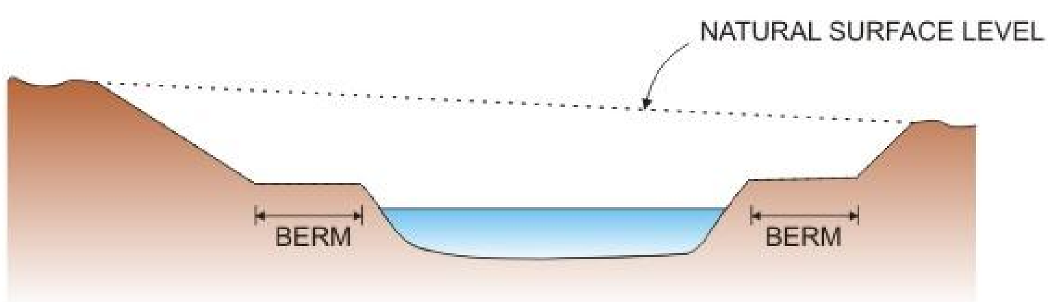


FIGURE 3. Berms for canal banks at deep cutting

The berms serve as a road for inspection vehicles and also help to absorb any soil or rock that may drop from the cut-face of soil or rock of the excavations. Berm width may be kept at least 2m. If vehicles are required to move, then a width of at least 5m may be provided.

4. For canal sections in filling, banks on either side have to be provided with sufficient top width for movement of men or vehicles, as shown in Figure 4.

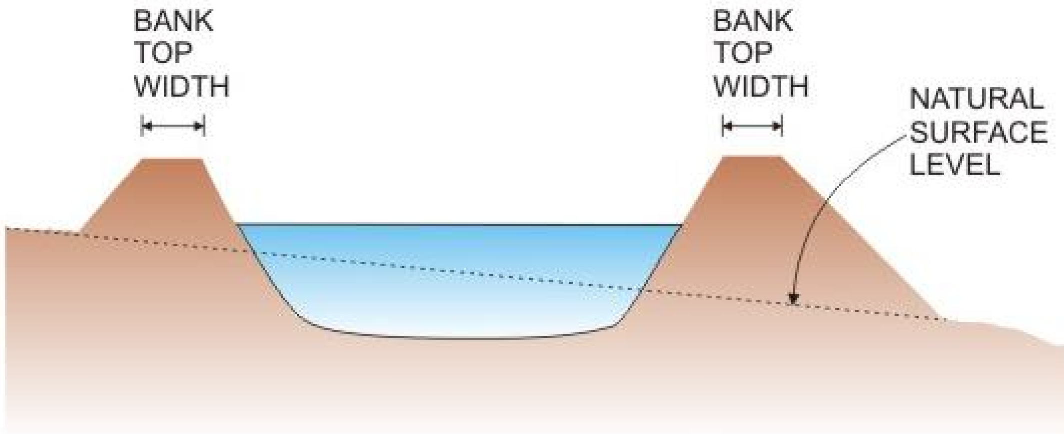


FIGURE 4. Canal section in filling

The general recommendations for bank top width are as follows:

Discharge (m^3/s)	Maximum bank top width (m)	
	For inspection road	For non-inspection banks
0.15 to 7.5	5.0	1.5
7.5 to 10.0	5.0	2.5
10.0 to 15.0	6.0	2.5
15.0 to 30.0	7.0	3.5
Greater than 30.0	8.0	5.0

Next, the cross section is to be determined for the channel section.

5. Assume a safe limiting velocity of flow, depending on the type of lining, as given below:
   - Cement concrete lining: 2.7 m/s
   - Brick tile lining or burnt tile lining: 1.8 m/s
   - Boulder lining: 1.5 m/s

6. Assume the appropriate values of flow friction coefficients. Since Manning's equation would usually be used for calculating the discharge in canals, values of Manning's roughness coefficient, ***n***, from the following table may be considered for the corresponding type of canal lining.

Surface Characteristics	Value of ***n***
Concrete with surfaces as:	
a) Formed, no finish/PCC tiles or slabs	0.018-0.02
b) Trowel float finish	0.015-0.018
c) Gunited finish	0.018-0.022
Concrete bed trowel finish with sides as:	0.019-0.021
a) Hammer dressed stone masonry	
b) Course rubble masonry	0.018-0.02
c) Random rubble masonry	0.02-0.025
d) Masonry plastered	0.015-0.017
e) Dry boulder lining	0.02-0.03
Brick tile lining	0.018-0.02

7. The longitudinal slope (S) of the canal may vary from reach to reach, depending upon the alignment. The slope of each reach has to be evaluated from the alignment of the canal drawn on the map of the region.

8. For the given discharge Q, permissible velocity V, longitudinal slope S, given side slope , and Manning' roughness coefficient, ***n,*** for the given canal section, find out the cross section parameters of the canal, that is, bed width (B) and depth of flow (D).

Since two unknowns are to be found, two equations may be used, which are:

- Continuity equation: $Q = A * V$ (6)
- Dynamic equation: $V = \frac{1}{n}(A\, R^{2/3} S^{1/2})$ (7)

In the above equations, all variables stand for their usual notation as mentioned earlier, A and R is cross sectional area and hydraulic radius, respectively.

3.7.2 Typical sections of lined channels

Though there may be a large number of combinations of the factors on which the cross-section of a lined canal depends, some typical examples are given in the following figures, which may give an idea of laying and a practical channel cross section.

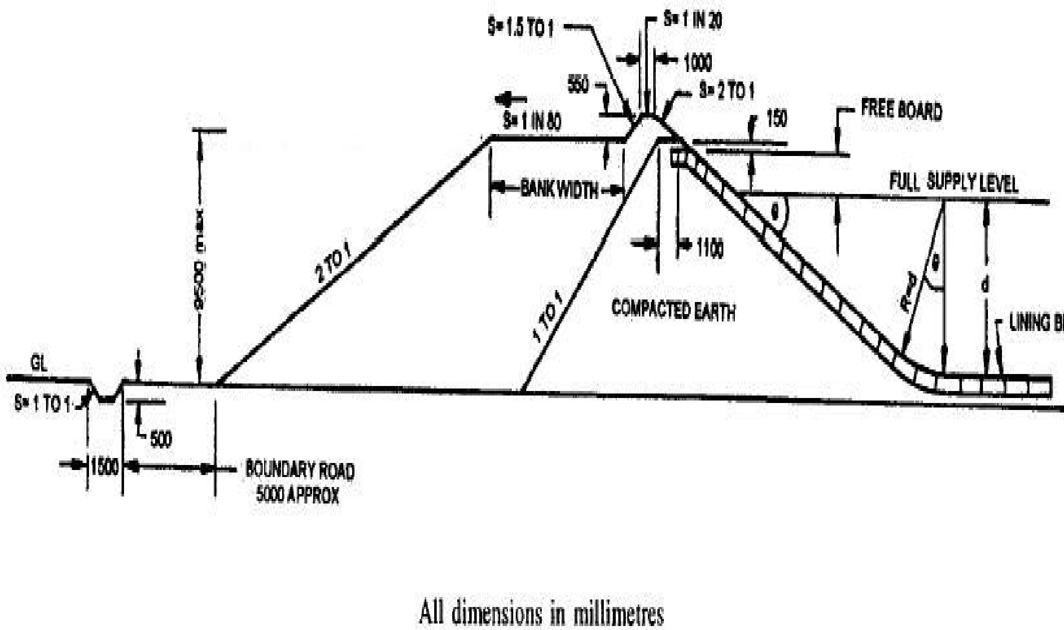
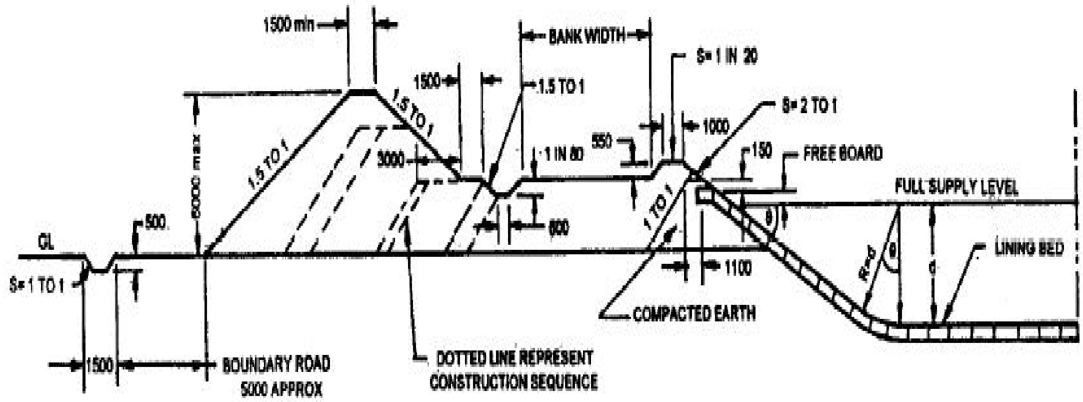
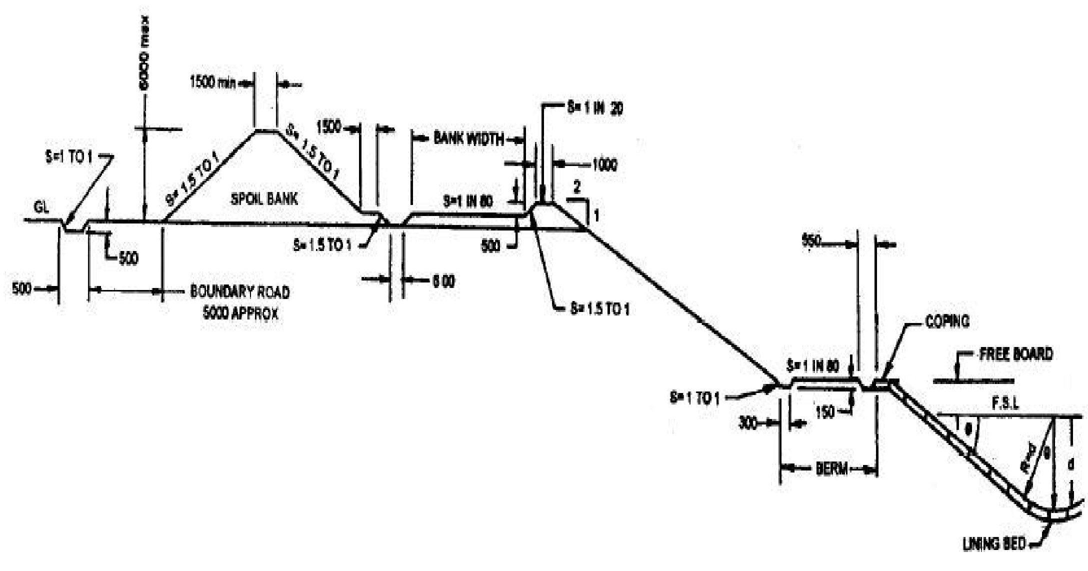


FIGURE 5 (a). Typical cross section of canal when natural ground level is below bed level



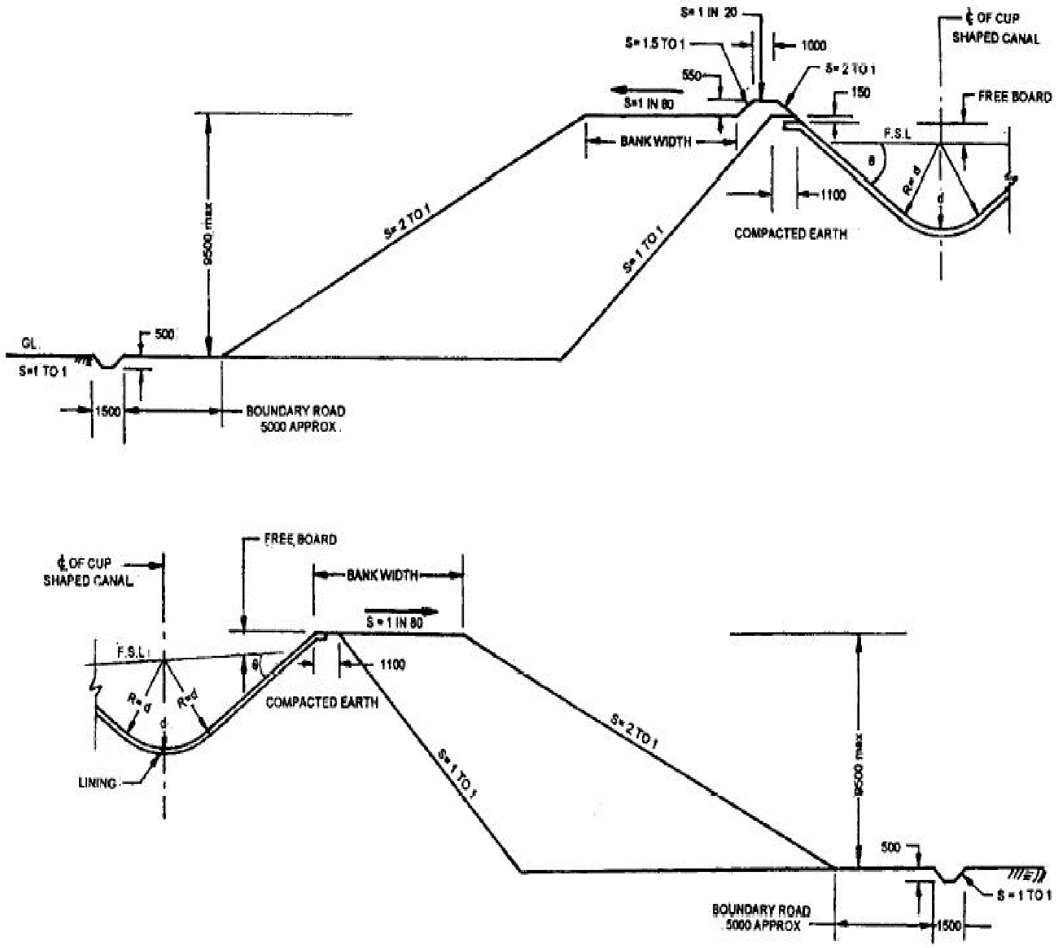
All dimensions in millimetres.

FIGURE 5 (b). Typical cross section of canal when natural ground level is between canal bed and full supply levels



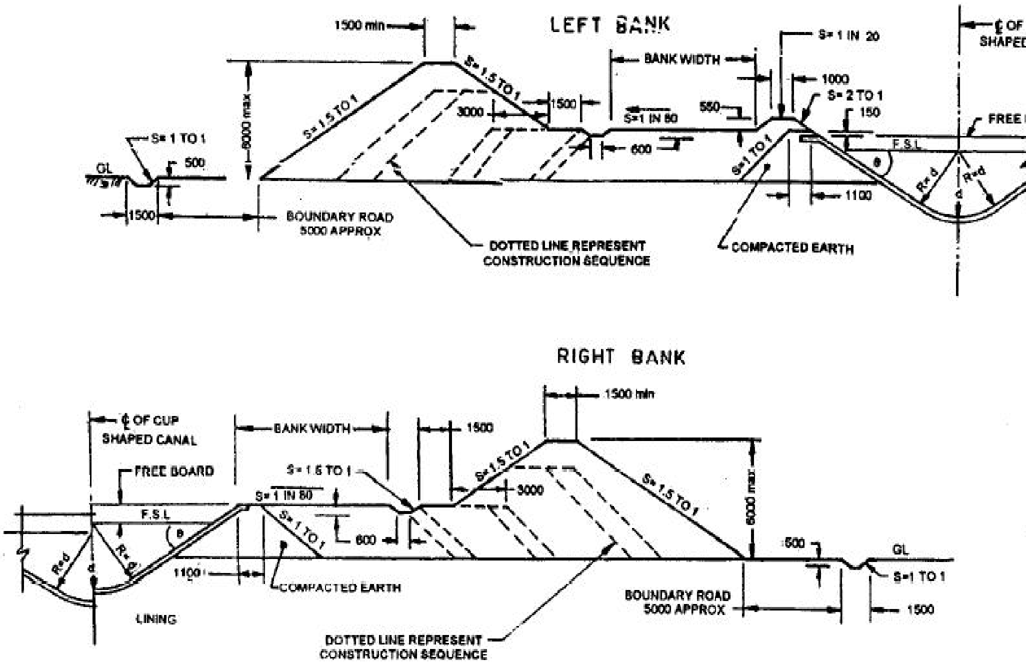
All dimensions in millimetres

FIGURE 5 (c). Typical cross section of canal when natural ground level is above the top level of lining



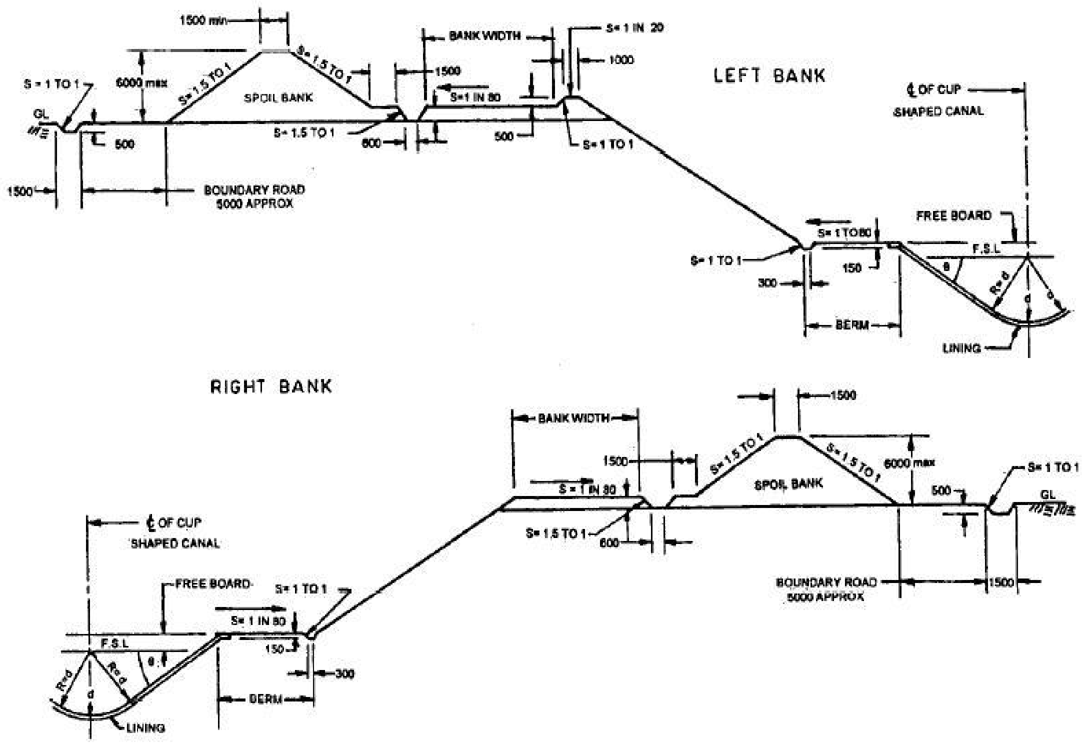
All dimensions in millimetres

FIGURE 5(d). Typical cross sections of canals when canal bed level is above natural ground level



All dimensions in millimetres.

FIGURE 5 (e). Typical cross sections of canals when natural ground level is between canal bed and full supply levels



All dimensions in millimetres

FIGURE 5 (f). Typical cross sections of canals when natural ground level is above top of canal lining

The Bureau of Indian Standard code IS: 10430-1982 “Criteria for design of lined canals and guidelines for selection of type of lining” (Reaffirmed in 1991) may generally be used, in addition to special codes like IS: 9451-1985 “Guidelines for lining of canals in expansive soils (first revision)” (Reaffirmed in 1991), which may be used under particular circumstances.

3.7.3 Subsurface drainage of lined canals

Lined canals passing through excavations may face a situation when the canal is dry and the surrounding soil is saturated, like when the ground table is very near the surface. Similar situation may occur for lined canals in filling when the confining banks become saturated, as during rains and the canal is empty under the circumstances of repair of lining or general closure of canal. The hydrostatic pressure built up behind the linings, unless released, causes heaving of the lining material, unless it is porous enough to release the pressure on its own. Hence, for most of the linings (except for the porous types like the boulder or various types of earth linings which develop inherent cracks), there is a need to provide a mechanism to release the back pressure of the water in the subgrade. This may be done by providing pressure relief valves, as shown in Figure 6.

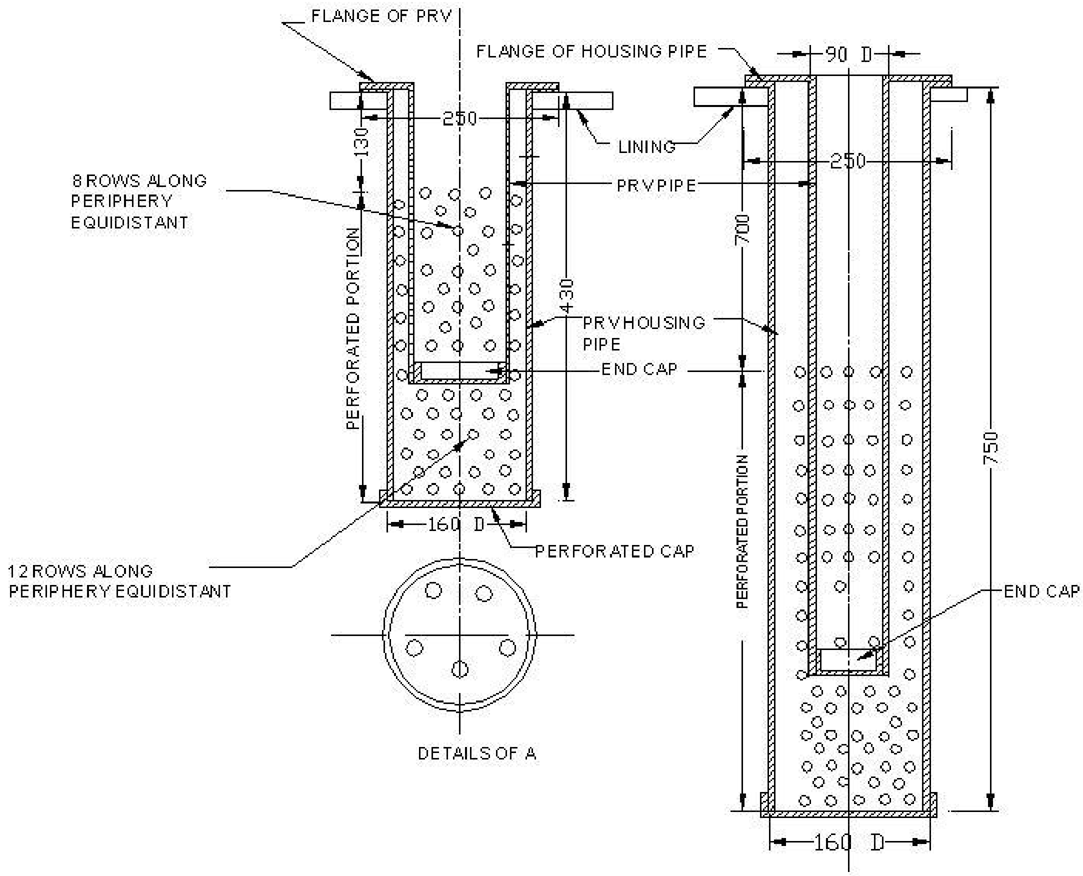


FIGURE 6a. Details of a Pressure Relief Valve (PRV)

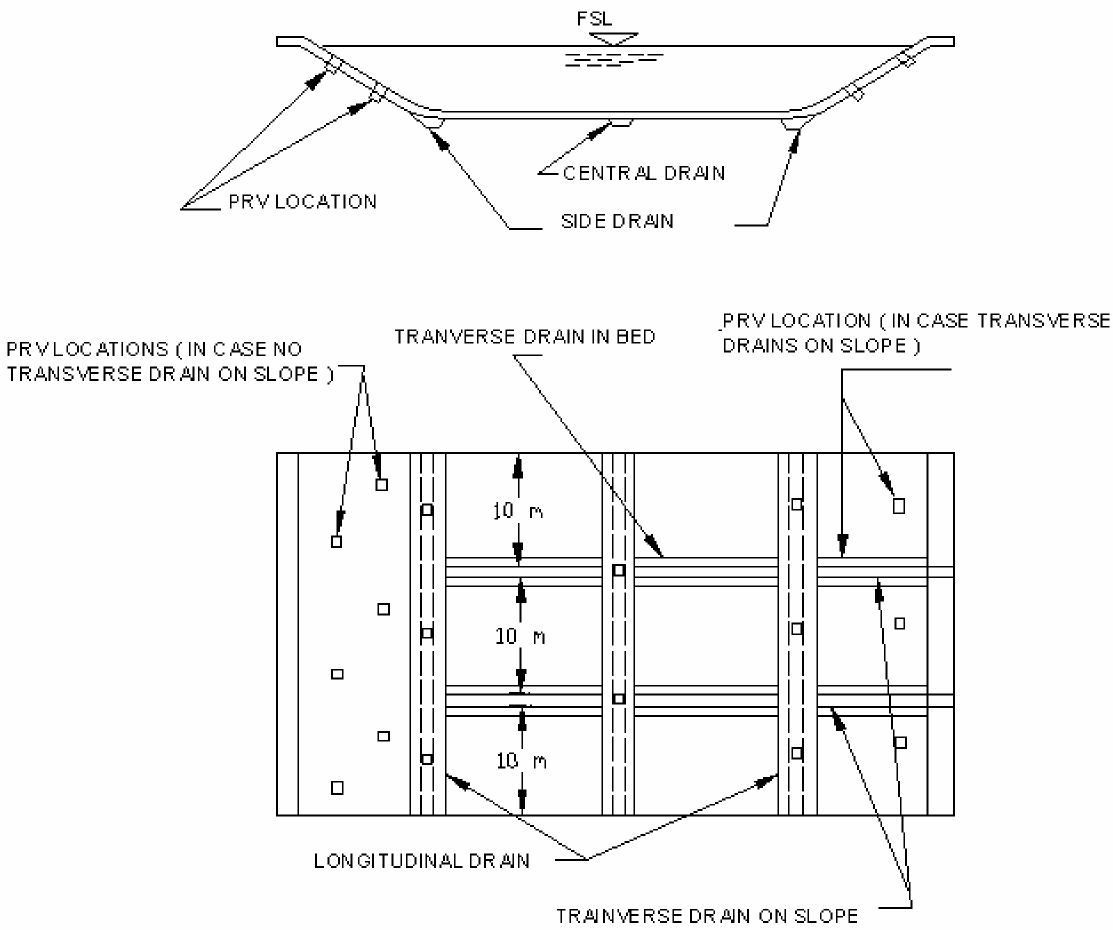


FIGURE 6b. Possible locations of PRVs

The Bureau of Indian Standard code IS: 4558-1983 "Code of practice for under design of lined canals" (First revision) discusses various methods for relieving uplift pressure below canal linings.

3.7.4 Design of unlined canals

The Bureau of Indian Standard code IS: 7112-1973 "Criterion for design of cross-section for unlined canals in alluvial soils" is an important document that may be consulted for choosing various parameters of an unlined channel, specifically in alluvial soils. There are unlined canals flowing through other types of natural material like silty clay, but formal guidelines are yet to be brought out on their design. Nevertheless, the general principles of design of unlined canals in alluvial soils are enumerated here, which may be suitably extended for other types as well after analyzing prototype data from a few such canals.

The design of unlined alluvial canals as compared to lined canals is more complex since here the bed slope cannot be determined only on the basis of canal layout, since there would be a limiting slope, more than which the velocity of the flowing water would start eroding the particles of the canal bed as well as banks. The problem becomes further complicated if the water entering the canal from the head-works is itself carrying sediment particles. In that case, there would be a limiting slope, less than which the sediment particles would start depositing on the bed and banks of the canal. In the following sections the design concept of unlined canals in alluvium for clear water as well as sediment-laden water is discussed separately.

3.7.4.1 *Unlined alluvial canals in clear water*

A method of design of stable channels in coarse non-cohesive material carrying clear water has been developed by the United States Bureau of Reclamation as reported by Lane (1955), which is commonly known as the Tractive Force Method. Figure 7 shows schematically shows such a situation where the banks are inclined to the horizontal at a given angle θ.

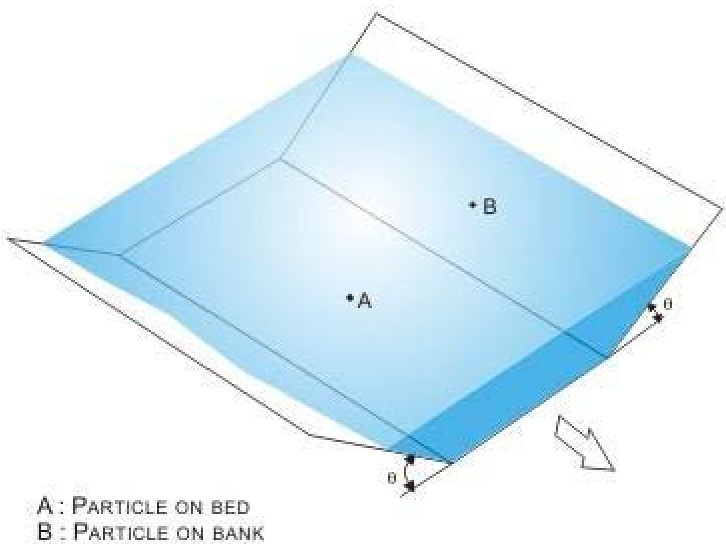


FIGURE 7. Trapezoidal shaped unlined alluvial canal
Two particles A and B are on the bed and bank

It is also assumed that the particles A and B both have the same physical properties, like size, density, etc. and also possess the same internal friction angle Φ. Naturally, the bank inclination θ should be less than Φ, for the particle B to remain stable, even under a dry canal condition. When there is a flow of water, there is a tendency for the particle A to be dragged along the direction of canal bed slope, whereas the particle B tries to get dislodged in an inclined direction due to the shear stress of the flowing water as shown in Figure 8.

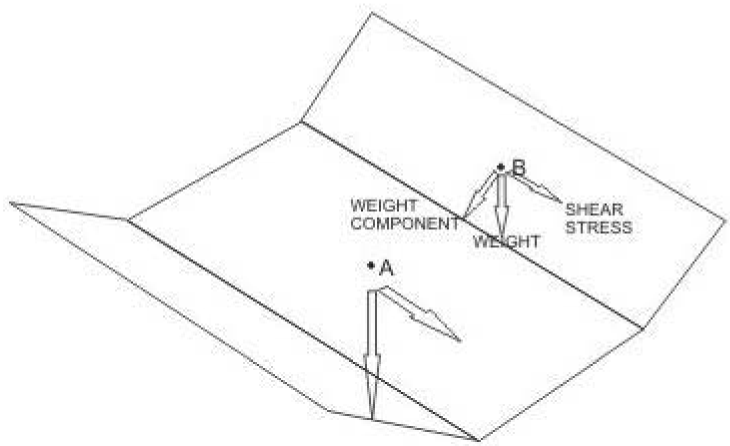


FIGURE 8. Shear stress and weight components of the particles A and B on the bed and bank, respectively

The particle A would get dislodged when the shear stress, τ, is just able to overcome the frictional resistance. This critical value of shear stress is designated as τ_C may be related to the weight of the particle, W, as

$$\tau_{C\,b} = W \tan \phi \qquad (8)$$

For the particle B, a smaller shear stress is likely to get it dislodged, since it is an inclined plane. In fact, the resultant of its weight component down the plane, W Sin □ and the shear stress (designated as $\tau_C{}'$) would together cause the particle to move. Hence, in this case,

$$(\tau_{CS})^2 + (W \sin \theta)^2 = \left[W \cos \theta\right] \tan \phi \qquad (9)$$

In the above expression it must be noted, that the normal reaction on the plane for the particle B is W Cos θ.

Eliminating the weight of the particles, W, from equations (8) and (9), one obtains,

$$\tau_{CS}{}^2 + \left[\frac{\tau_{Cb}}{\tan\phi}\sin\theta\right]^2 = \left[\frac{\tau_{Cb}}{\tan\phi}\cos\theta\tan\phi\right]^2$$

This simplifies to

$$\tau_{CS}{}^2 = \tau_{Cb}{}^2\left[\cos^2\theta - \frac{\sin^2\theta}{\tan^2\phi}\right]$$

Or

$$\frac{\tau_{CS}}{\tau_{Cb}} = \cos\theta\sqrt{1-\frac{\tan^2\theta}{\tan^2\phi}} \qquad (10)$$

As expected, τ_{CS} is less than τ_{Cb} , since the right hand side expression of equation (3) is less than 1.0. This means that the shear stress required moving a grain on the side slope is less than that required to move on the bed.

It is now required to find out an expression for the shear stress due to flowing water in a trapezoidal channel. From Lesson 2.9 it is known that in a wide rectangular channel, the shear stress at the bottom, τ_0 is given by the following expression

$$\tau_0 = \gamma\, R\, S \qquad (11)$$

Where γ is the unit weight of water, R is the hydraulic radius of the channel section and S is the longitudinal bed slope. Actually, this is only an average value of the shear stress acting on the bed, but actually, the shear stress varies across the channel width. Studies conducted to find the variation of shear stress have revealed interesting results, like the variation of maximum shear stress at channel base (τ_b) and sides (τ_S) shown in Figure 9 to 11.

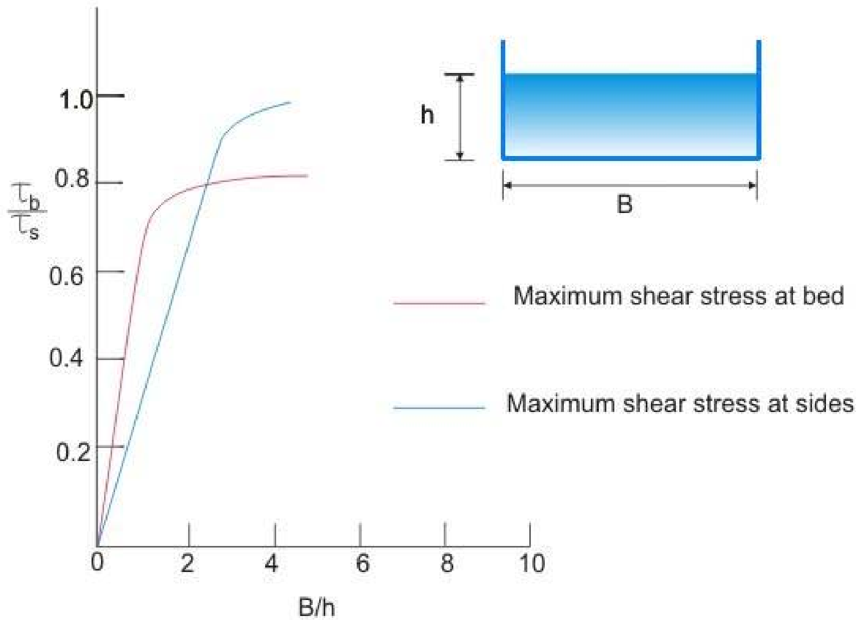


FIGURE 9. Variation of maximum shear stress for rectangular channel

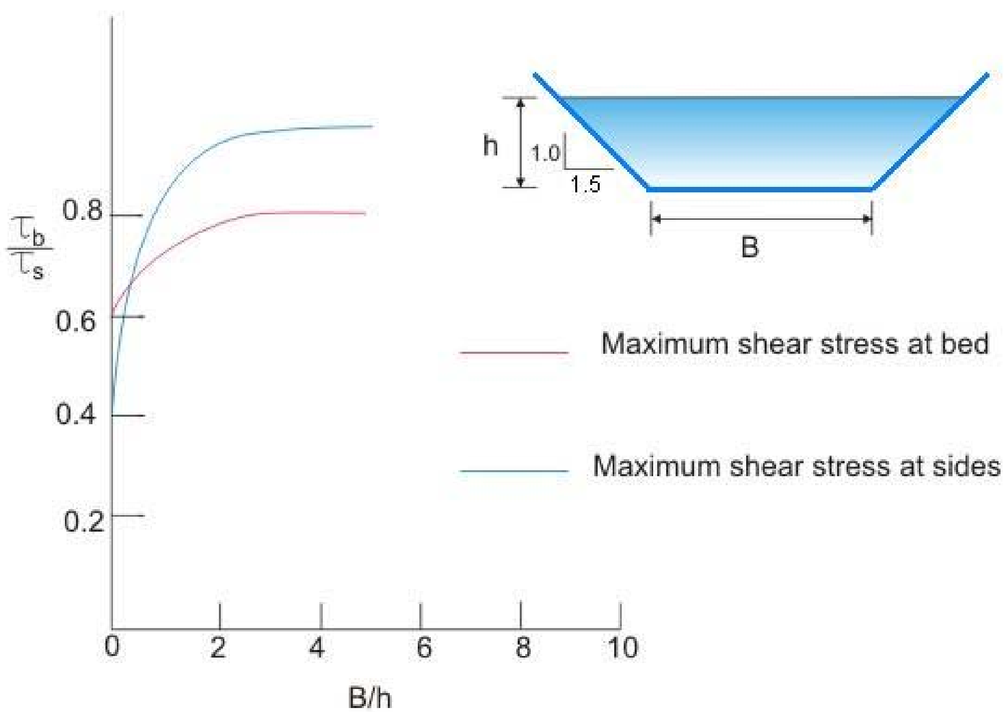


FIGURE 10. Variation of maximum shear stress fo trapezoidal channel with side slope 1V:I.5H.

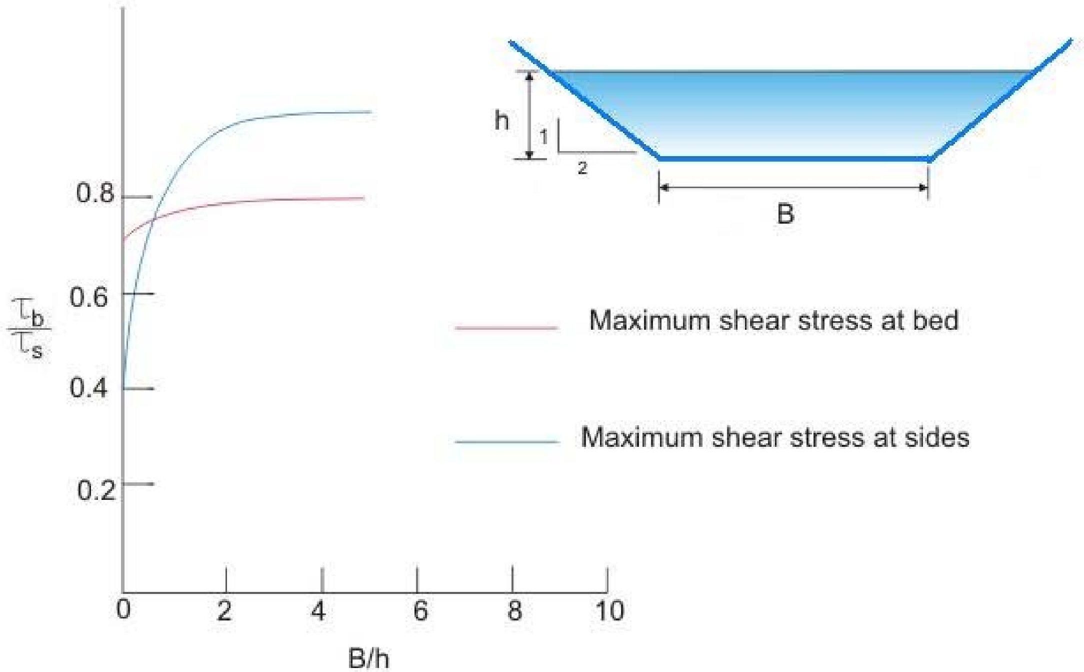


FIGURE 11. Variation of maximum shear stress of trapezoidal channel with side slope 1V : 2H

As may be seen from the above figures, for any type of channel section, the maximum shear stress at the bed is somewhat more than for that at the sides for a given depth of water (Compare τ_b and τ_S for same B/h value for any graph). Very roughly, for trapezoidal channels with a wide base compared to the depth as is practically provided, the bottom stress may be taken as γRS and that at the sides as 0.75 γRS. Finally, it remains to find out the values of B and h for a given discharge Q that may be passed through an unlined trapezoidal channel of given side slope and soil, such that both the bed and banks particles are dislodged at about the same time. This would ensure an optimum channel section.

Researchers have investigated for long, the relation between shear stress and incipient motion of non-cohesive alluvial particles in the bed of a flowing stream. One of the most commonly used relation, as suggested by Shields (1936), is provided in Figure 12.

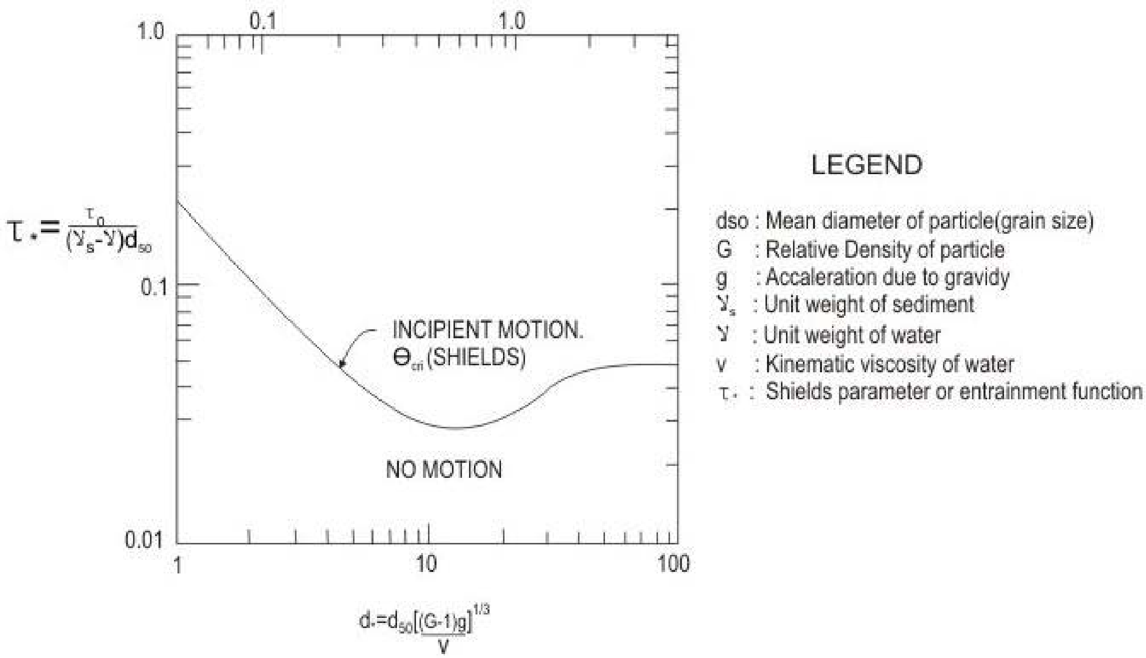


FIGURE 12. Curve for incipient motion

Swamee and Mittal (1976) have proposed a general relation for the incipient motion which is accurate to within 5 percent. For γ_s =2650 kg/m^3 and γ =1000kg/ m^3 the relation between the critical shear stress τ_C (in N/m^2) diameter of particle d_s (in mm) is given by the equation

$$\tau_C = 0.155 + \frac{0.409\, d_s^{\,2}}{\sqrt{1 + 0.177\, d_s^{\,2}}} \qquad (12)$$

The application of the above formula for design of the section may be illustrated with an example.

Say, a small trapezoidal canal with side slope 2H: 1V is to be designed in a soil having an internal friction angle of 35^0 and grain size 2mm. The canal has to be designed to carry 10m^3/s on a bed slope of 1 in 5000.

To start with, we find out the critical shear stress for the bed and banks. We may use the graph in figure (12) or; more conveniently, use Equation (12). Thus, we have the critical shear stress for bed, τ_{Cb}, for bed particle size of 2mm as:

$$\tau_C = \tau_{Cb} = 0.155 + \frac{0.409 \, * \, 2^2}{\sqrt{1 + 0.177 \, * \, 2^2}}$$
$$= 1.407 \text{ N/m}^2$$

The critical shear stress for the sloping banks of the canal can be found out with the help of expression (10). Using the slope of the banks (2H: 1V), which converts to θ= 26.6^0

$$\frac{\tau_{CS}}{\tau_{Cb}} = \text{Cos } 26.6^0 \sqrt{1 - \frac{\tan^2 26.6^0}{\tan^2 35^0}} = 0.625$$

From which,

$$\tau_{CS} = 0.625 \text{ x } 0.1.4068 = 0.880 \text{ N/m}^2$$

The values for the critical stresses at bed and at sides are the limiting values. One does not wish to design the canal velocity and water depth in such a way that the actual shear stress reaches these values exactly since a slight variation may cause scouring of the bed and banks. Hence, we adopt a slightly lower value for each, as:

Allowable critical shear stress for bed $\tau_{Cb}{}' = 0.9\,\tau_{Cb}$ = 1.266 N/m^2

Allowable critical shear stress for banks $\tau_{CS}{}' = 0.9\,\tau_{CS}$ = 0.792 N/m^2

The dimensions of the canal is now to be determined, which means finding out the water depth D and canal bottom B. for this, we have to assume a B/D ratio and a value of 10 may be chosen for convenience. We now read the shear stress values of the bed and banks in terms of flow variable 'R', the hydraulic radius, canal slope 'S' and unit weight of water γ from the figure-corresponding to a channel having side slope 2H: 1V. However, approximately we may consider the bed and bank shear stresses to be γ RS and 0.75 γ RS, respectively. Further, since we have assumed a rather large value of B/D, we may assume R to be nearly equal to D. this gives the following expressions for shear stresses at bed and bank;

Unit shear stress at bed = τ_b = γ D S = 9810 x D x $\frac{1}{5000}$ = 1.962 D N/m^2 per metre width.

Unit shear stress at bank τ_S = 0.75 γ D S = 0.75 x 9810 x D x $\frac{1}{5000}$ = 1.471 D N/m^2 per metre width.

For stability, the shear stresses do not exceed corresponding allowable critical stresses.

Thus,

$$(\tau_b =)1.962\, D < (\tau_{Cb}{}' =)1.266\, N/m^2$$
$$or\ D < 0.645\, m$$
$$and$$
$$(\tau_S =)1.471\, D < (\tau_{CS}{}' =)0.792 N/m^2$$
$$or\ D < 0.538\, m$$

Therefore, the value of D satisfying both the expression is the minimum value of the two, which means D should be limited to 0.538 m, say 0.53 m. Since the B/D ratio was chosen to be 10, we may assume B to be 5.3 m, or say, 5.5 m for practical purposes. For a trapezoidal shaped channel with side slopes 2H: 1V, we have

$$A = D\,(B+2D) = 3.445 \text{ m}^2$$

And $P = B + 2\sqrt{5}\,D = 7.87$ m

Thus $R = A/P = \dfrac{D\,(B+2D)}{B+2\sqrt{5}\,D} = 0.438\, m$

For the grain size 2mm, we may find the corresponding Manning's roughness coefficient 'n' using the Stricker's formula given by the expression

$$\boldsymbol{n} = \frac{d_s{}^{1/6}}{25.6}$$
$$= \frac{0.002^{1/6}}{25.6} = 0.014$$

Using the Manning's equation of flow, we have

$$Q = \frac{1}{n} A R^{2/3} S^{1/2}$$
$$= \frac{1}{0.014} \times 3.445 \times 0.438^{2/3} \times \left(\frac{1}{5000}\right)^{1/2}$$
$$= 2 \text{ m}^3\text{/s}$$

Since the value of Q does not match the desired discharge that is to be passed in the channel, given in the problem as 100m^3/s, we have to change the B/D ratio, which was assumed to be 10. Suppose we assume a B/D ratio of, say, k we obtain the following expression for the flow

$$Q = \frac{1}{n} A \left(\frac{A}{P}\right)^{2/3} S^{1/2}$$
$$= \frac{1}{n} \frac{A^{5/3}}{P^{2/3}} S^{1/2}$$

And substituting known values, we obtain

$$10 = \frac{1}{0.014} \times \frac{[D(k.D+2D)]^{5/3}}{[k.D+2\sqrt{5}\,D]^{2/3}} \times \left[\frac{1}{5000}\right]^{1/2}$$

Substituting the value of D as 0.53m, as found earlier, it remains to find out the value of k from the above expression. It may be verified that the value of k is evaluates to around 55, from which the bed width of the canal, B, is found out to be 29.15m, say, 30m, for practical purposes.

It may be noted that IS: 7112-1973 gives a list of Manning's ***n*** values for different materials. However, it recommends that for small canals (Q<15 m^3/s), ***n*** may be taken as 0.02. (In the above example, ***n*** **was** evaluated as 0.014 by Strickler's formula).

3.7.4.2 Unlined alluvial channels in sediment laden water

It is natural for channel carrying sediment particles along with its flow to deposit them if the velocity is slower than a certain value. Velocity in excess of another limit may start scouring the bed and banks. Hence, for channels carrying a certain amount of sediment may neither deposit, nor scour for a particular velocity. Observations by the irrigation engineers of pre-independence India of the characteristics of certain canals in north India that had shown any deposition or erosion for several years, led to the theory of regime channels, as explained in Lesson 2.10. These channels generally carry a sediment load smaller than 500ppm. The first regime equation was proposed by Kennedy in the year 1895, who was an engineer in the Punjab PWD. Lindley, another engineer in the Punjab proposed certain regime relations in 1919. Later these equations were modified by Lacey, who was at one time the Chief Engineer of the UP Irrigation Department. In 1929 he published a paper describing his findings, which have been quite popularly used in India. These have even been adopted by the Bureau of Indian Standards code IS: 7112-1973 'Criteria for design of cross section for unlined canals in alluvial soils" (Reaffirmed in 1990), which prescribes that the following equations have to be used:

$$S = \frac{0.003\ f^{5/3}}{Q^{1/6}} \qquad (13)$$

$$P = 4.75\sqrt{Q} \qquad (14)$$

$$R = 0.47\left(\frac{Q}{f}\right)^{1/3} \qquad (15)$$

Where the variables are as explained below:

- ***S:*** Bed slope of the channel
- ***Q:*** the discharge in m3/s
- ***P:*** wetted perimeter of the channel, in m
- ***R:*** Hydraulic mean radius, in m
- ***f:*** The silt factor for the bed particles, which may be found out by the following formula, in which d_{50} is the mean particle size in mm.

$$f = 1.76 \sqrt{d_{50}} \qquad (16)$$

The Indian Standard code IS: 7112-1973 has also recommended simplified equations for canals in certain parts of India by fitting different equations to data obtained from different states and assuming similar average boundary conditions throughout the region. These are listed in the following table.

S.No	Hydraulic Parameter	All Indian Canals	Punjab canals	UP canal	Bengal canals
1 2 3	S (Bed slope) P (wetted Perimeter) R (Hydraulic radius)				

It may be noted that the regime equations proposed by Lacey are actually meant for channels with sediment of approximately 500ppm. Hence, for canals with other sediment loads, the formula may not yield correct results, as has been pointed out by Lane (1937), Blench and King (1941), Simons and Alberts on (1963), etc. however, the regime equations proposed by Lacey are used widely in India, though it is advised that the validity of the equations for a particular region may be checked before applying the same. For example, Lacey's equations have been derived for non-cohesive alluvial channels and hence very satisfactory results may not be expected from lower reaches of river systems where silty or silty-clay type of bed materials are encountered, which are cohesive in nature.

Application of Lacey's regime equations generally involves problems where the discharge (***Q***), silt factor (***f***) and canal side slopes (***Z***) are given and parameters like water depth (***D***), canal bed width (***B***) or canal longitudinal slope (***S***) have to be determined or Conversely, if ***S*** is known for a given ***f*** and ***Z***, it may be required to find out ***B***, ***D*** and ***Q***.

3.7.5 Longitudinal section of canals:

The cross section of an irrigation canal for both lined and unlined cases was discussed in the previous sections. The longitudinal slope of a canal therefore is also known or is adopted with reference to the available country slope. However, the slope of canal bed would generally be constant along certain distances, whereas the local ground slope may not be the same. Further in Lesson 3.6, the alignment of a canal system was shown to be dependent on the topography of the land and other factors. The next step is to decide on the elevation of the bed levels of the canal at certain intervals along its route, which would allow the field engineers to start canal construction at the exact locations. Also, the full supply level (FSL) of the canal has to be fixed along its length, which would allow the determination of the bank levels.

The exercise is started by plotting the plan of the alignment of the canal on a ground contour map of the area plotted to a scale of 1 in 15,000, as recommended by Bureau of Indian standards code IS: 5968-1987 "Guidelines for planning and layout of canal system for irrigation" (Reaffirmed 1992). At each point in plan, the chainages and bed elevations marked clearly, as shown in Figure 13. The canal bed elevations and the FSLs at key locations (like bends, divisions, etc) are marked on the plan. It must be noted that the stretches AB and BC of the canal (in Figure 13) shall be designed that different discharges due to the offtaking major distributary. Hence, the canal bed slope could be different in the different stretches.

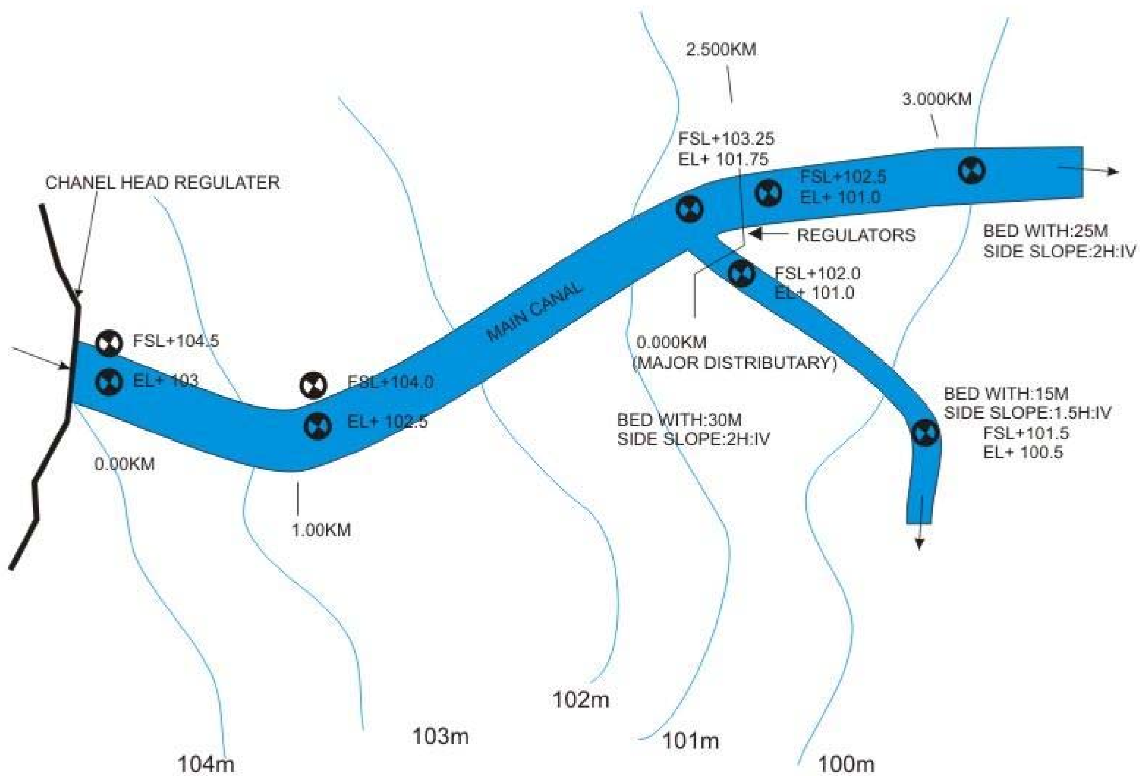


FIGURE 13. Typical layout of a canal showing bed and canal full supply levels

The determination of the FSL starts by calculating from the canal intake, where the FSL is about 1m below the pond level on the upstream of the canal head works. This is generally done to provide for the head loss at the regulator as the water passes below the gate. It is also kept to maintain the flow at almost at full supply level even if the bed is silted up to some extent in its head reaches. On knowing the FSL and the water supply depth, the canal bed level elevation is fixed at chainage 0.00KM, since this is the starting point of the canal. At every key location, the canal bed level is determined from the longitudinal slope of the canal, and is marked on the map. If there is no offtake between two successive key locations and no change in longitudinal slope is provided, then the cross-section would not be changed, generally, and accordingly these are marked by the canal layout.

At the offtakes, where a major or minor distributary branches off from the main canal, there would usually be two regulators. One of these, called the cross regulator and located on the main canal heads up the water to the desired level such that a regulated quantity of water may be passed through the other, the head regulator of the distributary by controlling the gate opening. Changing of the cross regulator gate opening has to be done simultaneously with the adjustment of the head regulator gates to allow the desired quantity of water to flow through the distributary and the remaining is passed down the main canal.

The locus of the full supply levels may be termed as the full supply line and this should generally kept above the natural ground surface line for most of its length such that most of the commanded area may be irrigated by gravity flow. When a canal along a watershed, the ground level on its either side would be sloping downward, and hence, the full supply line may not be much above the ground in that case. In stretches of canals where there is no offtake, the canal may run through a cutting within an elevated ground, and in such a case, the full supply line would be lower than the average surrounding ground level. In case irrigation is proposed for certain reaches of the canal where the adjacent ground level is higher than the supply level of the canal, lift irrigation by pumping may be adapted locally for the region.

Similarly, for certain stretches of the canal, it may run through locally low terrain. Here, the canal should be made on filling with appropriate drainage arrangement to allow the natural drainage water to flow below the canal. The canal would be passing over a water-carrying bridge, called aqueducts, in such a case.

As far as possible, the channel should be kept in balanced depth of cutting and filling for greatest economy and minimum necessity of ***borrow pits*** and ***spoil banks*.**

The desired canal slope may, at times, is found to be much less than the local terrain slope. In such a case, if the canal proceeds for a long distance, an enormous amount of filling would be required. Hence, in such a case, canal falls are provided where a change in bed elevation is effected by providing a ***drop structure*** usually an energy dissipater like hydraulic jump basin is provided to kill the excess energy gained by the fall in water elevation. At times, the drop in head is utilized to generate electricity through suitable arrangement like a bye-pass channel installed with a bulb-turbine.

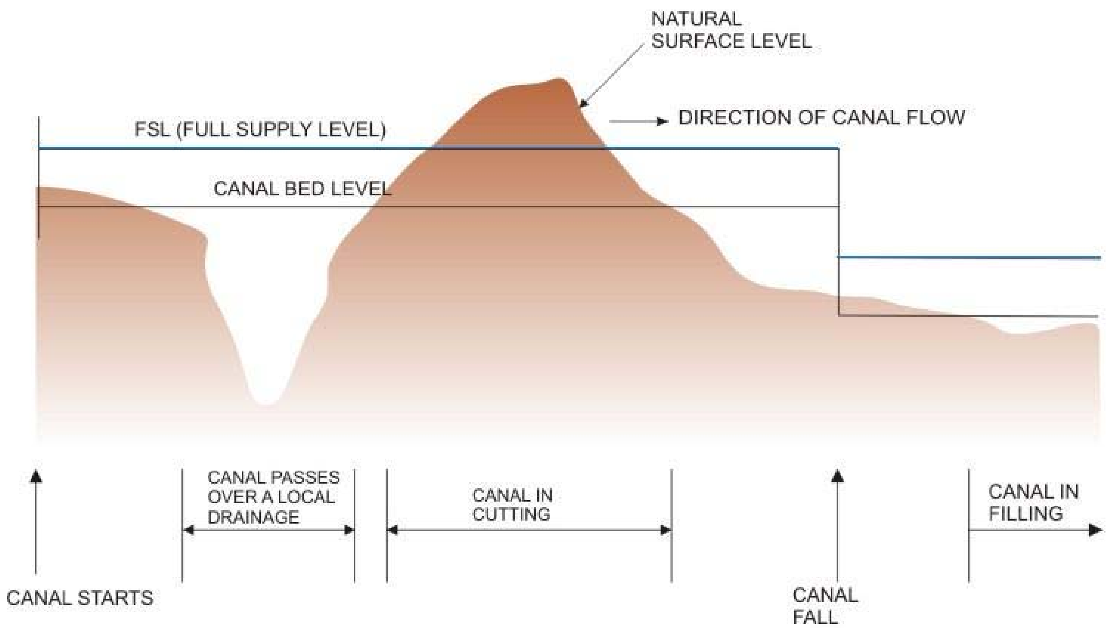


FIGURE 14. Longitudinal section of a canal assuming no withdrawals in this stretch

A typical canal section is shown in Figure 14, for a canal stretch passing through varying terrain profile. Here, no withdrawals have been assumed and hence, the discharge in the entire stretch of the canal is assumed to remain same. Hence, the canal bed slope and water depth are also not shown varying. It is natural that if the canal has outlets in between, the change in discharge would result in corresponding changes in the full supply line.

The elevation of the banks of the canal is found out by adding the ***freeboard*** depth. Though the free board depth depends upon many factors, the Bureau of Indian standards code IS: 7112-1973 “Criteria for design of cross sections for unlined canals in alluvial soils” recommends that a minimum free board of 0.5m be provided for canals carrying discharges less than 10m3/s and 0.75m for canals with higher discharges.

3.7.6 Important terms

Free Board: A depth corresponding to the margin of safety against overtopping of the banks due to sudden rise in the water level of a channel on account of accidental or improper opening or closing of gates at a regulator on the downstream.

Borrow pits*:* Specific site within a borrow area from which material is excavated for use is called a borrow pit.

Spoil Banks: Piles of soil that result from the creation of a canal, deepened channel, borrow pit, or some similar structure.

References:

- IS: 10430 -1982 "Criteria for design of lined canals and guidelines for selection of type of lining"
- IS: 4558-1983 "Code of practice for under design of lined canals" (First revision)
- IS: 5968-1987 "Guidelines for planning and layout of canal system for irrigation" (Reaffirmed 1992).
- IS: 7112-1973 "Criteria for design of cross-section for unlined canals in alluvial soils"
- IS: 9451-1985 "Guidelines for lining of canals in expansive soils" (first revision)
- Shields, A (1936) "Anwendung der Aehnlichkeitsmechanic und dser tuirbulenz-forchung auf die Geschiebebewgung", Mitteilungen der Pruessischen Versuchsanstalt fur wasserbau und Schiffbau, Berlin
- Swamee, P K and Mittal, M K (1976) "An explicit equation for critical shear stress in alluvial streams", CBIP Jnl of Irrigation and Power, New Delhi.

Module 3

Irrigation Engineering Principles

Lesson
8
Conveyance Structures for Canal Flows

Instructional objectives

On completion of this lesson, the student shall learn the following:

1. The need for structures of a canal irrigation system for conveying water from one point to another.
2. Structures for conveying water across, over or under natural streams
3. Transitions in canals at change of cross section

3.8.0 Introduction

A canal conveying water from the head works has to run for large distances and has to maintain the water levels appropriately, as designed along its length. It has to run through terrains which generally would have a different slope small than the canal . The surrounding areas would invariably have its own drainage system ranging from small streams to large rivers . The canal has to carry the water across these water bodies as well as across artificial obstacles like railway line or roads .

The main structures of a canal system for conveyance of canal flow and control of water levels are as follows .

1. Pipe conduits, culverts and inverted syphons to carry flow under railways and highways .

2. Aqueducts, syphon aqueducts, super-passage, canal siphon or level crossings across natural drainage courses or other depressions.

3. Transitions at changes in cross sections.

This lesson deals with the concepts of planning, layout and design of canal structures for flow conveyance across artificial and natural obstacles .

3.8.1 Structures for crossing canals across roads and railway lines

These are structural elements to convey canal water under roads or railway lines. For small roads, carrying relatively less traffic, the pipe conduit is sufficient. A general view of the pipe conduit is shown in Figure 1 and its typical plan and cross section in Figure 2. For canals crossing under major highways and railway tracks, reinforced concrete culverts are more commonly adopted. These roads or railway crossings are usually having a straight profile along its length. The water level in the canal for this type of crossing is lower than the level of the obstruction it crosses, as may be noticed from Figure 2 and the flow through the pipe may be free or under mild pressure.

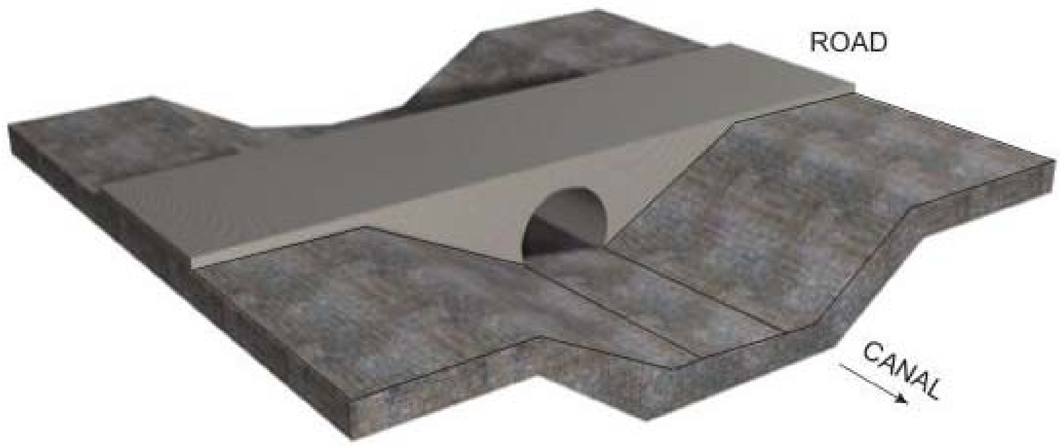


FIGURE 1. Pipe conduits for canals crossing small roads

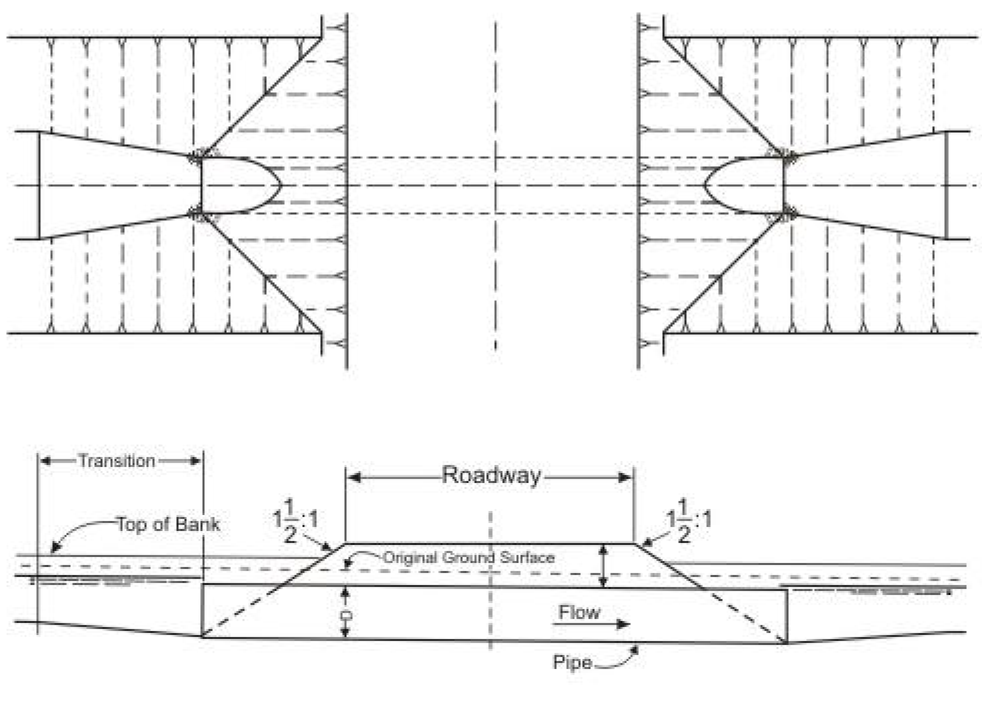


FIGURE 2. Plan and section of road crossing with pipe conduit

Pipe road crossings are relatively economical, easily designed and built, and have proven a reliable means of conveying water under a roadway. Pipe installations are normally installed by cut and cover method below minor roads but for important roads, where traffic cannot be interrupted, it may be accomplished by jacking the pipe through the roadway foundation.

The inverted syphons are structures for canal water conveyance below roads, railway lines and other structures (Figure 3). The longitudinal profile is not exactly in a straight line and the central portion is seen to sag beneath the object to be crossed. The inverted syphon, therefore, is provided where the water level in the canal is about the same as the level of the obstruction (Figure 4).

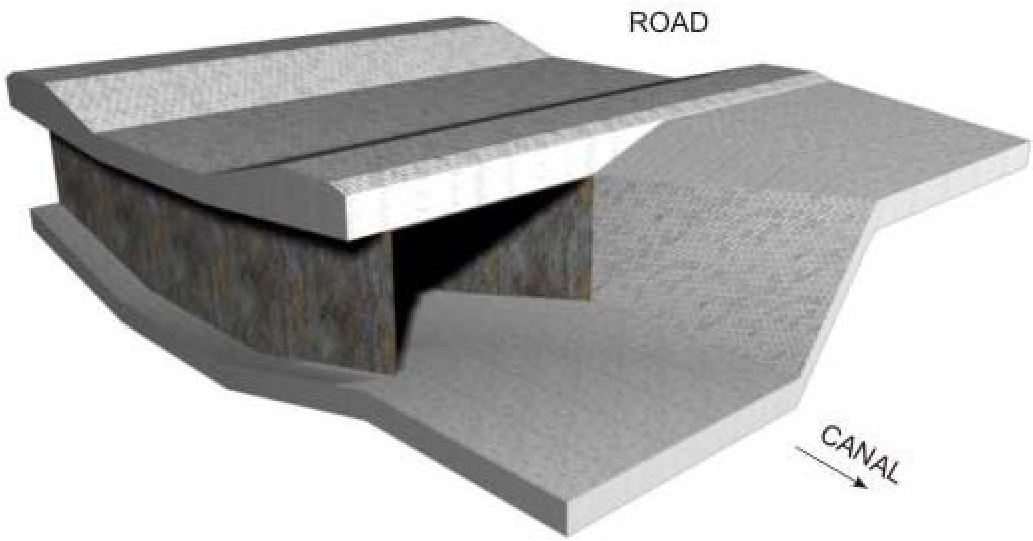


FIGURE 3. Inverted Syphon below roads showing rectangular section.
Circular section also possible

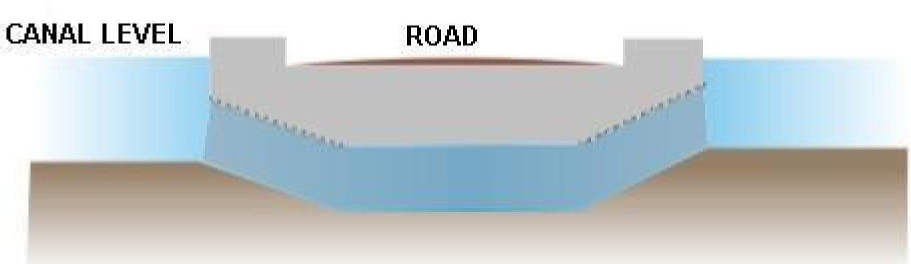


FIGURE 4a. Canal Full Supply Level(FSL) and road level are nearly same

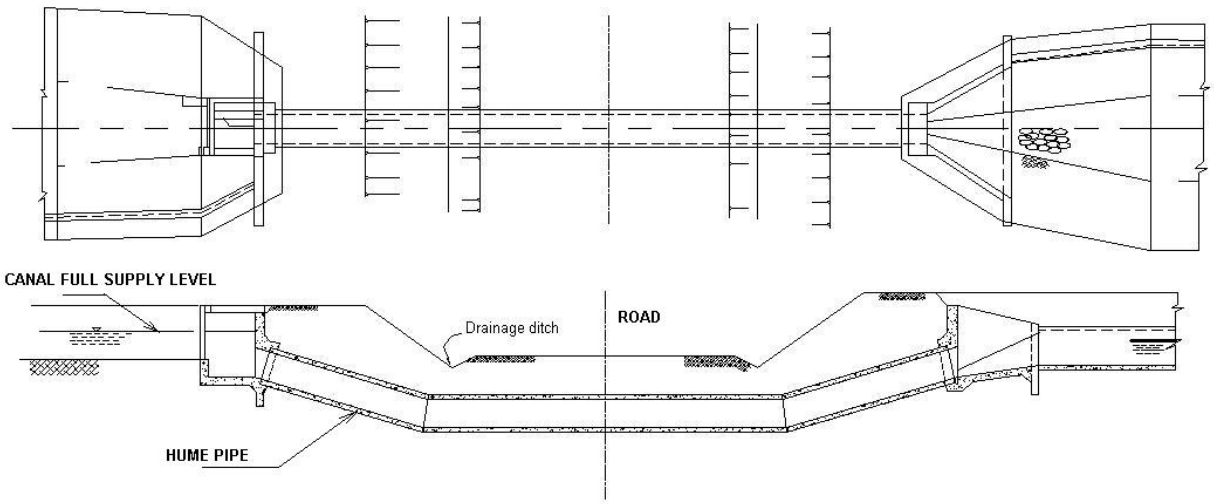


FIGURE 4b. An example of an inverted syphon of a small canal crossing a road

The inverted syphon is a closed conduit designed to run full and under pressure . If made of pressure pipes , they should be able to withstand the load of cover and wheel from outside and the hydrostatic head from inside . Transitions for changes in cross sections are nearly always used at inlet and outlet of a siphon to reduce head losses and prevent erosion in unlined canals caused by the velocity changes between the canal and the pipe.

3.8.2 Structures for crossing canals across natural streams (cross drainage works)

These structural elements are required for conveying the canals across natural drainage. When a canal layout is planned, it is usually seen to cross a number of channels draining the area, varying from small and shallow depressions to large rivers. It is not generally possible to construct cross-drainage structures for each of the small streams. Some of the small drainage courses are, therefore, diverted into one big channel and allowed to cross the canal. However, for larger streams and river, where the cost of diversion becomes costlier than providing a separate cross-drainage work, individual structures to cross the canal across the stream is provided.

There could be a variety of combinations of the relative position of the canal with respect the natural channel that is to be crossed. These conditions are shown in Figures 5 to 9. The notations used in the figures are as follows:

(a) CBL: Canal Bed Level;

(b) SBL: Stream Bed Level;
(c) FSL: Canal Full Supply Level; and
(d) HFL: Stream High Flood Level

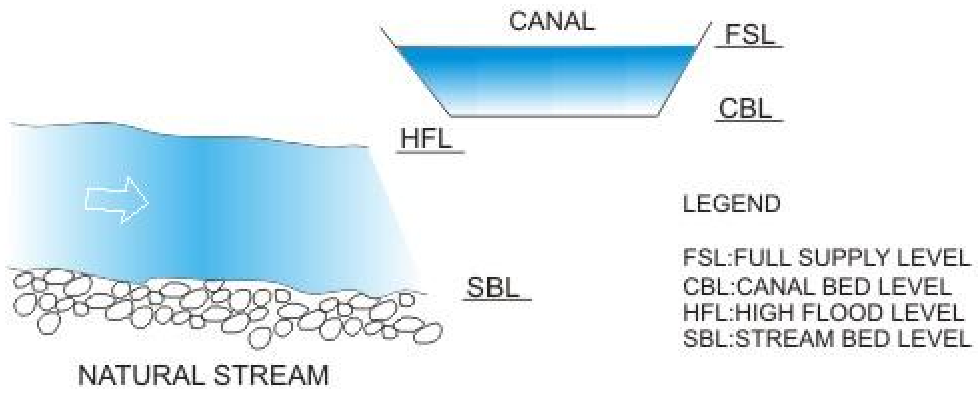


FIGURE 5. Relative position of canal (shown in cross section) with respect to a natural stream (shown in longitudinal section), when canal bed level is higher than stream high flood level

Figure 5 shows the relative position of canal (shown in cross-section) with respect to a natural stream (shown in longitudinal section), when canal bed level is higher than stream high flood level.

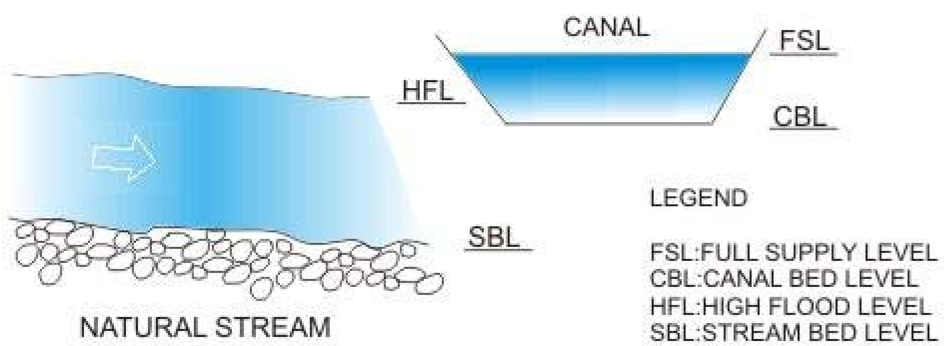


FIGURE 6 . Relative position of a canal whose canal bed level is below the high flood of the stream.

Figure 6 shows the relative position of a canal whose bed level is below but full supply level is above the stream high flood level.

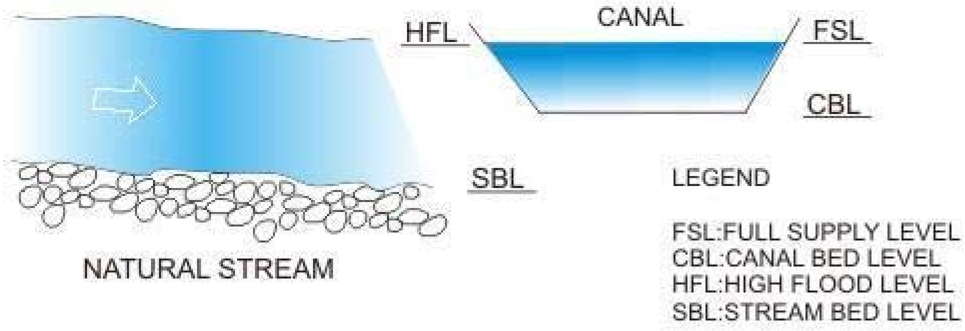


FIGURE 7 . Canal with full supply level almost matching the high flood level of the natural stream

Figure 7 shows a canal with full supply level almost matching the high flood level of the natural stream.

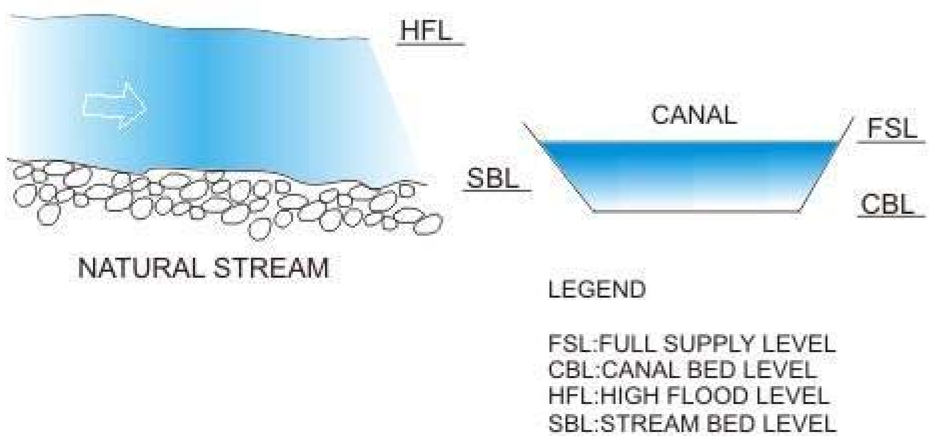


FIGURE 8 . Canal full supply level and bed levels below the levels of high flood level and bed level of the stream, respectively.

Figure 8 shows a canal full supply level and bed levels below the levels of high flood level and bed level of stream, respectively.

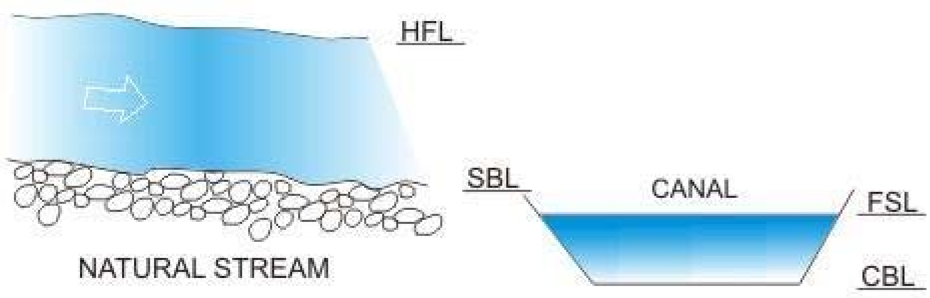


FIGURE 9 . Relation position of canal with respect to the natural stream where the canal full supply level is below the stream bed level

Figure 9 shows the relative position of canal with respect to the natural stream where the canal full supply level is below the stream bed level.

In general , the solution for all the illustrated conditions possible for conveying an irrigation canal across a natural channel is by providing a water conveying structure which may:

(a) Carry the canal over the natural stream;
(b) Carry the canal beneath the natural stream; or
(c) Carry the canal at the same level of the natural stream.

These three broad types of structures are discussed further in this lesson .

3.8.3 Structures to carry canal water over a natural stream

Conveying a canal over a natural watercourse may be accomplished in two ways:

(a) Normal canal section is reduced to a rectangular section and carried across the natural stream in the form of a bridge resting on piers and foundations (Figure 10) . This type of structure is called a ***trough type aqueduct*** .
(b) Normal canal section is continued across the natural stream but the stream section is flumed to pass through 'barrels' or rectangular passages (Figure 11). This type is called a ***barrel type aqueduct***.

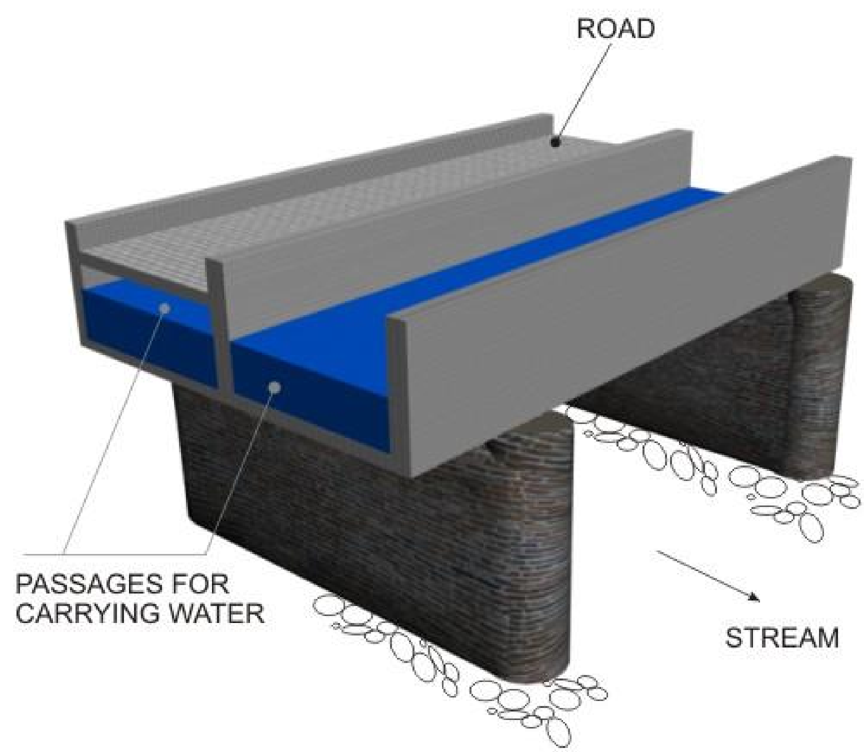


FIGURE 10. Trough type aqueduct

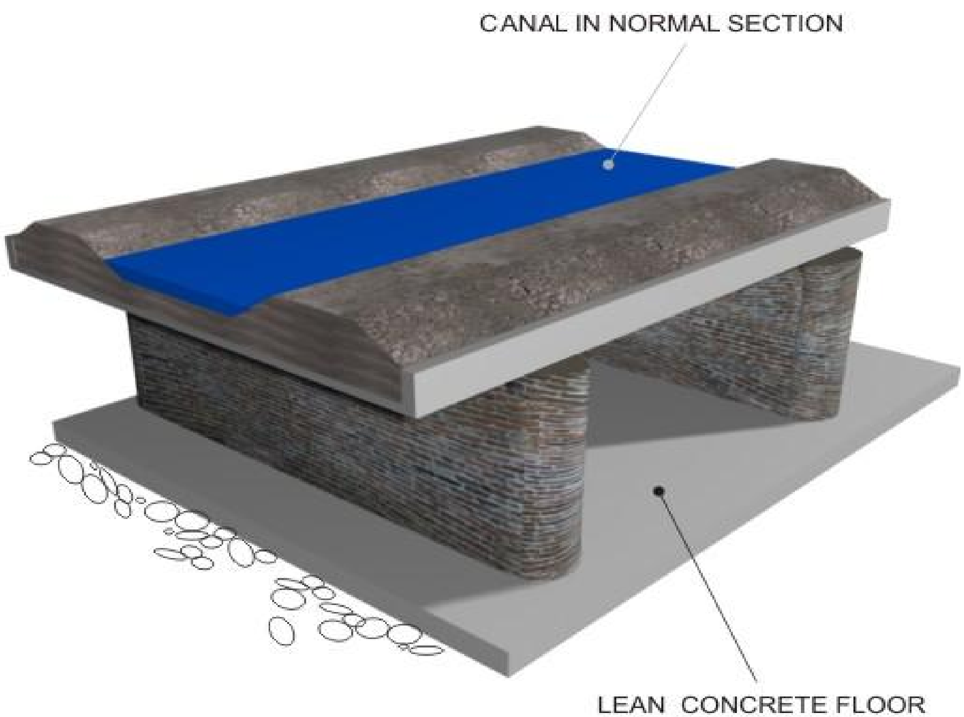


FIGURE 11. Barrel type aqueduct

Typical sections and plans of a trough type and a barrel type aqueducts are shown in Figures 12 and 13 respectively.

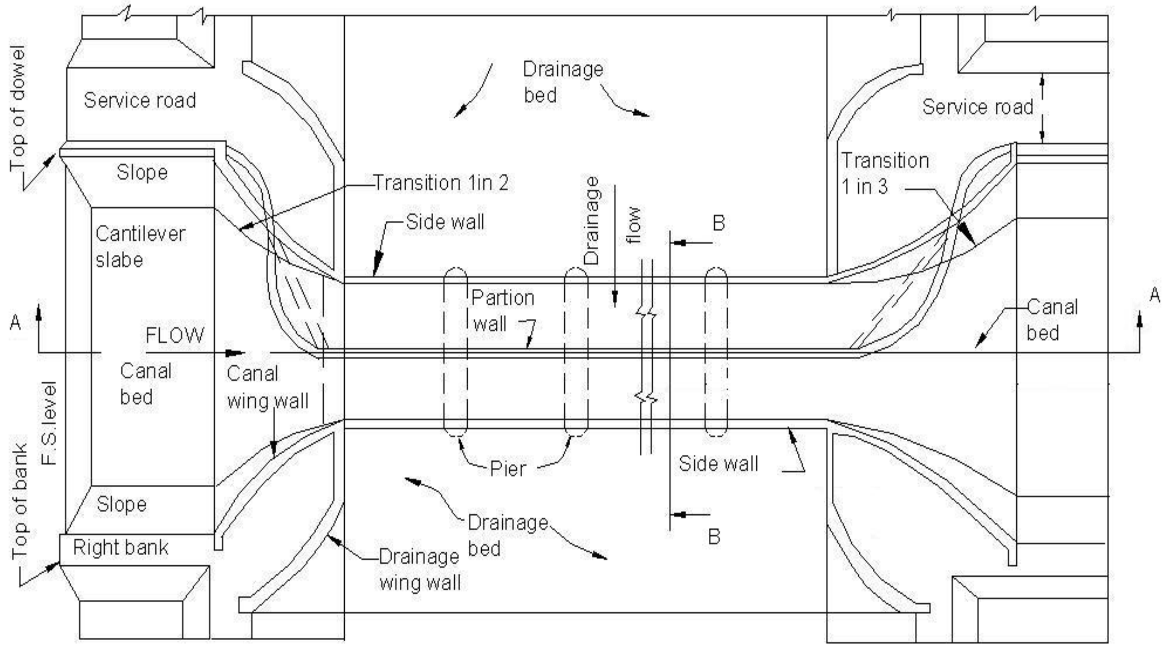


FIGURE 13a. A typical plan of a barrel type aqueduct

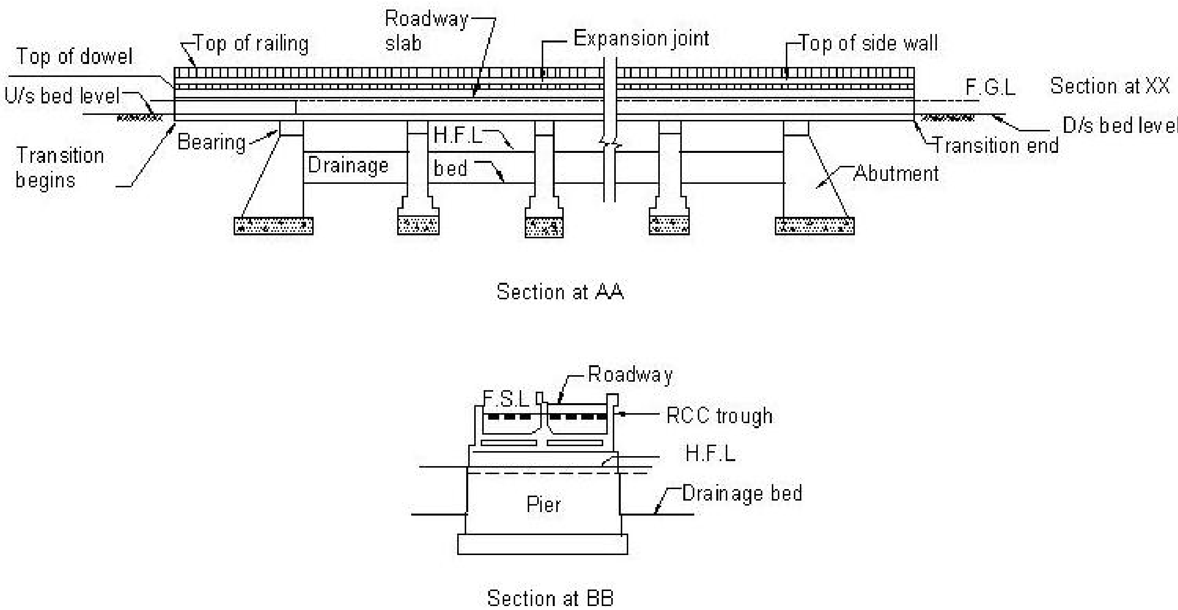


FIGURE 13b. Cross sections of the barrel type aqueduct shown in Fig. 13a.

For the aqueducts, it may be observed from Figures 12 and 13 that the HFL of the natural stream is lower than the bottom of the trough (or the roof of the barrel). In this case, the flow is not under pressure, that is, it has a free surface exposed to atmospheric pressure.

In case the HFL of the natural stream goes above the ***trough bottom level*** (TBL) or the ***barrel roof level*** (BRL), then the flow in the natural watercourse would be pressured and the sections are modified to form which is known as ***syphon aqueducts*** (Figures 14 and 15).

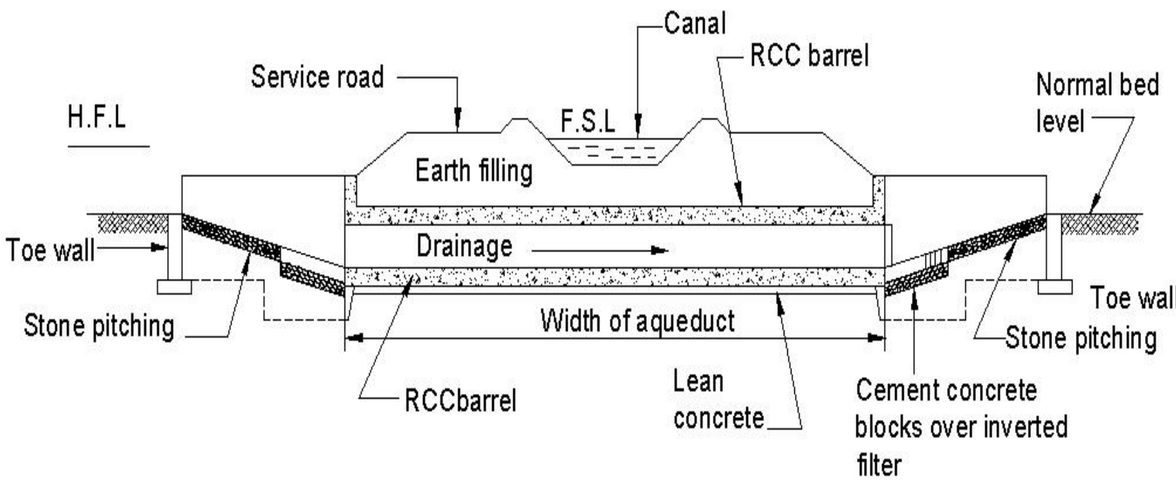


FIGURE 14. Section through a syphon aqueduct showing condition of pressured flow in natural drain

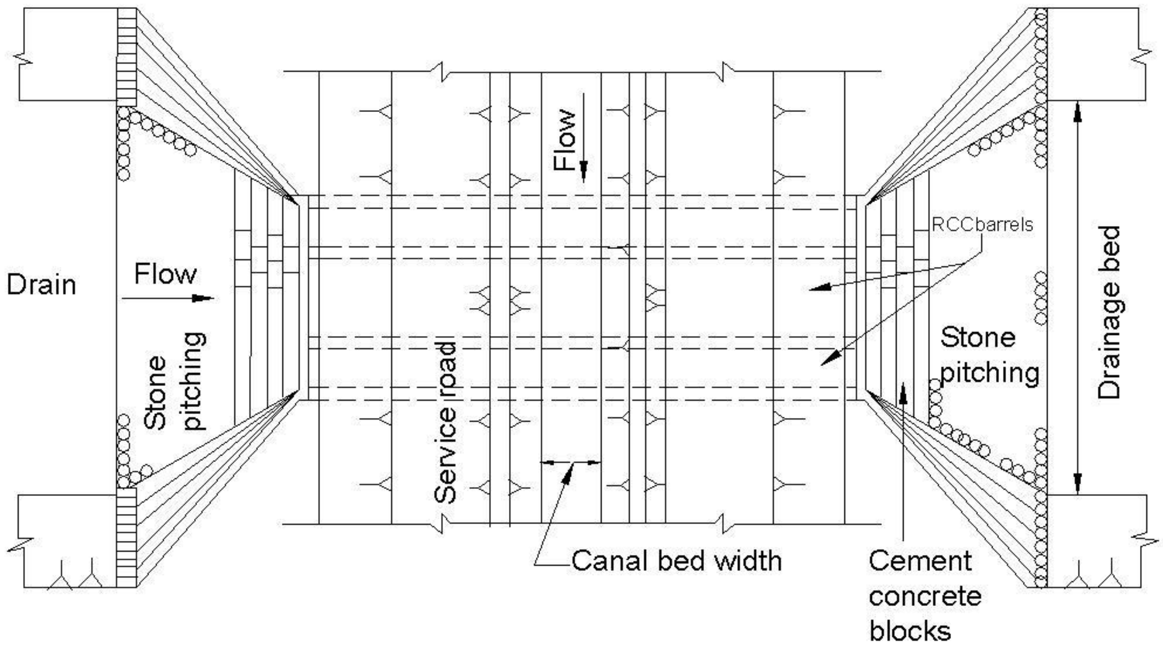


FIGURE 15. Plan of a syphon aqueduct if flow in natural drain is pressured

It may be observed that the trough type aqueduct or syphon aqueduct would be suitable for the canal crossing a larger stream or river, whereas the barrel type is suitable if the natural stream is rather small. The relative economics of the two types has to be established on case to case basis.

Further, the following points maybe noted for the two types of aqueducts or siphon aqueducts:

Trough type: The canal is flumed to not less than 75 percent of the bed width keeping in view the permissible head loss in the canal .Transitions 3:1 on the upstream and 5:1 on the downstream side are provided to join the flumed section to the normal canal section . For the trough-type syphon aqueduct the designer must consider the upward thrust also that might act during high floods in the natural stream when the stream water flows under pressure below the trough base and for worst condition , the canal may be assumed to be dry at that time. The dead weight of the trough may be made more than that of the upward thrust or it may be suitably anchored to the piers in order to may be counteract the uplift condition mentioned .

Barrel type: The barrel may be made up of RCC, which could be single or multi-cell, circular or rectangular in cross section. Many of the earlier structures were made of masonry walls and arch roofing. Precast RCC pipes may be economical for small discharges. For barrel-type syphon aqueducts, the barrel is horizontal in the central portion but slopes upwards on the upstream and downstream side at about an inclination of 3H : 1V and 4H : 1V respectively. A self-cleaning velocity of 6m/s and 3m/s is considered while designing RCC and masonry barrels respectively.

3.8.4 Structures to carry canal water below a natural stream

A canal can be conveyed below a natural stream with the help of structures like a ***super-passage*** or a siphon. These are exactly opposite in function to that of the aqueducts and siphon aqueducts, which are used to carry the canal water above the natural stream. The natural stream is flumed and made to pass in a trough above the canal. If the canal water flows with a free surface, that is, without touching the bottom of the trough, it is called a ***super-passage*** (Figure 16). Else, when the canal passes below the trough as a pressure flow, then it is termed as a ***syphon*** or a ***canal syphon***.

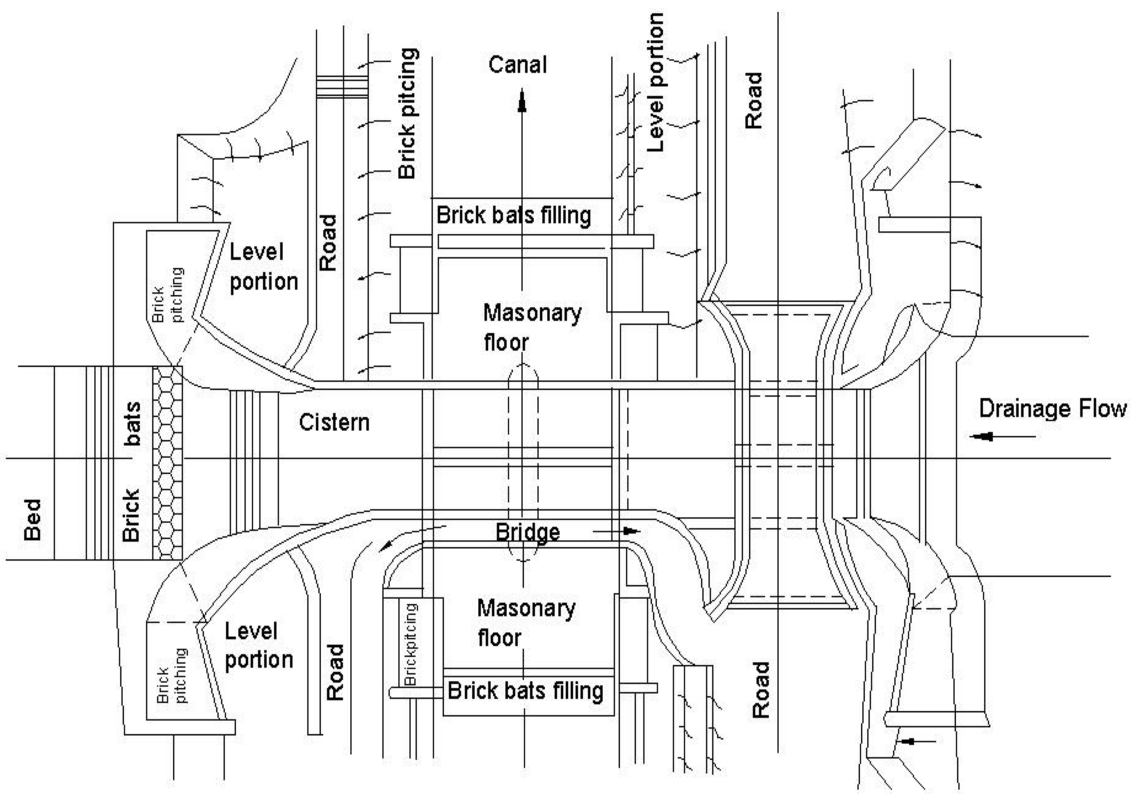


FIGURE 16a. Typical layout of a Super-passage

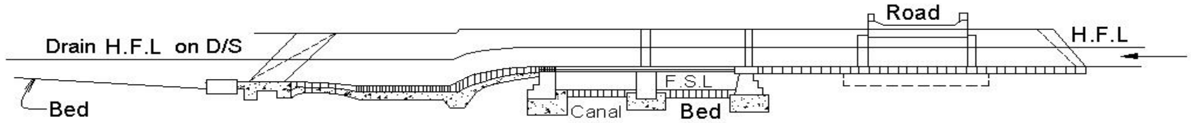


FIGURE 16b. Section through the Super-passage shown in Figure 16a.

Instead of a trough, the canal flow may be conveyed below the natural stream using small pre-cast RCC pipes (for small discharges) and rectangular or circular barrels, either in single or multiple cells, may be used (for large discharges), as shown in Figure

17.

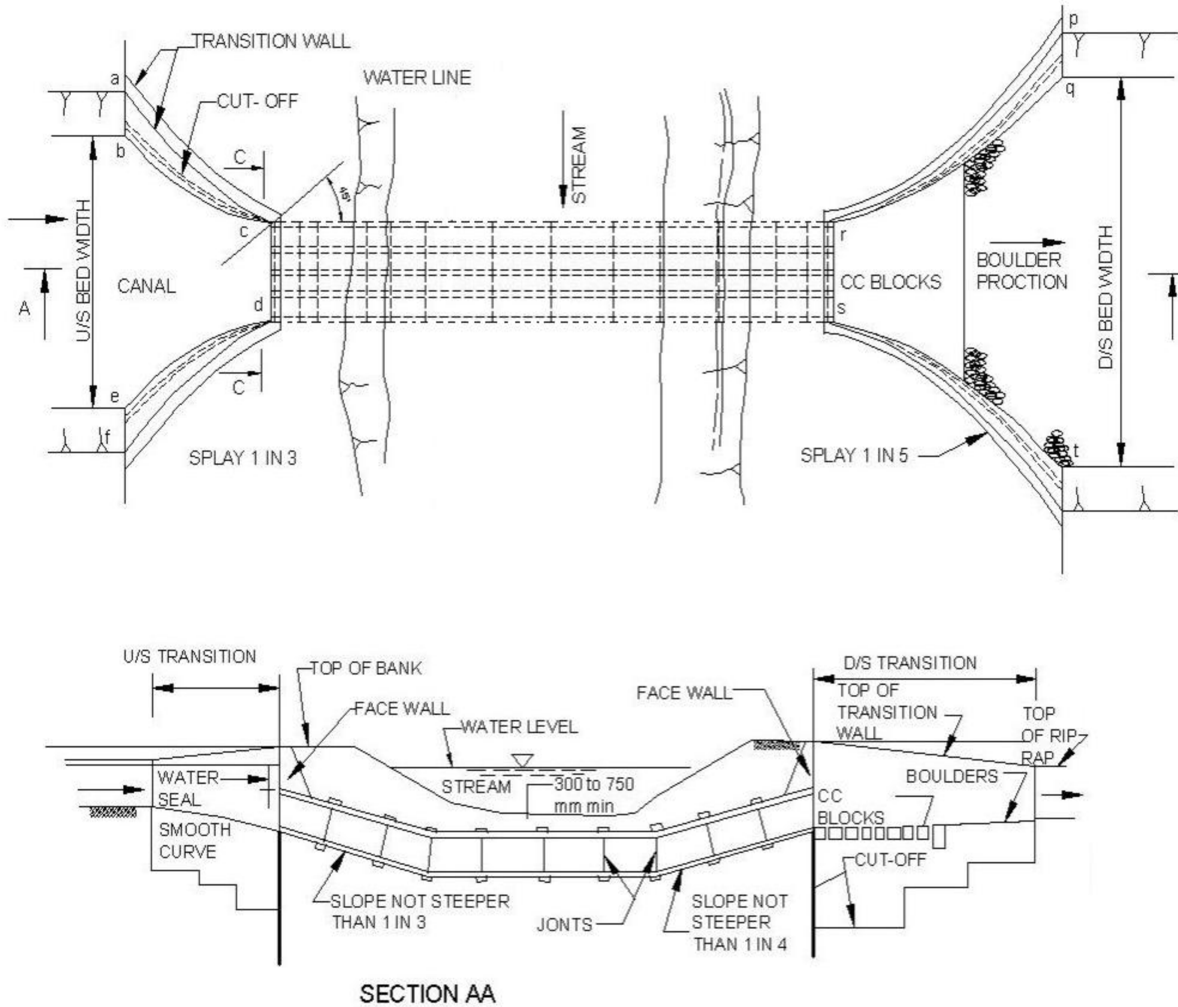


FIGURE 17. Plan and section of canal siphon

3.8.5 Structures to carry canal water at the same level as a natural stream

A structure in which the water of the stream is allowed to flow into the canal from one side and allowed to leave from the other, known as a ***level crossing*** , falls into this category (Figure 18).

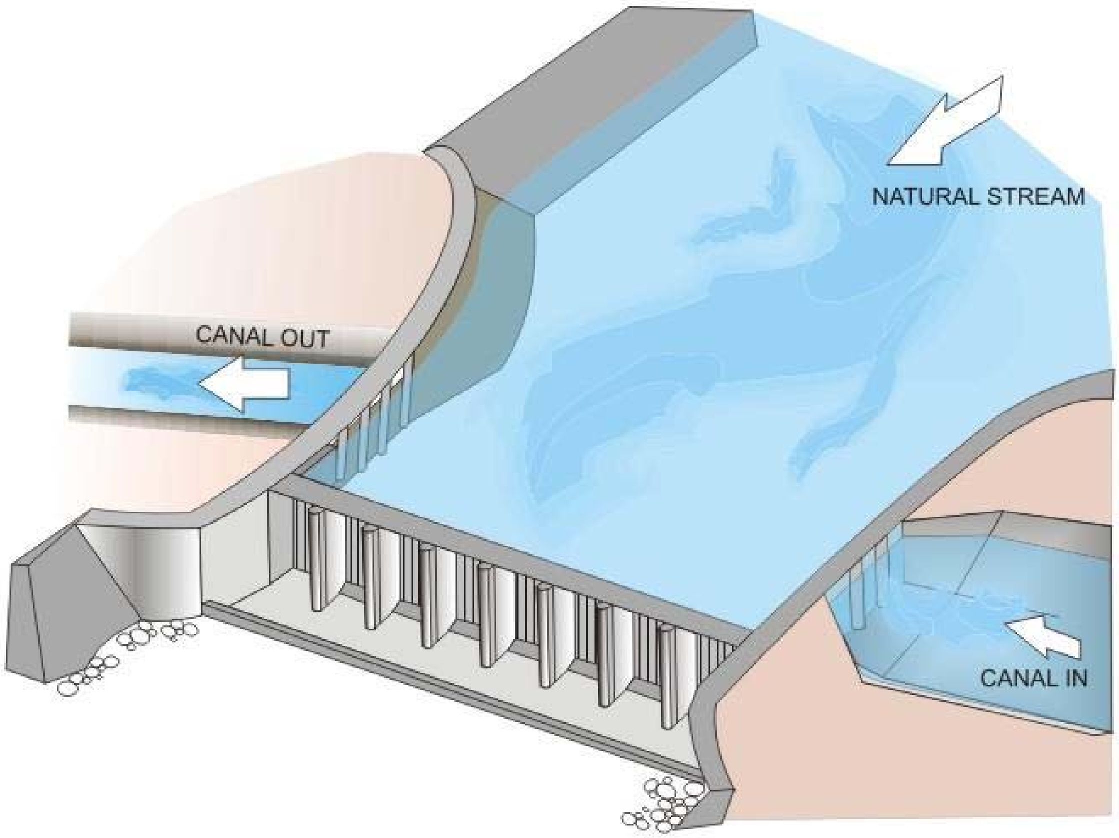


FIGURE 18. General view level crossing (gate hoisting arrangements not shown)

This type of structure is provided when a canal approaches a large sized drainage with high flood discharges at almost the same level. The flow control is usually provided on either side of the canal and on the outlet side of the drain. As such, this type of arrangement is very similar to canal head-works with a barrage. Advantage may be taken of the flow of the natural drainage to augment the flow of the outgoing canal. The barrage type regulator is kept closed during low flows to head up the water and allows the lean season drainage flow to enter the outgoing canal. During flood seasons, the barrage gates may be opened to allow much of the silt-laden drainage discharge to flow down.

Another structure, called an ***inlet***, is sometimes provided which allows the entry of the stream water into the canal through an opening in the canal bank, suitably protected by pitching the bed and sides for a certain distance upstream and downstream of the inlet. If the natural stream water is not utilized in the canal then an ***outlet,*** which is an opening on the opposite bank of the canal is provided. The canal bed and sides suitably pitched for protection.

3.8.6 Transitions at changes in canal cross-sections

A canal cross section may change gradually, in which case suitable flaring of the walls may be made to match the two sections (Figure 19).

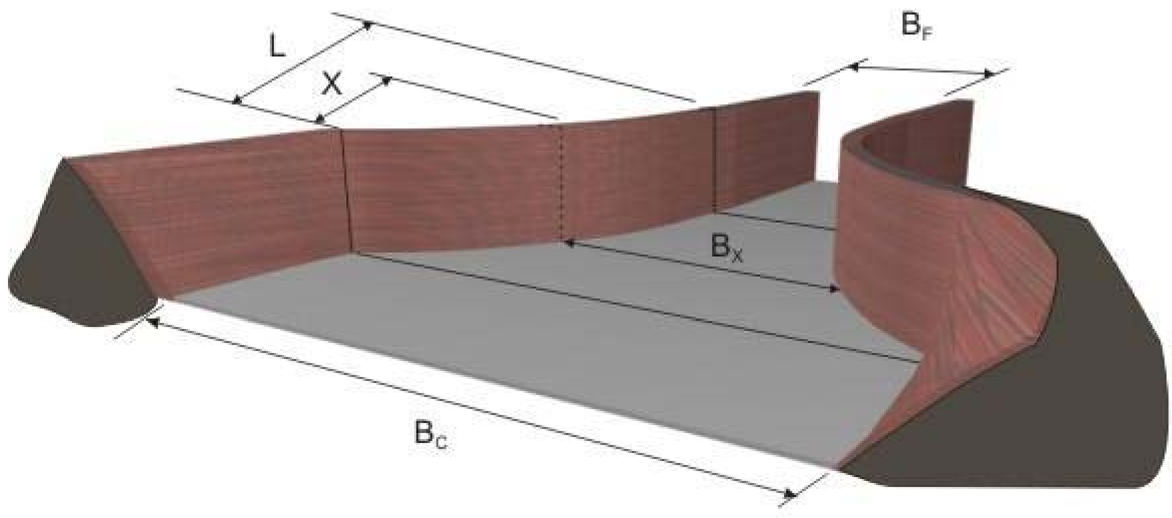


FIGURE 19. Transition canal banks warped to vertical and then flumed

For more abrupt changes, like a normal canal section being changed to a vertical walled aqueduct, suitable transitions have been designed which would avoid formation of any hydraulic with consequent loss of energy. A typical view of transition of a normal canal bank to a vertical walled flume section is shown in (Figure 20).

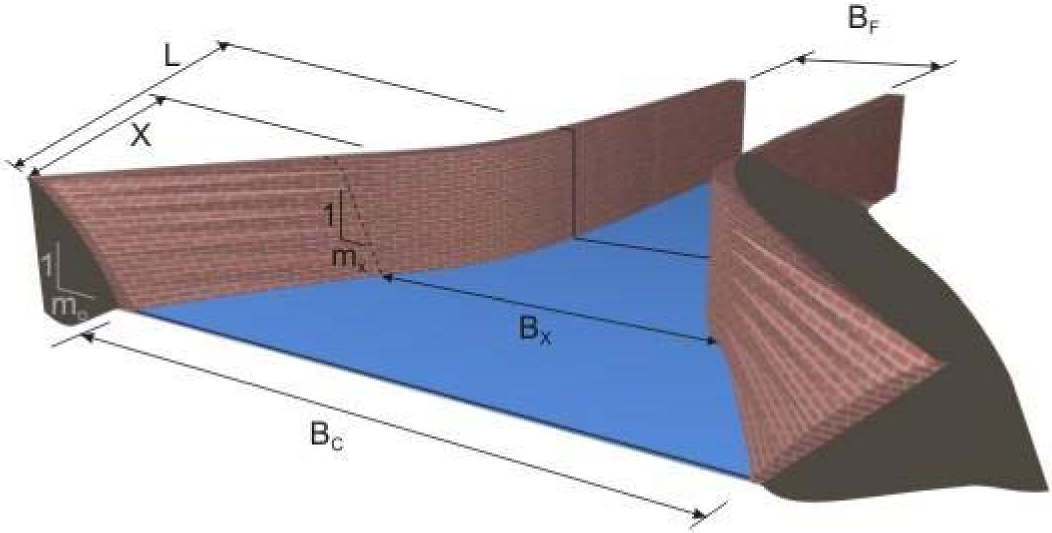


FIGURE 20. Transition with canal banks warped to vertical along with fluming

As may be observed, the banks of the normal canal section are first changed to vertical walls keeping the same canal bed width (***B_c***). Beyond this, the vertical section is reduced gradually to form a reduced sized flume of width (***B_f***). Various formulae have been proposed for deciding the intermediate curve, that is, an equation deciding the width (***B_x***) at any distance ***x*** from the start of the fluming, assuming a length L for the transition. One formula that is commonly used for this kind of transition is the UPIRI method, commonly known as Mitra's transition and is given as follows:

$$B_x = (B_c * B_f * L) / (L * B_c - X (B_c - B_f)) \qquad (1)$$

The length ***L*** of the transition is assumed to be equal to 2 *(B_c – B_f). In another type of transition, the vertical curved walls of a normal canal section is both transformed in to vertical walls of a flume as well as its section is reduced gradually, as shown in Figure 20. This results in reduction of the canal bed width from ***B_c*** to ***B_f*** and the side slopes from M_0 to O. The values for the bed width B_x at any length X from the start of the transition and the corresponding side slope ***m_x*** are given by the following expressions

$$B = B_c + X/L\,[1- (1- X/L)^n] \quad (B_c - B_f) \qquad (2)$$

$$m_x = m_0\,[1 - (1- X/L)^{1/2}] \qquad (3)$$

Where $n = 0.8 - 0.26(m_0)^{1/2}$ and the length of transition L, is expressed as

$$L = 2.35\,(B_c - B_f) + 1.65\, m_0\, h_c \qquad (4)$$

3.8.7 Planning and design of canal conveyance structures

Though a number of books are available for detailed design of conveyance structures of a canal, as mentioned in the reference list, one may consult the following Bureau of Indian Standard codes for planning, initial designs and construction of these structures.

- IS: 7784 "Code of practice for design of cross drainage works"

  (1) Part 1 -1975 General features (Reaffirmed 1987)

  (2) Part 2-1983 Specific requirements (Reaffirmed 1992) Section1. Aqueducts

  (3) Part 2 - 1980 Specific requirements (Reaffirmed 1992) Section2. Super-passage

  (4) Part 2 -1981 Specific requirements (Reaffirmed 1992) Section3. Canal syphons

  (5) Part 2 - 1980 Specific requirements (Reaffirmed 1992) Section4. Level crossings

  (6) Part 2 -1980 Specific requirements (Reaffirmed 1992) Section5. Syphon aqueducts

- IS: 11385-1985 "Code of practice for subsurface exploration for canals and cross drainage works" (Reaffirmed 1990)

- IS: 9913-1981 "Code of practice for construction of cross drainage works" (Reaffirmed 1992)

References

Asawa, G L (1996) "Irrigation engineering", Second edition, New Age Publications

- Garg, S K (1996) "Irrigation engineering and hydraulic structures", Twelfth Edition, Khanna Publishers

- IS: 11385-1985 “Code of practice for subsurface exploration for canals and cross drainage works” (Reaffirmed 1990)
- IS: 7784 “Code of practice for design of cross drainage works”, Parts 1 to 6
- IS: 9913-1981 “Code of practice for construction of cross drainage works” (Reaffirmed 1992)
- Varshney, R S, Gupta, S C and Gupta, R L (1993) “Theory and design of irrigation structures”, Volume II, Sixth Edition, Nem Chand Publication

Module 3

Irrigation Engineering Principles

Lesson 9

Regulating Structures for Canal Flows

Instructional objectives

On completion of this lesson, the student shall be able to learn:

1. The necessity of providing regulating structures in canals.
2. The basics of canal drops and falls.
3. The importance of canal regulators.
4. The need for Groyne Walls, Curved Wings and Skimming Platforms.
5. The functions of escapes in a canal.

3.9.0 Introduction

A canal obtains its share of water from the pool behind a barrage through a structure called the ***canal head regulator***. Though this is also a regulation structure for controlling the amount of water passing into the canal (with the help of adjustable gates), it shall be discussed under diversion works (Module 4). In this lesson, attention is focussed on structures that regulate the discharge and maintain the water levels within a canal network (Figure 1).

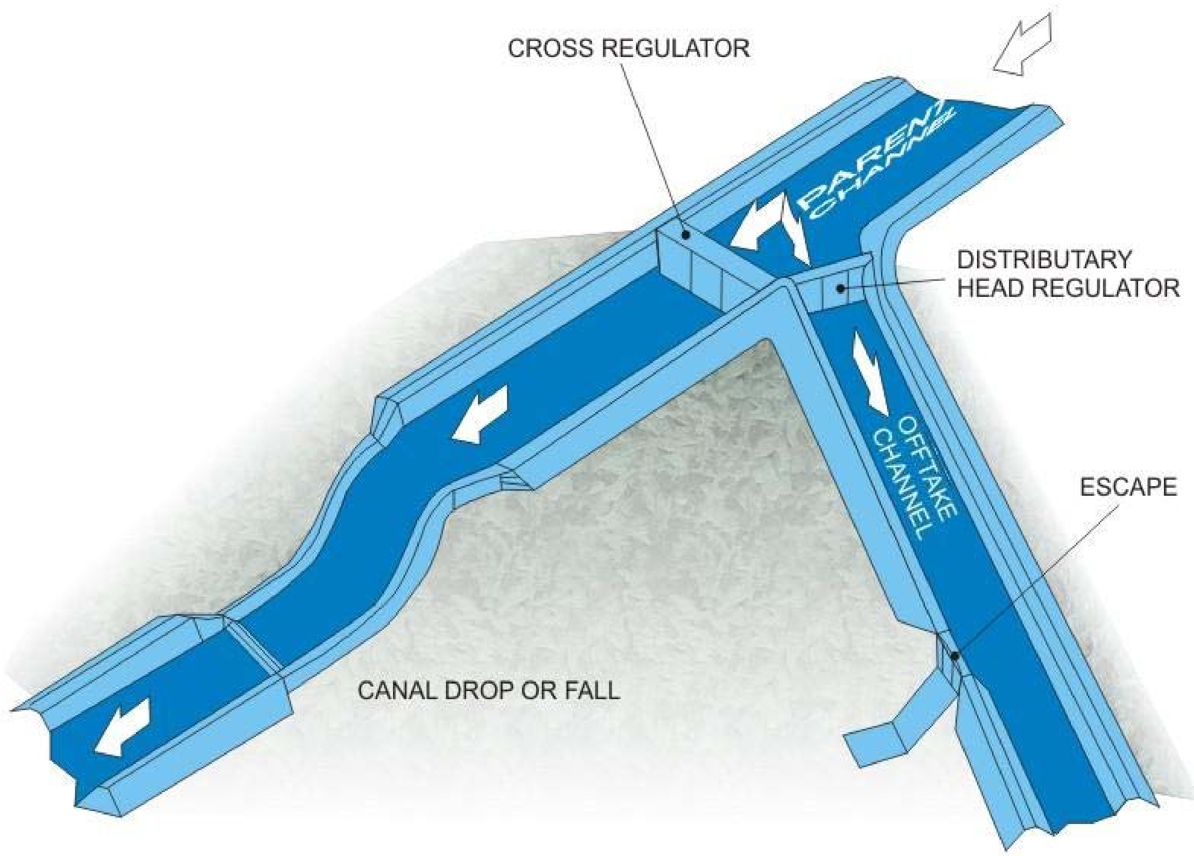


FIGURE 1. Canal structures for flow regulation and control

These structures may be described as follows:

1. ***Drops*** and ***falls*** to lower the water level of the canal
2. ***Cross regulators*** to head up water in the parent channel to divert some of it through an off take channel, like a ***distributary***.
3. ***Distributary head regulator*** to control the amount of water flowing in to off take channel.
4. ***Escape***s, to allow release of excess water from the canal system.

These structures are described in detail in this lesson.

3.9.1 Canal drops and falls

A canal has a designed longitudinal slope but has to pass through an undulating terrain. When a canal crosses an area that has a larger natural surface slope, a ***canal drop***, also called ***fall*** in India, has to be provided suitably at certain intervals (Figure 2)

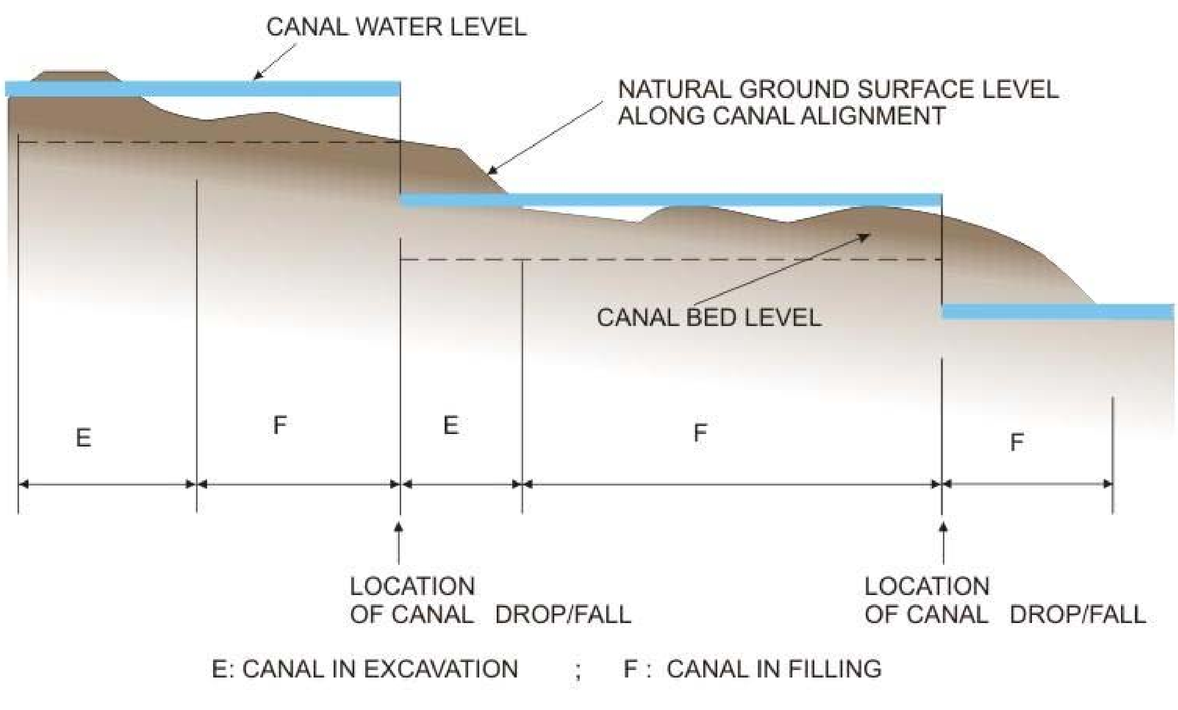


FIGURE 2 . Typical location for providing canal drop or fall

The location of a fall has to be judiciously worked out such that there should be a balance between the quantities of excavation and filling. Further the height of the fall has to be decided, since it is possible to provide larger falls at longer intervals or smaller falls at shorter intervals. It may be observed that the portion of the canal which is running in filling (Figure2) may be able to serve the surrounding area by releasing water by gravity. For the portion of the canal that is running in excavation, if surrounding areas have to be irrigated, it has to be done through pumping.

There are various types of fall structures, some of which are no more provided these days. However, there are many irrigation projects in India which have these structures in the canal network, as they were designed many years ago. Many of these structures used boulder masonry as their construction material, whereas now brick masonry or, more commonly, mass concrete is being used commonly in modern irrigation projects.

3.9.2 Falls of antiquity

The Ogee type of fall has been one of the first to be tried in the Indian canal irrigation system, probably since more than a century back (Figure 3a). However, according to the earliest structures provided, the crest of the fall was in the same elevation as that of the upstream section of the canal. This caused a sharp draw-down of the water surface on the upstream side. On the downstream, the drop in elevation added energy to the falling water which exited the falls as a shooting flow, causing erosion of the canal bed immediately downstream. These difficulties were later removed by raising the crest level of the fall above the upstream canal bed level and providing suitable stilling basin with end sill at the downstream end of the fall which kills most of the excess energy of the leaving water by helping to form a hydraulic jump (Figure 3b).

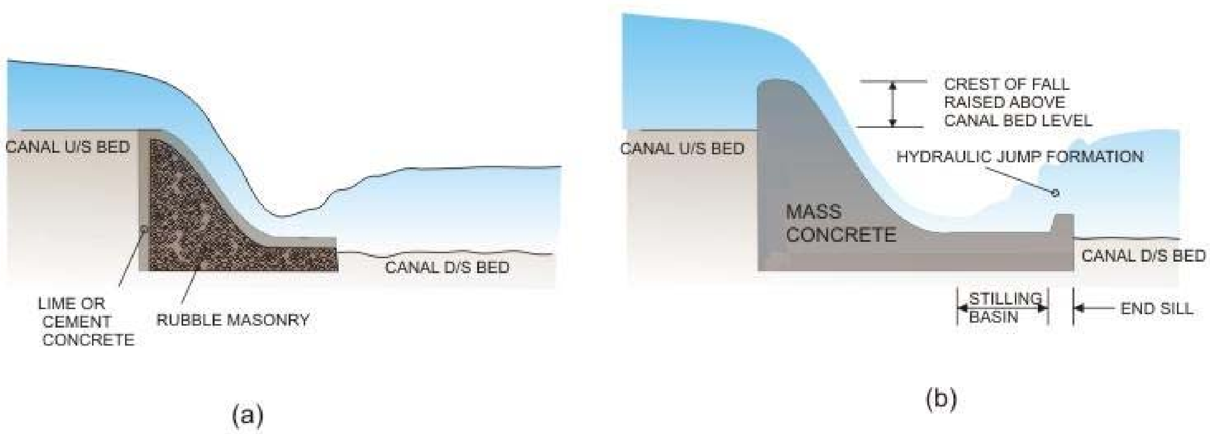


FIGURE 3. (a) Ogee-type fall made of rubble masonry
(b) Same type of fall, but made of concrete and equipped with a stilling basin for energy dissipation

The ***rapid-fall*** was tried in some of the north-Indian canals which were constructed with boulders cemented together by lime concrete (Figure 4). These were quite effective but, the cost being prohibitive, was gradually phased out.

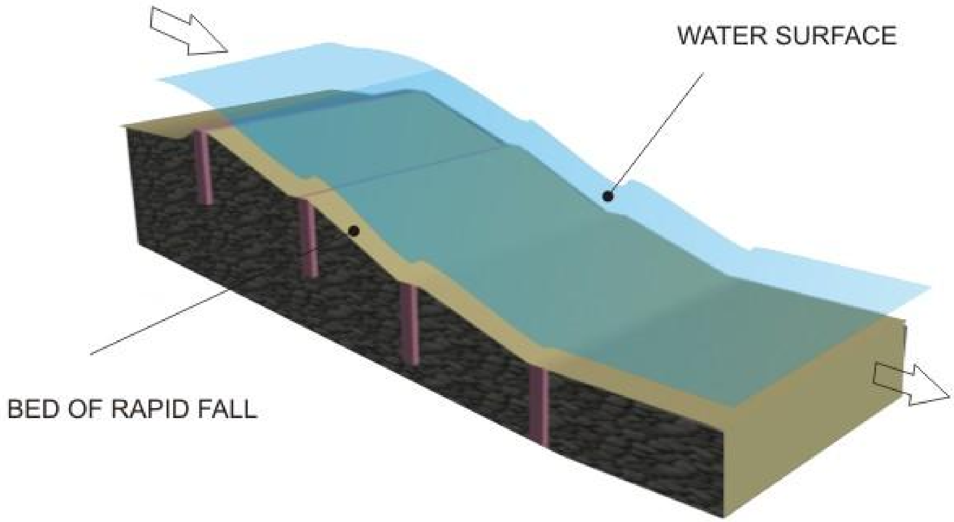


FIGURE 4. Rapid falls

The ***trapezoidal-notch fall*** consists of one or more notches in a high crested wall across the channel with a smooth entrance and a flat circular lip projecting downstream from each notch to disperse water (Figure 5). This type of fall was started around the late nineteenth century and continued to be constructed due to its property of being able to maintain a constant depth-discharge relationship, until simpler and economical alternatives were designed.

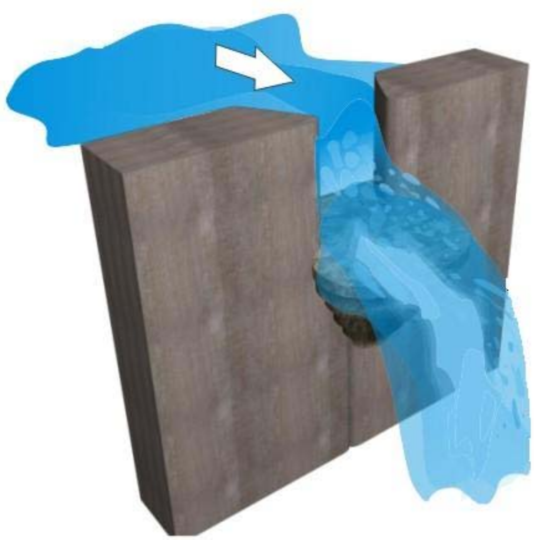


FIGURE 5. The notch-fall

3.9.3 Modern falls

Some falls have been commonly used in the recent times in the canal systems of India. These are described in the following sections. Detailed references may be had from the following two publications of the Food and Agriculture Organisation (FAO):

1. FAO Irrigation and Drainage paper 26/1: Small Hydraulic Structures, Volume 1 (1982)
2. FAO Irrigation and Drainage paper 26/2: Small Hydraulic Structures, Volume 2 (1982)

These books are also available from the web-site of FAO under the title "Irrigation and Drainage Papers" at http://www.fao.org/ag/agL/public.stm#aglwbu.

3.9.3.1 Falls with vertical drop

These are falls with impact type energy dissipators. The ***vertical-drop fall*** (Figure 6) uses a raised crest to head up water on the upstream of the canal section and allows it to fall with an impact in a pool of water on a depressed floor which acts like a cushion to dissipate the excess energy of the fall. This type of fall was tried in the Sarda canal of Uttar Pradesh, which came to be commonly called as the ***Sarda-type fall***.

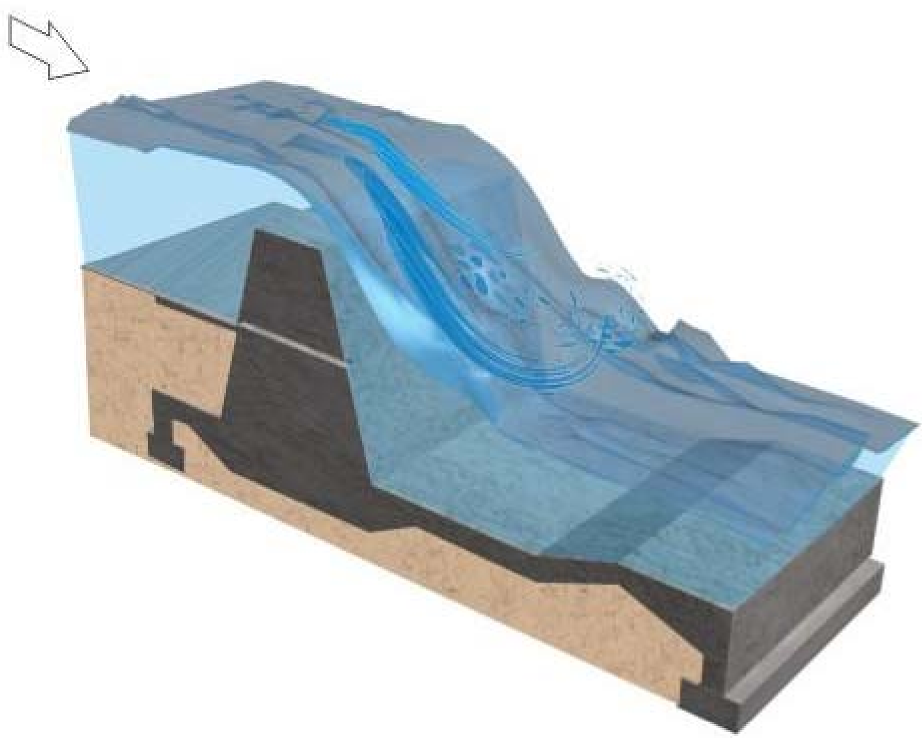


FIGURE 6 . Vertical drop fall

Typical plan and section of a Sarda-type fall is shown in Figure 7. Usually, two different crests for the fall are adopted, as shown in Figure 8. For canals conveying discharges less that $14 m^3/s$, crest with rectangular cross section is adopted, and for discharges more than that, trapezoidal crest with sloping upstream and downstream faces is chosen.

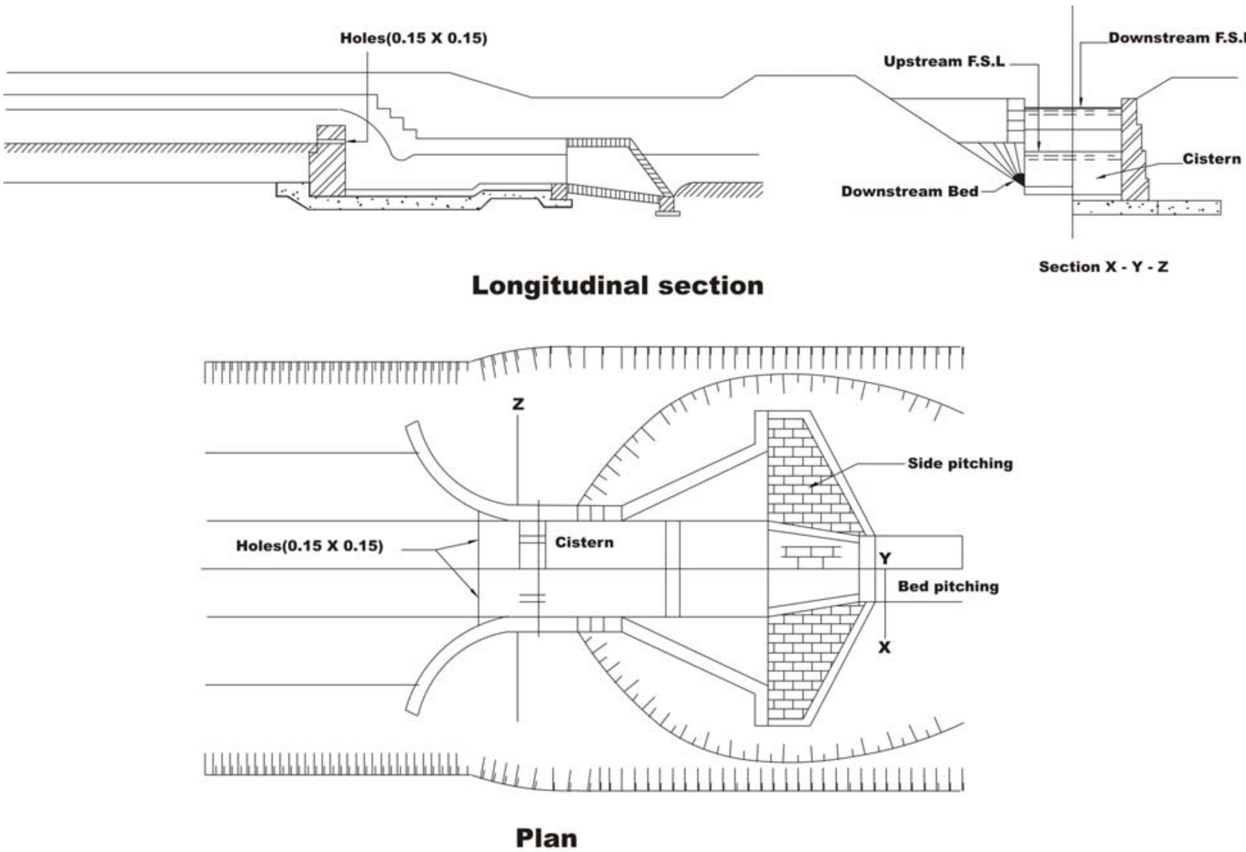


FIGURE 7. SARDA TYPE FALL

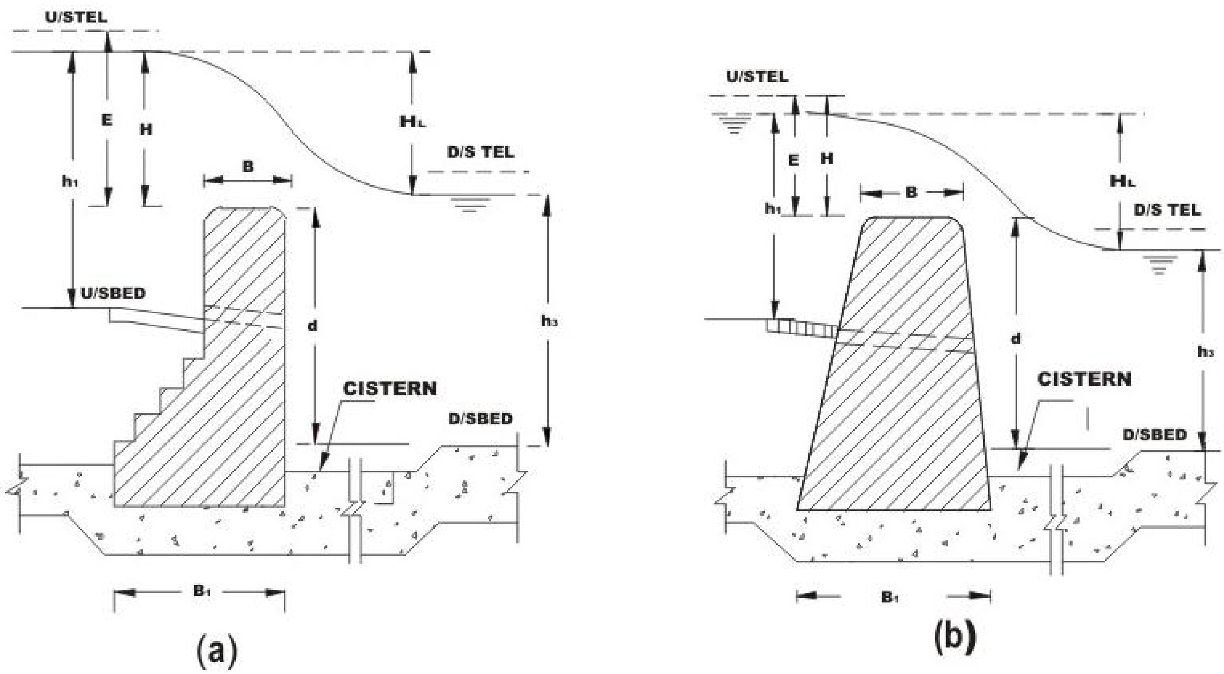
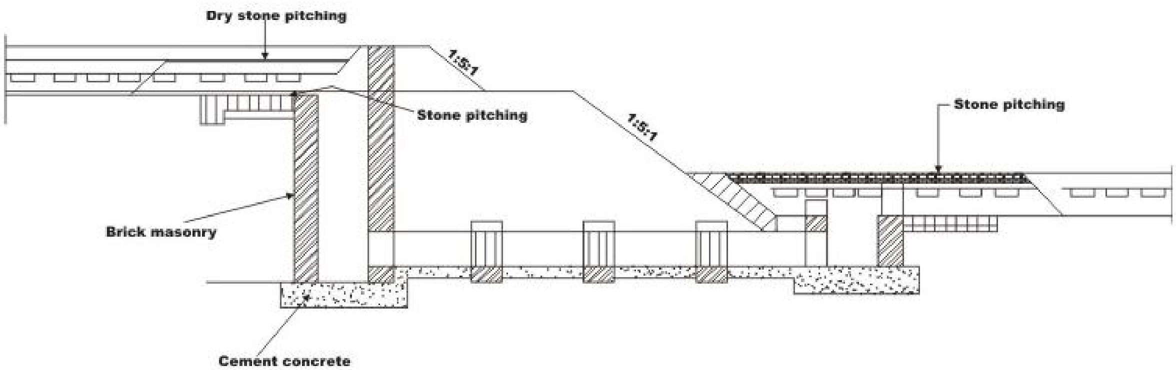


FIGURE 8 Types of cross-section for Sarada Fall

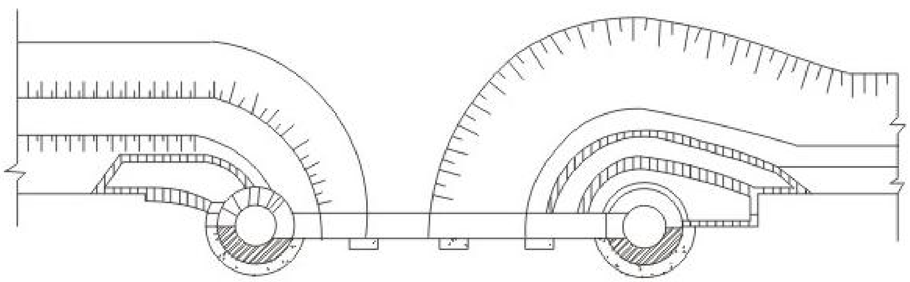
(a) Rectangular Crest (Drowned Flow)
(b) Trapezoidal Crest (Free flow)

For smaller discharges, the following a may be provided.

- ***Well drop fall*** (Figure 9)
- ***Pipe drop fall*** (Figure 10)
- ***Baffled apron drop*** (Figure 11)



Longitudinal section



Half plan of top and half plan of bottom

FIGURE 9 . WELL DROP

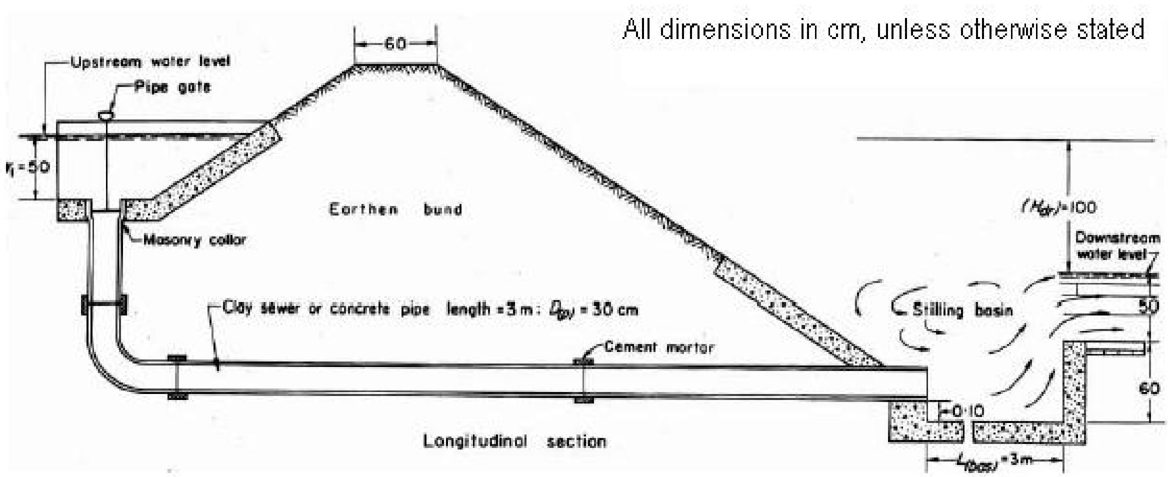


FIGURE 10. Pipe drop spillway

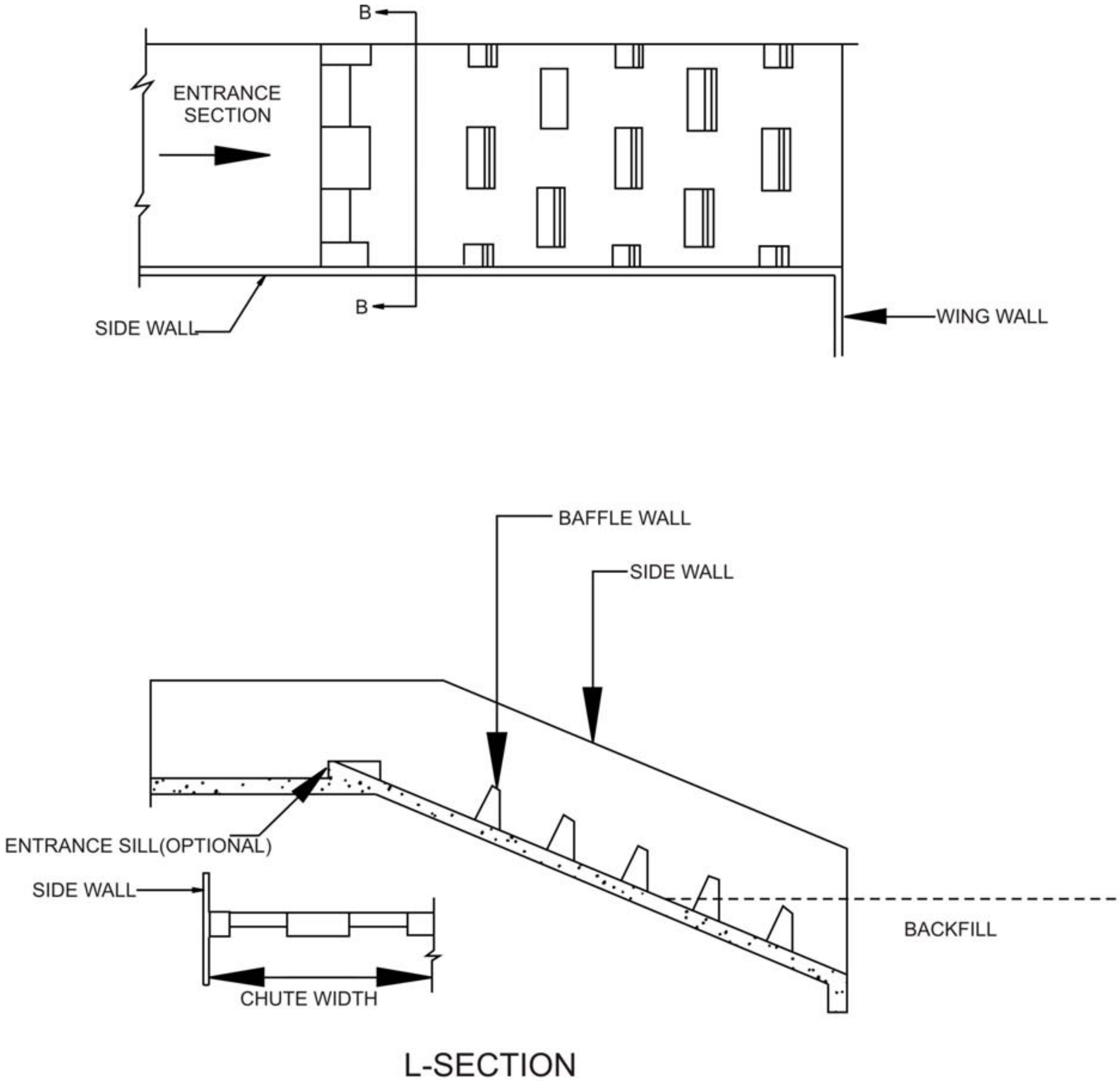


FIGURE.11 PLAN AND SECTION OF BAFFLED APRON DROP

3.9.3.2 Falls with drop along inclined glacis

These are falls with and inclined glacis along which the water glides down and the energy is dissipated by the action of a hydraulic jump at the toe of the structure. Inclined drops are often designed to function as flume measuring devices. These may be with and without baffles as shown in Figures 12 and 13 respectively and supplemented by friction blocks and other energy dissipating devices (Figure 14).

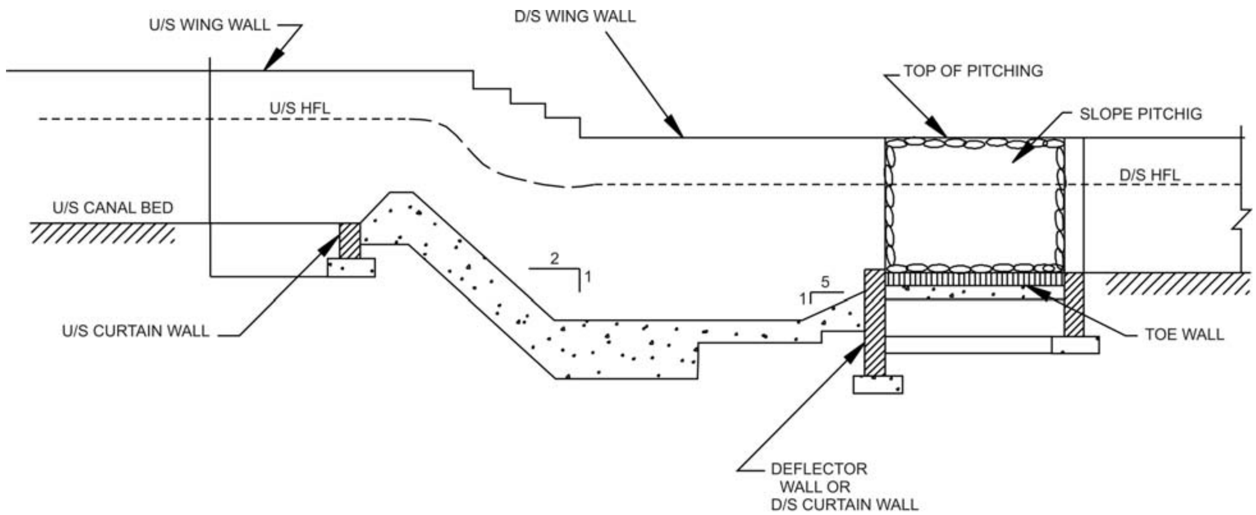


FIGURE 12. SECTIONAL VIEW THROUGH GLACIS FALLS WITHOUT BAFFLES

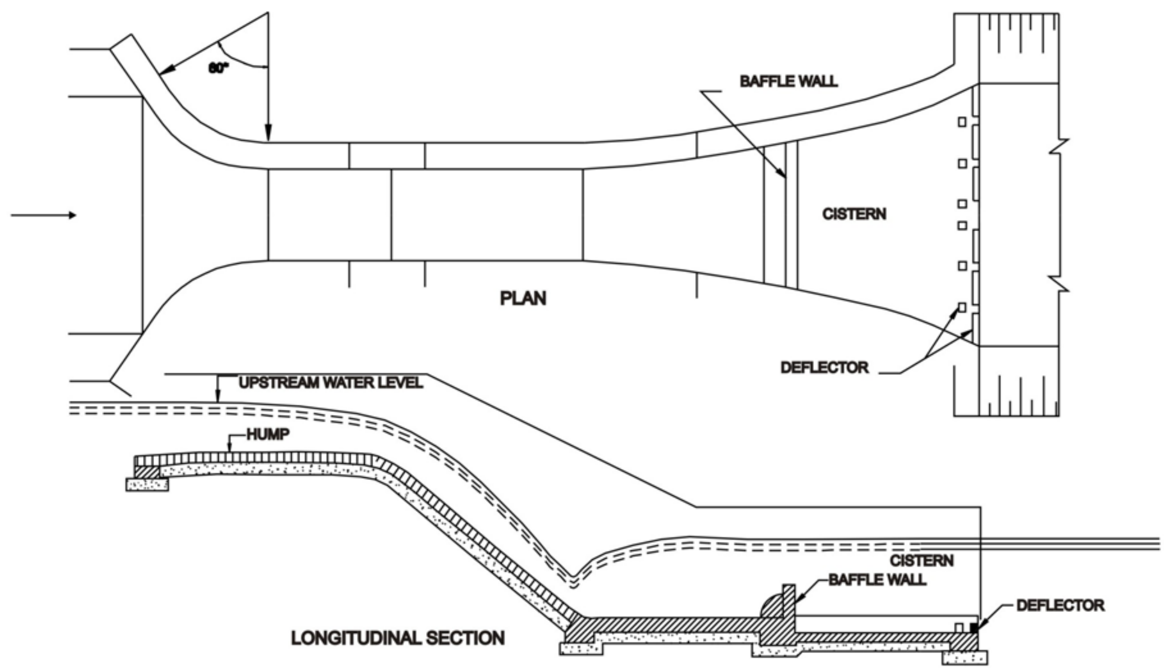


FIGURE 13. PLAN AND SECTION OH STANDING WAVE FLUME- FALL

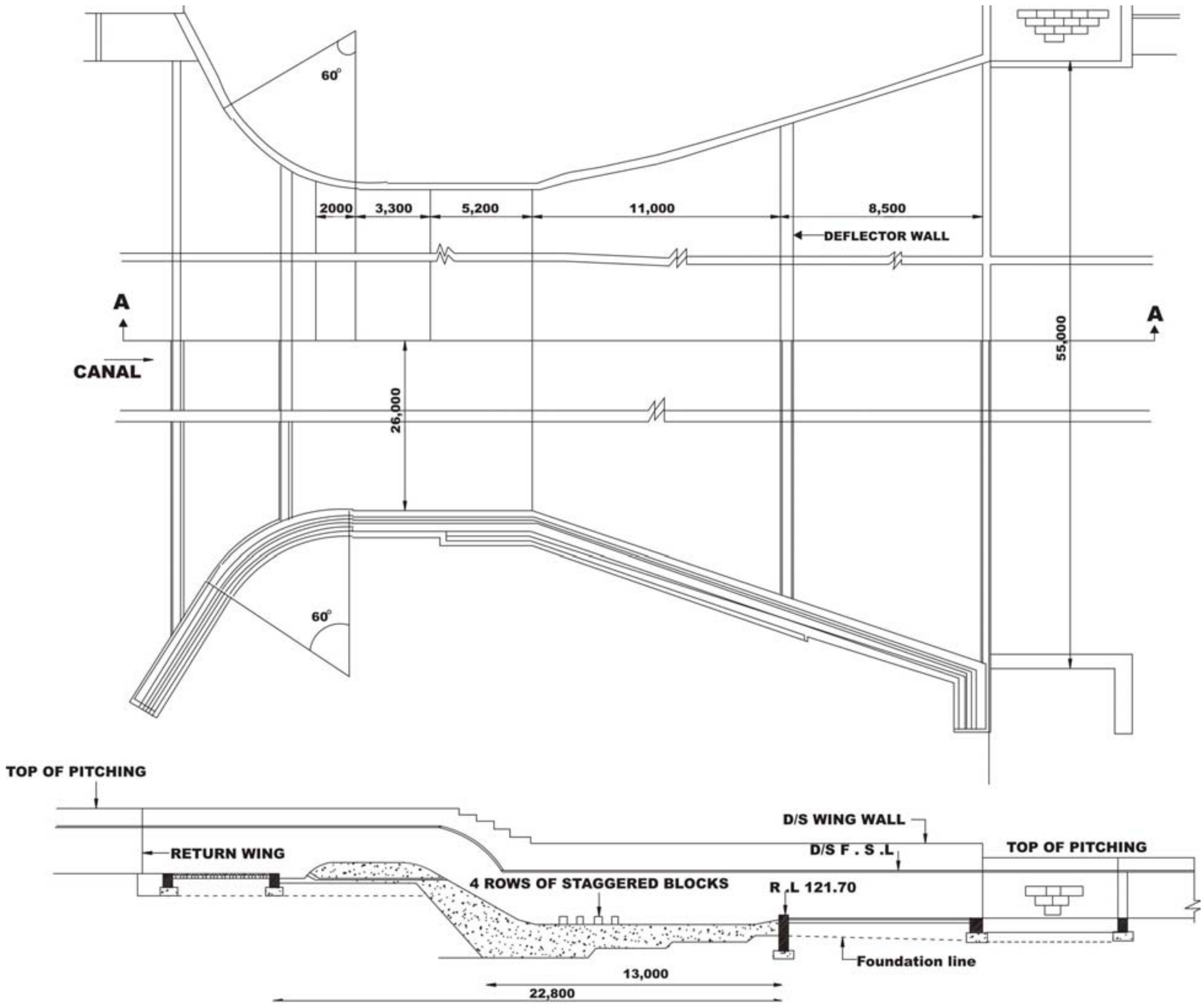
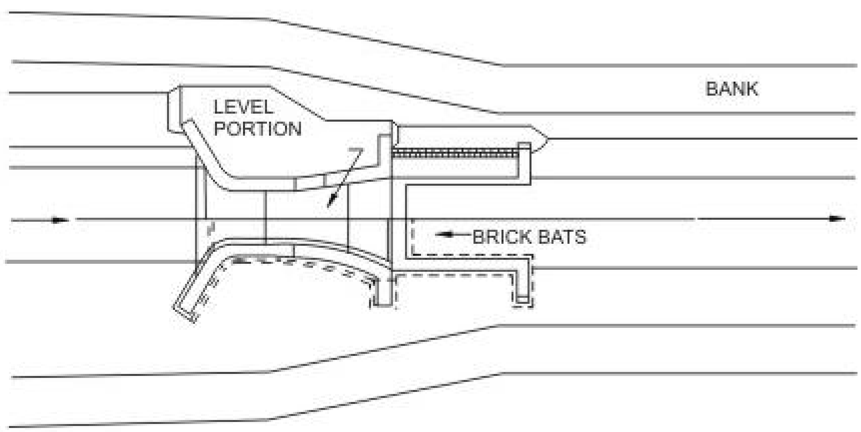
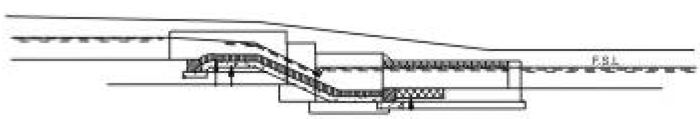


FIGURE 14 Flumed glacis with metered fall

Similar type of fall was also developed in Punjab which was called the ***CDO type fall***, as shown in Figure 15 (for hydraulic drop up to 1m) and Figure 16 (for hydraulic drop above 1m).

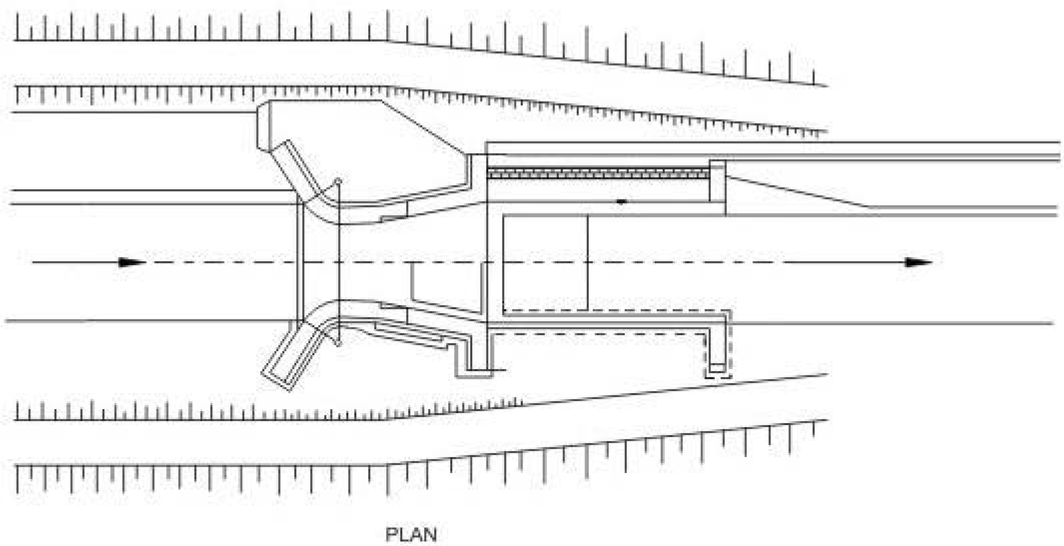


PLAN



LONGITUDINAL SECTION

FIGURE.15. CDO PUNJAB TYPE FALL UPTO 1m DROP



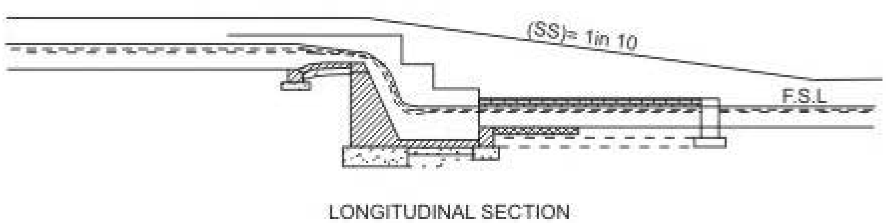


FIGURE.16. CDO PUNJAB TYPE FALL FOR GREATER THAN 1m DROP

The glacis type falls may be modified in the following ways:

(a) ***Flumed*** or ***un-flumed***, depending upon the crest width being smaller or equal to the bed width of the canal (Figure17).

(b) ***Meter*** or ***non-meter*** fall depending upon whether the canal fall may be used to measure the discharge as well. Details of a meter-fall is described in Lesson 3.10

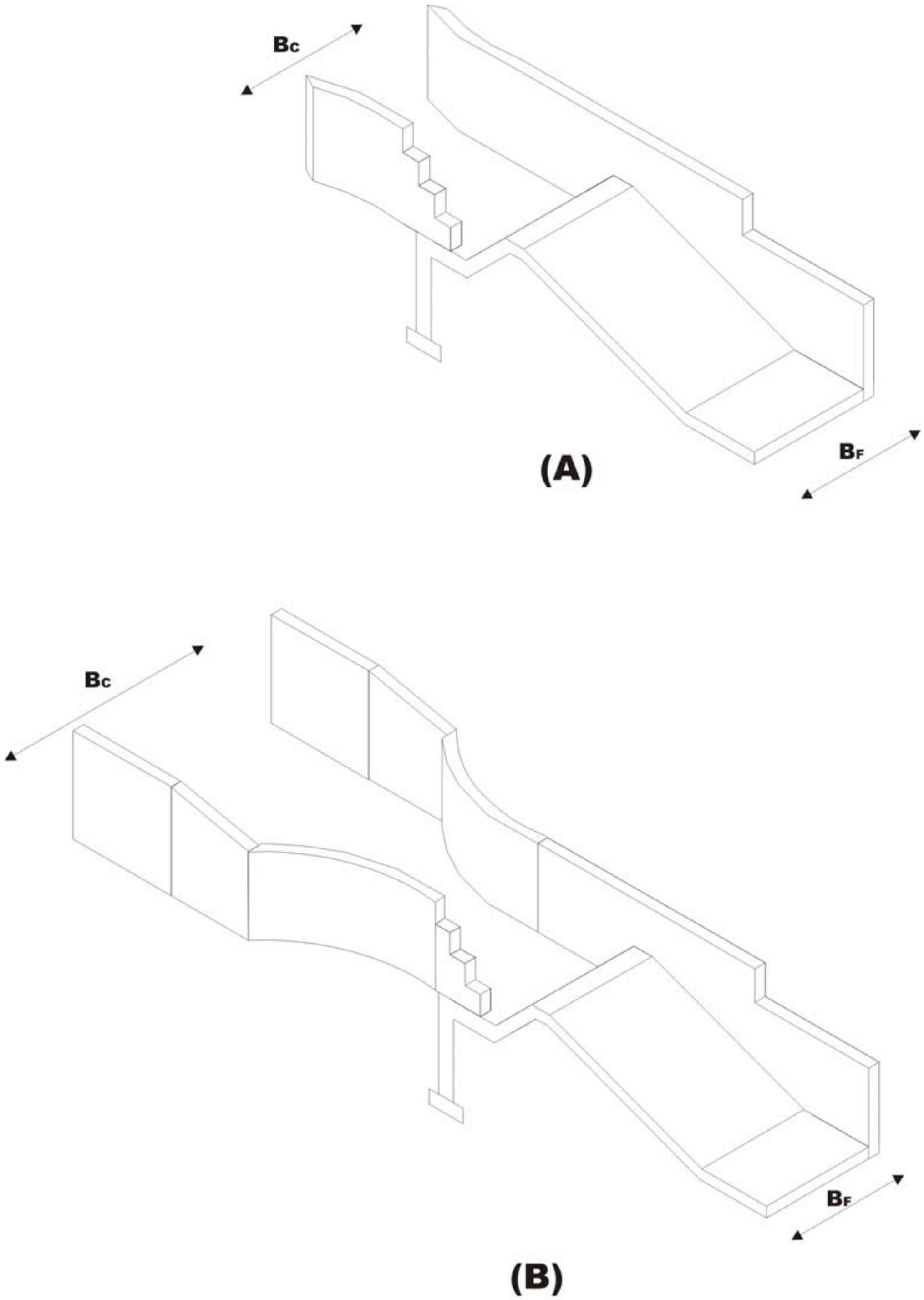


Figure 17 (A) A flumed glacis fall with fall width(B_F) being less than canal width(B_c).
(B) An un - flumed glacis fall, where (B_F) is the same as (B_c).

The following appurtenant structures should be considered while providing a vertical-drop or a glacis-type fall:

- The floor of the falls should be able to resist the uplift pressure under the condition of dry canal and a high ground water table.
- ***Cut-off walls*** or ***curtain walls*** either of masonry or concrete should be provided at the upstream and downstream ends of the floors of the falls.
- Bed protection with dry brick pitching should be provided in the canal just upstream and downstream of the fall.
- Side protection should be provided at the upstream and downstream splays with brick pitching.

Since falls are structures across a canal, it is usual for providing a bridge along with the fall structure for crossing the canal.

3.9.4 Canal regulators

These include the ***cross regulator*** and the ***distributary head regulator*** structures for controlling the flow through a parent canal and its off-taking distributary as shown in Figure 1. They also help to maintain the water level in the canal on the upstream of the regulator. Canal regulators, which are gated structures, may be combined with bridges and falls for economic and other considerations, like topography, etc.

A typical view of a distributary head regulator and a cross regulator (shown partly in section) is illustrated in Figure 18.

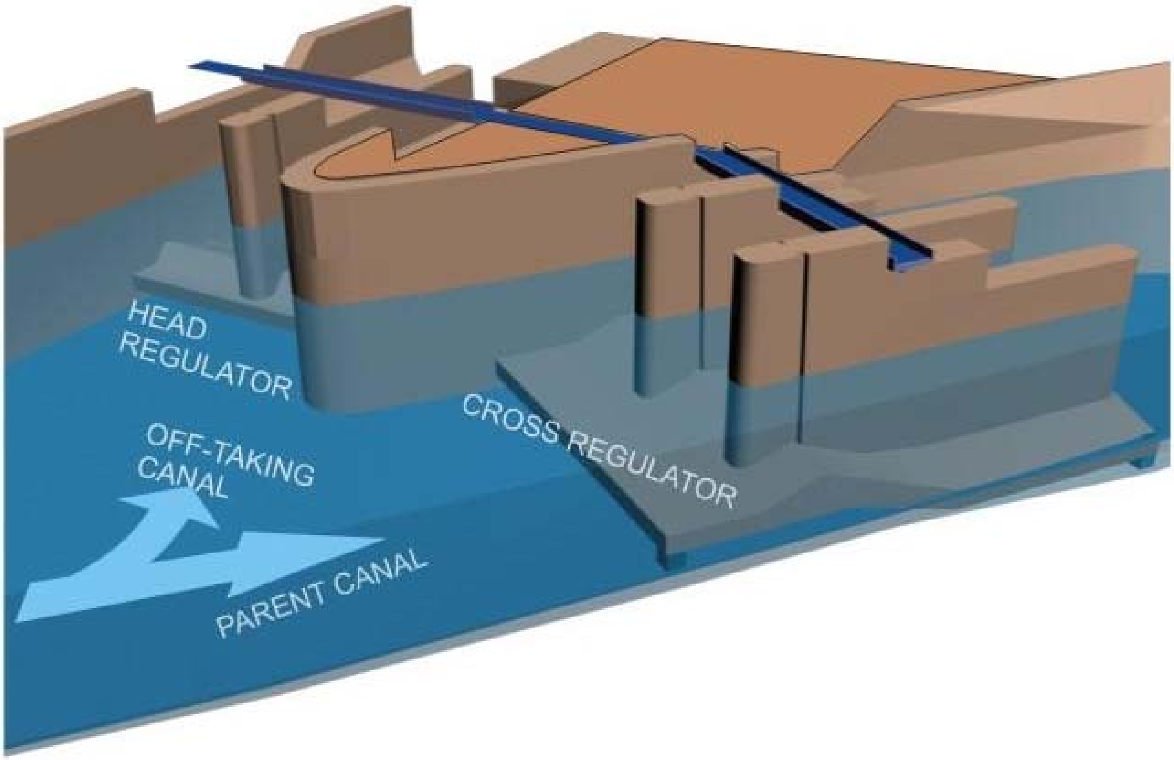


Figure 18 . Distributary head regulator and parent canal cross regulator showing combination with glacis fall and bridge
Gates and gate hoisting arrangements have not been shown for clarity

In the figure, the gates and gate hoisting arrangements have not been shown, for clarity. Further, the floor of the regulators would be protected on the upstream and downstream with concrete blocks and boulder apron. A typical sectional drawing through a regulator is shown in Figure 19.

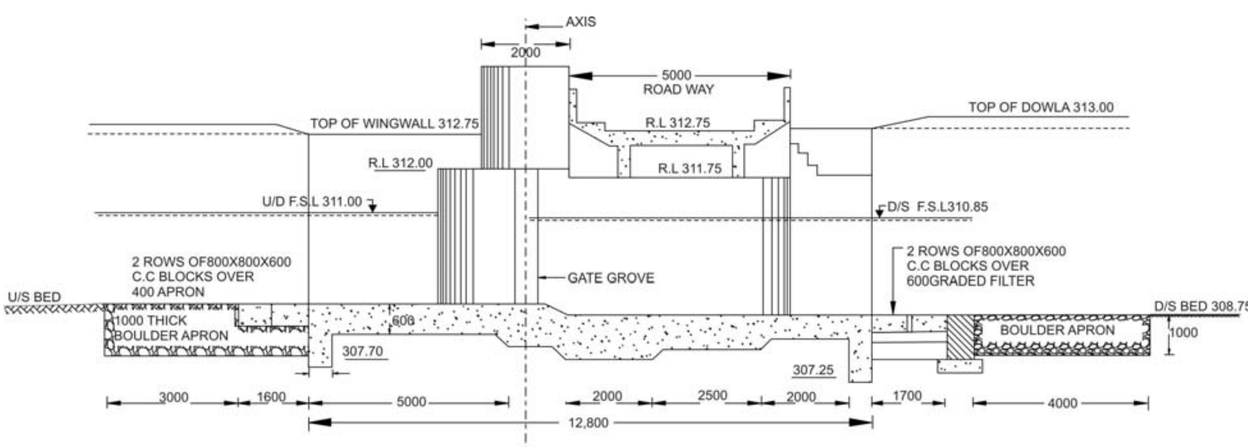


FIGURE 19. Section through a typical regulator

The angle at which a distributary canal off-takes from the parent canal has to be decided carefully. The best angle is when the distributary takes off smoothly, as shown in Figure 20(a). Another alternative is to provide both channels (off-taking and parent) at an angle to the original direction of the parent canal (Figure 20b). When it becomes necessary for the parent canal to follow a straight alignment, the edge of the canal rather than the centre line should be considered in deciding the angle of off-take (Figure 20c).

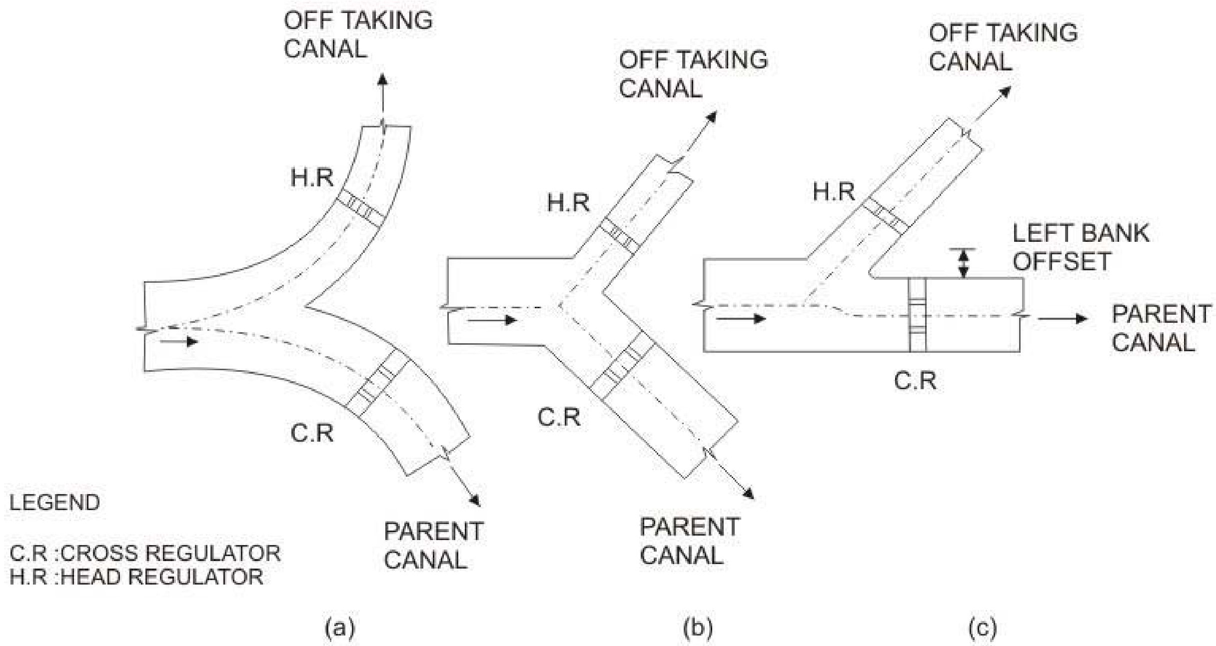


FIGURE 20. Alignment types for off taking canal from a parent canal
(a) Smooth off take
(b) Both inclined to original flow;
(c) Parent canal flows straight with reduced width.

To prevent excessive entry of silt deposition at the mouth of the off-take, the entry angle should be kept to between 60^0 and 80^0. For the hydraulic designs of cross regulators, one may refer to the Bureau of Indian Standard code IS: 7114-1973 “Criteria for hydraulic design of cross regulators for canals”. The water entering in to the off-taking distributary canal from the parent canal may also draw suspended sediment load.

The distributary should preferably be designed to draw sediment proportional to its flow, for maintaining non-siltation of either the parent canal or itself. For achieving this, three types of structures have been suggested as discussed below along with the relevant Bureau of Indian standard codes.

3.9.5 Silt vanes

(Please refer to IS: 6522-1972 "Criteria for design of silt vanes for sediment control in off-taking canals" for more details)

Silt vanes, or ***King's vanes***, are thin, vertical, curved parallel walled structures constructed of plain or reinforced concrete on the floor of the parent canal, just upstream of the off-taking canal. The height of the vanes may be about one-fourth to one-third of the depth of flow in the parent canal. The thickness of the vanes should be as small as possible and the spacing of the vanes may be kept about 1.5 times the vane height. To minimize silting tendency, the pitched floor on which the vanes are built should be about 0.15 m above the normal bed of the parent channel. A general three dimensional view of the vanes is shown in Figure 21 and a typical plan and sectional view in Figure 22.

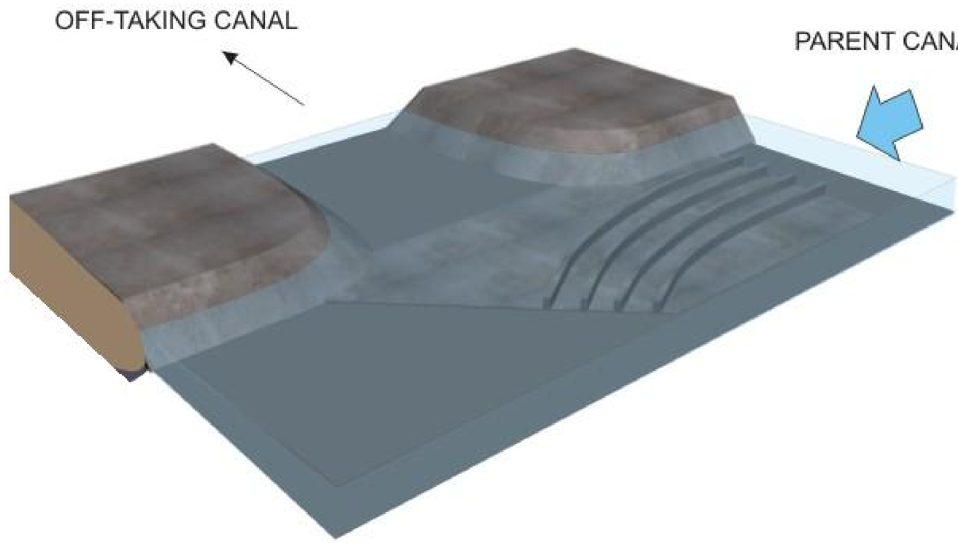


FIGURE 21. View of silt vanes for diverting sediment bed load of parent canal away from offtake

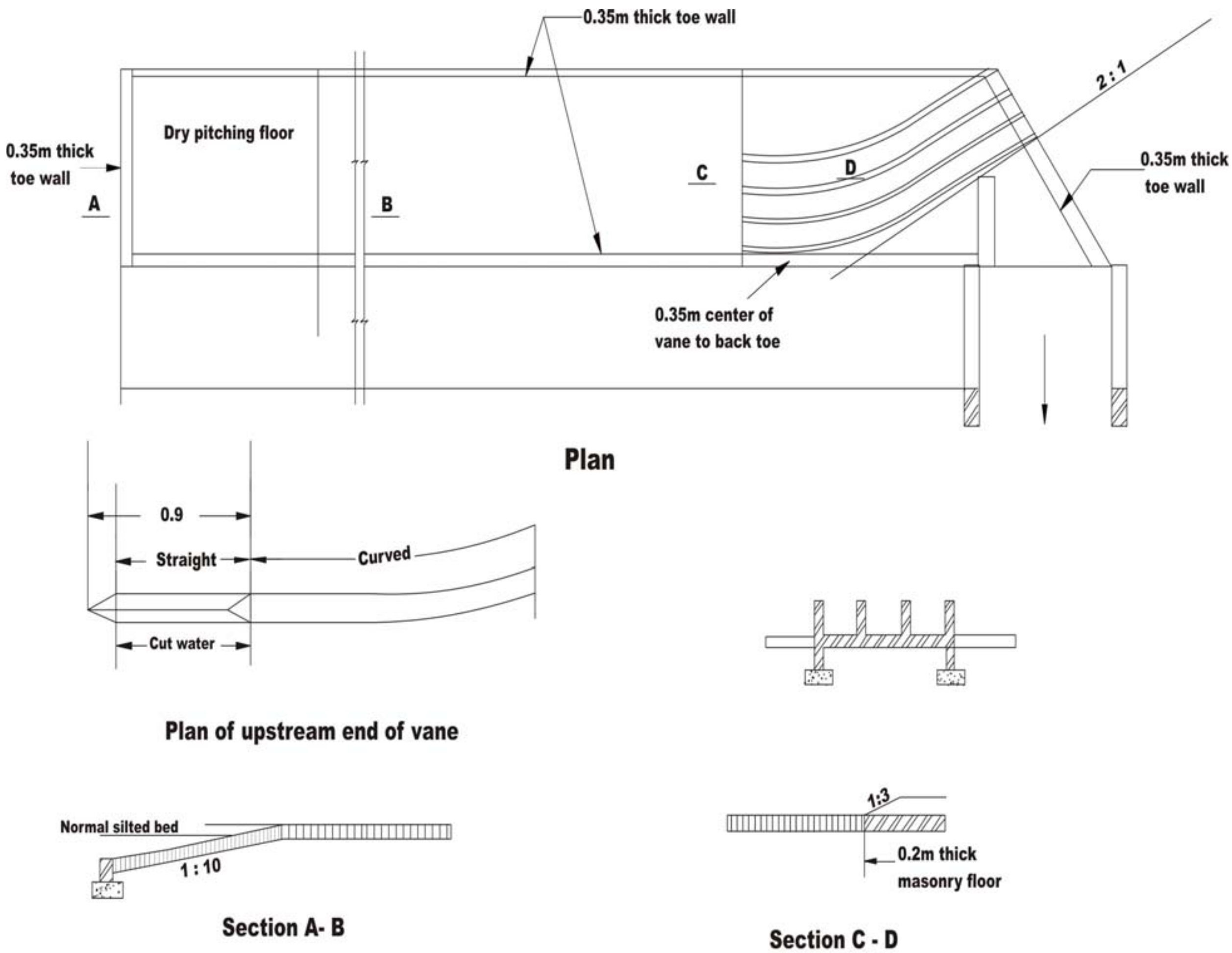


FIGURE 22. SILT VANES

3.9.6 Groyne walls or curved wings

(Please refer to IS: 7871-1975 “Criteria for hydraulic design of groyne wall (curved wing) for sediment distribution at off-take points in a canal” for more details)

These are curved vertical walls, also called ***Gibb’s groyne walls***, which project out in to the parent canal from the downstream abutment of the off-taking canal. The groyne wall is provided in such a way that it divides the discharge of the parent canal in proportion of the discharge requirement of the off-taking canal with respect to the flow in the downstream parent canal. The groyne wall extends upstream in to the parent canal to cover ¾ to full width of the off-take. The proportional distribution of flow in to the off-taking canal is expected to divert proportional amount of sediment, too. A general view of a groyne wall is shown in Figure 23.

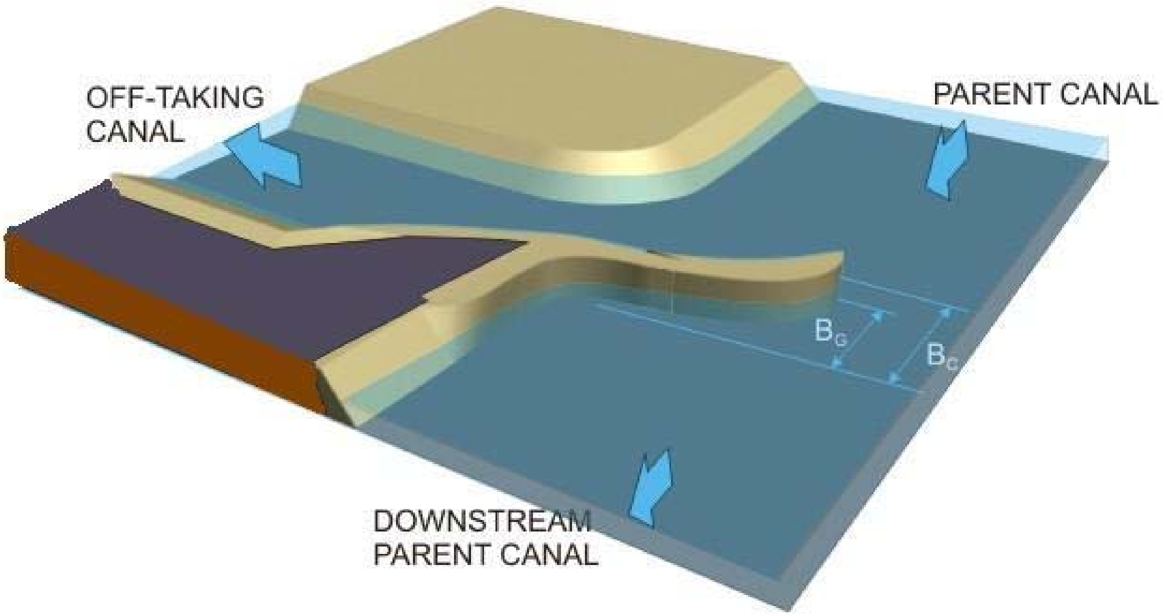


FIGURE 23. View of groyne wall (curved wing)

B_G(projected length of groyne wall) should vary from 0.75 to 1.00 B_C, where B_C is the bed width of the off-taking canal

The distance of the nose from the upstream abutment of the off-take may be kept so as to direct adequate discharge in to the off-take. The height of the groyne wall should be at least 0.3m above the full supply level of the parent canal. At times, a combination of groyne wall and sediment vane may be provided.

3.9.7 Skimming platforms

(Please refer to IS: 7880-1975 “Criteria for hydraulic design of skimming platform for sediment control in off-taking canal” for more details).

A skimming platform is an RCC slab resting on low height piers on the bed of the parent canal, and in front of the off-taking canal, and in front of the off-taking canal as shown in Figure 24.

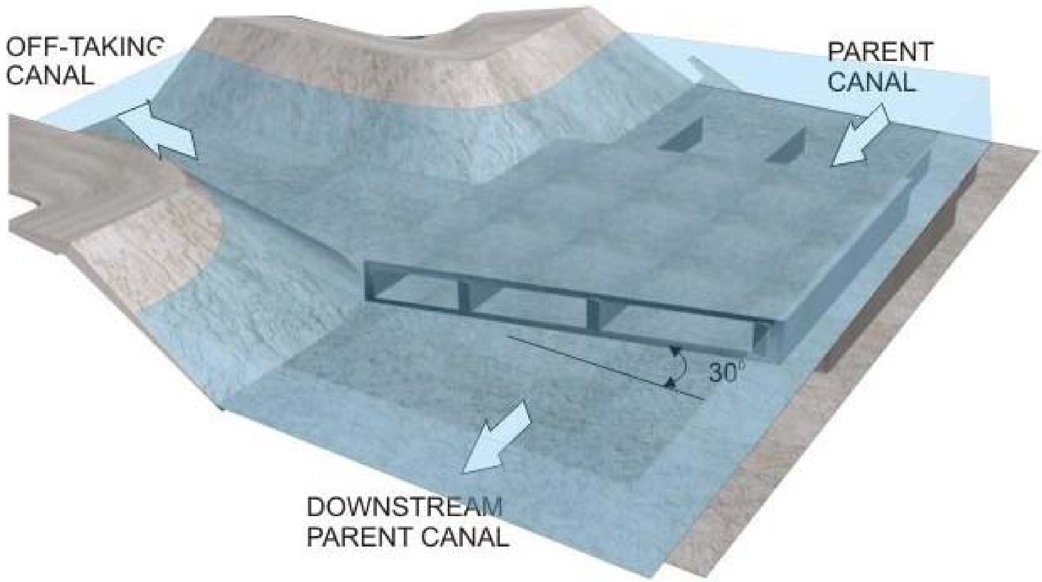


FIGURE 24. View of Skimming Platform

This arrangement actually creates a kind of low tunnel at the bed of the parent canal, which allows the sediment moving along its bed to pass through downstream. The floor of the off-taking canal being above the level of the platform thus only takes suspended sediment load coming along with the main flow in the parent canal. A skimming platform arrangement is suitable where the parent channel is deep (about 2m or more) and the off-take is comparatively small.

The tunnels should be at-least 0.6m deep. The upstream and downstream edges of the platform should be inclined at about 30^0 to the parent canal cross section. At times, silt vanes can be combined with a skimming platform. In that case, the piers of the platform are extended downstream in the form of vanes. A typical plan and section view of a skimming platform is shown in Figure 25.

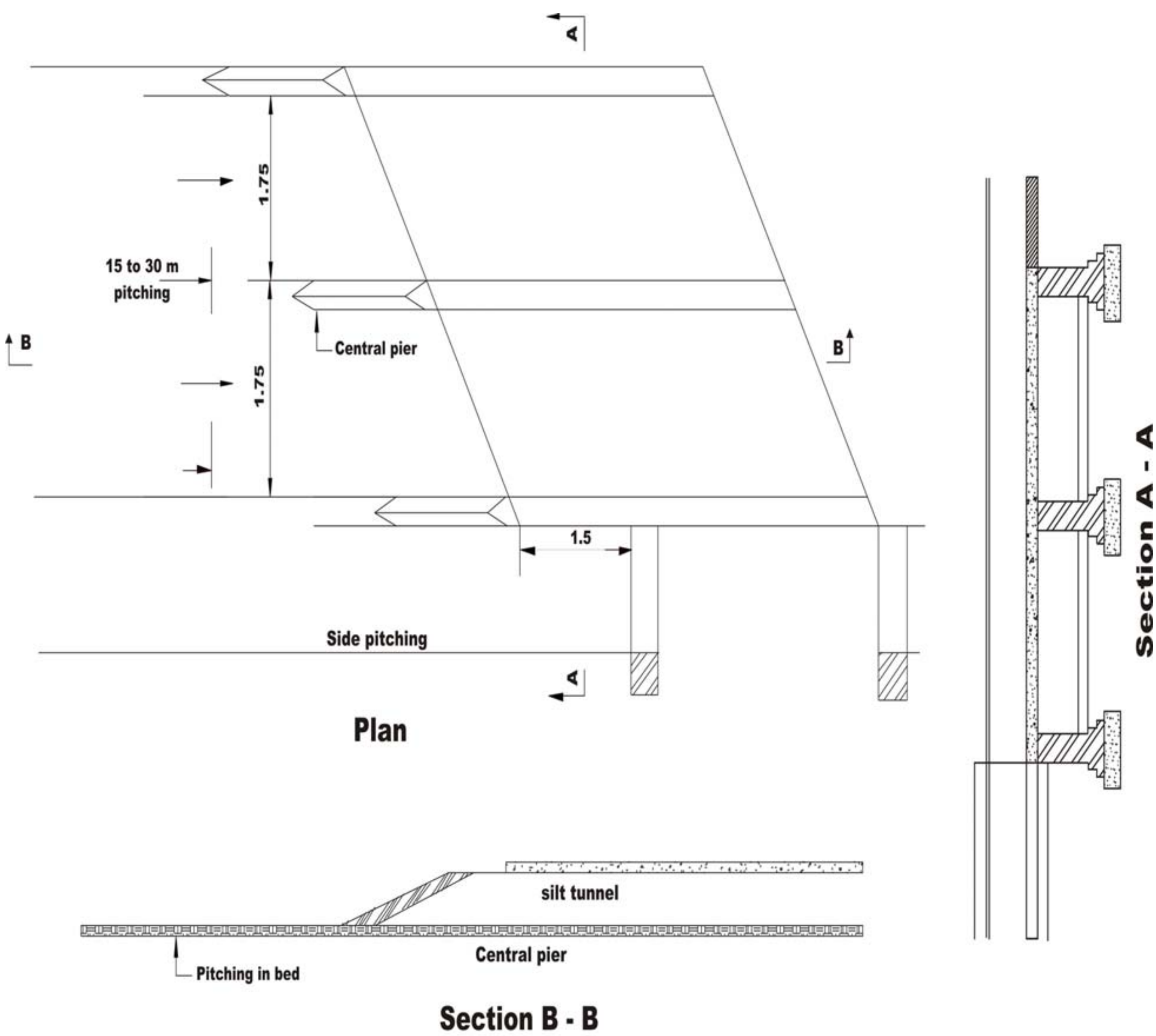


FIGURE 25. SILT (SKIMMING) PLATFORM

3.9.8 Canal escapes

These are structures meant to release excess water from a canal, which could be main canal, branch canal, distributary, minors etc. Though usually an irrigation system suffers from deficit supply in later years of its life, situations that might suddenly lead to accumulation of excess water in a certain reach of a canal network may occur due to the following reasons:

- Wrong operation of head works in trying to regulate flow in a long channel resulting in release of excess water than the total demand in the canal system downstream.

- Excessive rainfall in the command area leading to reduced demand and consequent closure of downstream gates.
- Sudden closure of control gates due to a canal bank breach.

The excess water in a canal results in the water level rising above the full supply level which, if allowed to overtop the canal banks, may cause erosion and subsequent breaches. Hence, canal escapes help in releasing the excess water from a canal at times of emergency. Moreover, when a canal is required to be emptied for repair works, the water may be let off through the escapes.

Escapes as also built at the tail end of minors at the far ends of a canal network. These are required to maintain the required full supply level at the tail end of the canal branch.

The construction feature of escapes allows it to be classified in to two types, as described below.

3.9.8.1 Weir or surface escapes

These are constructed in the form of weirs, without any gate or shutter (Figure 26) and spills over when the water level of the canal goes above its crest level

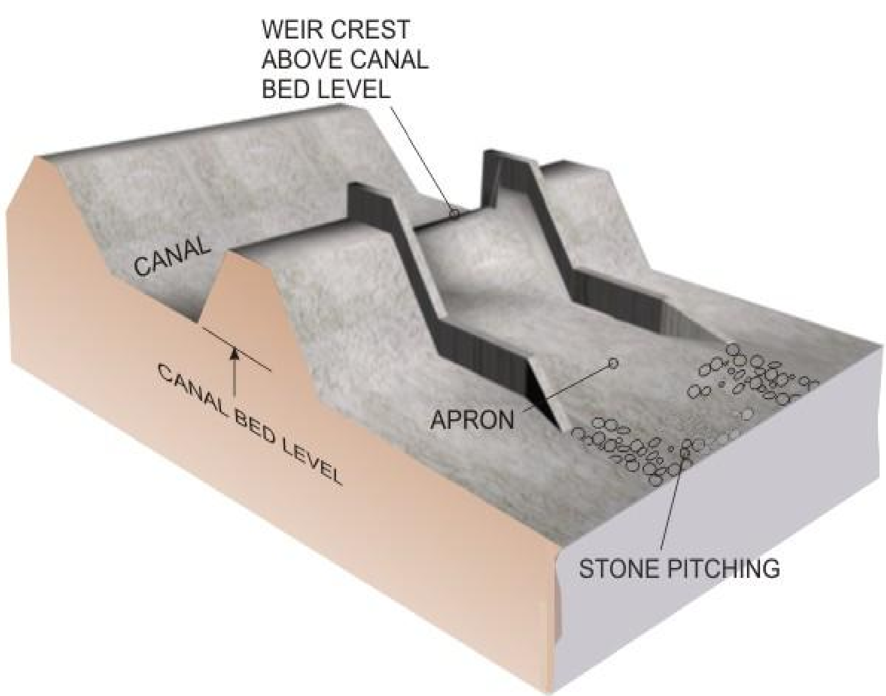


FIGURE 26. Weir Escape

Energy dissipation structures not shown

3.9.8.2 Sluice or surplus escapes

These are gated escapes with a very low crest height (Figure 27). Hence, these sluices can empty the canal much below its full supply level and at a very fast rate. In some cases, these escapes act as scouring sluices to facilitate removal of sediment.

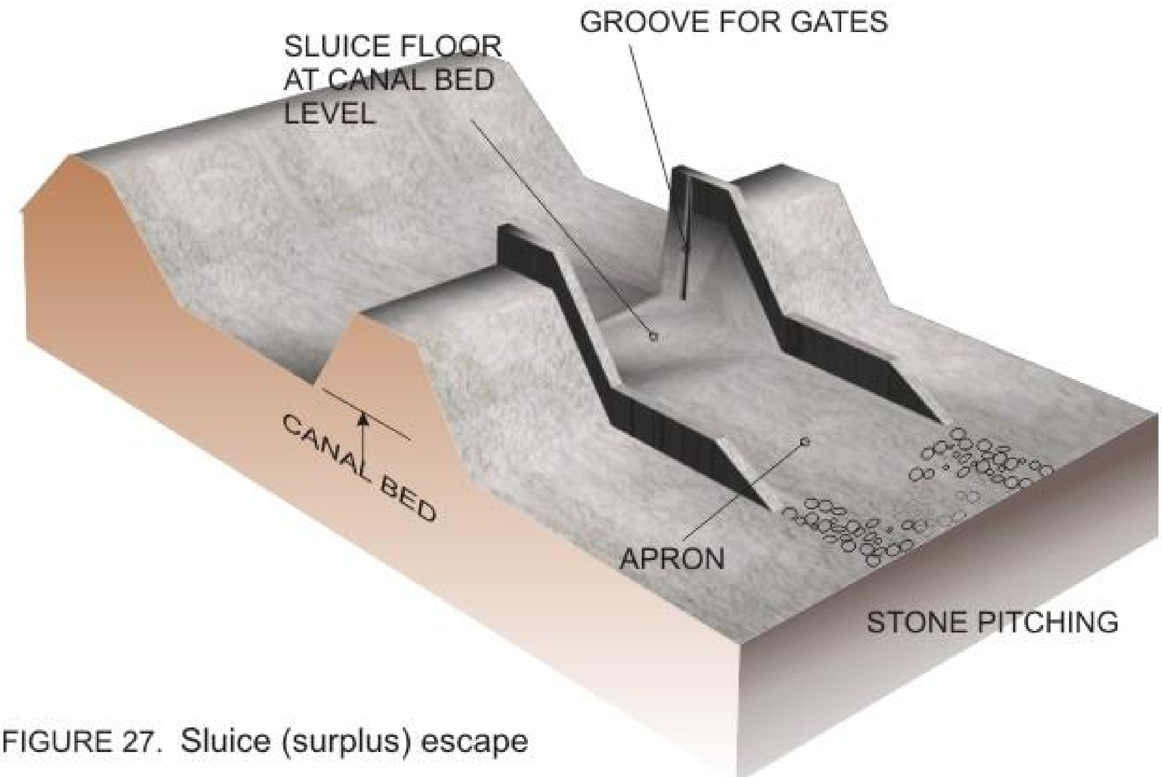


FIGURE 27. Sluice (surplus) escape

The locations for providing escapes are often determined on the availability of suitable drains, depressions or rivers with their bed level at or below the canal bed level so that any surplus water may be released quickly disposed through these natural outlets. Escapes may be necessary upstream of points where canals takeoff from a main canal branch. Escape upstream of major aqueducts is usually provided. Canal escapes may be provided at intervals of 15 to 20km for main canal and at 10 to 15km intervals for other canals.

The capacity of an escape channel should be large enough to carry maximum escape discharge. These should be proper energy dissipation arrangements to later for all flow conditions. The structural and hydraulic design would be similar to that of regulators or sluices or weirs, as appropriate.

The Bureau of Indian Standards code IS 6936:1992 (reaffirmed 1998) “Guide for location, selection and hydraulic design of canal escapes” may be referred to for further details.

References

- FAO Irrigation and Drainage paper 26/1: Small Hydraulic Structures, Volume 1 (1982)
- FAO Irrigation and Drainage paper 26/2: Small Hydraulic Structures, Volume 2 (1982)
- Garg, S K (1996) "Irrigation engineering and hydraulic structures", Twelfth Edition, Khanna Publishers
- IS 4410 : Part 15 : Sec 4 : 1977 Glossary of terms relating to river valley projects: Part 15 Canal structures Section 4 Regulating works
- IS 6522 : 1972 Criteria for design of silt vanes for sediment control in offtaking canals
- IS 6531 : 1994 Canal Head Regulators - Criteria for Design
- IS 6936 : 1992 Guide for location, selection and hydraulic design of canal escapes
- IS 7114 : 1973 Criteria for hydraulic design of cross regulators for canals
- IS 7495 : 1974 Criteria for hydraulic design of silt selective head regulator for sediment control in offtaking canals
- IS 7880 : 1975 Criteria for hydraulic design of skimming platform for sediment control in offtaking canal
- IS: 6936-1992 (reaffirmed 1998) "Guide for location, selection and hydraulic design of canal escapes"
- IS: 7114-1973 "Criteria for hydraulic design of cross regulators for canals"
- Varshney, R S, Gupta, S C and Gupta, R L (1993) "Theory and design of irrigation structures", Volume II, Sixth Edition, Nem Chand Publication

Module 3

Irrigation Engineering Principles

Lesson 10

Distribution and Measurement Structures for Canal Flows

Instructional objectives

On completion of this lesson, the student shall learn:

1. What are flow distributing and measurement structures in a canal system
2. What is the function of outlets and what are their classifications
3. What are modules
4. How are notches and weirs used to measure flow in a canal
5. How are flumes used to measure flow in a canal

3.10.0 Introduction

The flow of a main canal bifurcating into a branch canal with the rest flowing downstream is controlled with the help of a cross regulator across the parent canal and a head regulator across the branch canal. At times, the flow of a canal divides into two or three smaller branch canals without any regulating structure by designing the entrance of the canals IN such a way that the flow enters each branch canal proportionate to its size. Again, from a canal, outlet structures may take out water for delivery to the field channel or water courses belonging to cultivators. These outlet works, of course, are generally not provided on the main canal and branches, but are installed in the smaller distributaries. Apart from these, there could be a need to measure the flow in a canal section and different structures have been tried, mostly based on the formation of a hydraulic jump and calibrating the discharge with the depths of flow. Typical structures of these kinds are graphically represented in Figure 1 and this lesson deals with each type in detail.

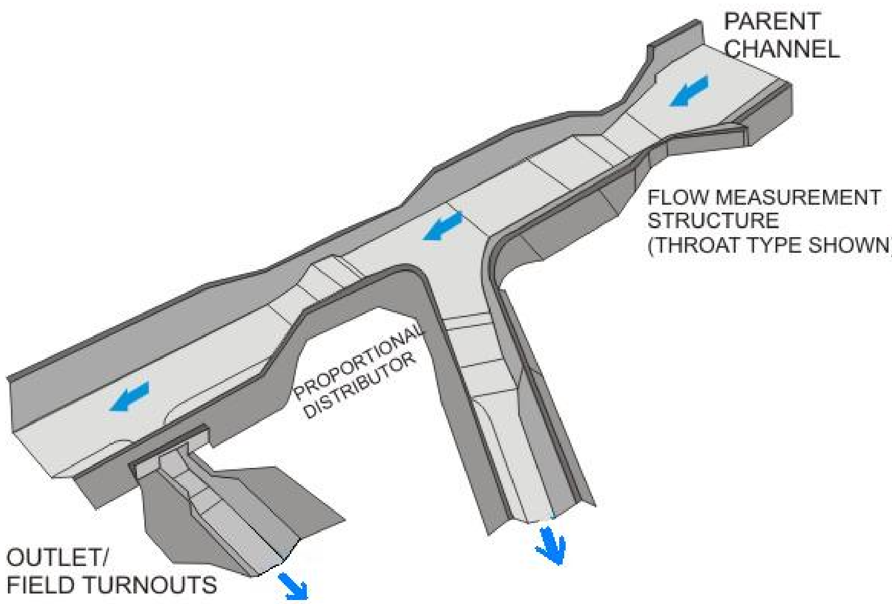


FIGURE 1. Canal structures for flow distribution and measurement

3.10.1 Flow distributing structures

The flow of a canal can be distributed in to smaller branches using a variety of structures which have been developed to suit a wide variety of conditions. The flow being diverted in to each branch is usually defined as a proportion of the total flow. Thus, these flow distributing structures differ from the flow regulating structures discussed in Lesson 3.9 since the latter are designed to draw off any amount of discharge irrespective of the flow in the parent channel. The flow distributing structures require a control section in both the off-take channel and in the parent channel. Flow distributors of fixed proportion type are generally used in India, whereas in some countries a flow splitter with a mechanical arrangement is used to change the flow distribution proportions.

The Punjab type proportional distributor has each opening or ***offtake*** constructed as a flume or ***free overfall weir*** and is dimensioned so as to pass a given fraction of the total flow. The controlling section consisting of the flume, elevated floor or weir crest is located in the individual offtakes, and not in the supply channel. A typical plan of a proportional distributor with two offtakes is shown in Figure 2.

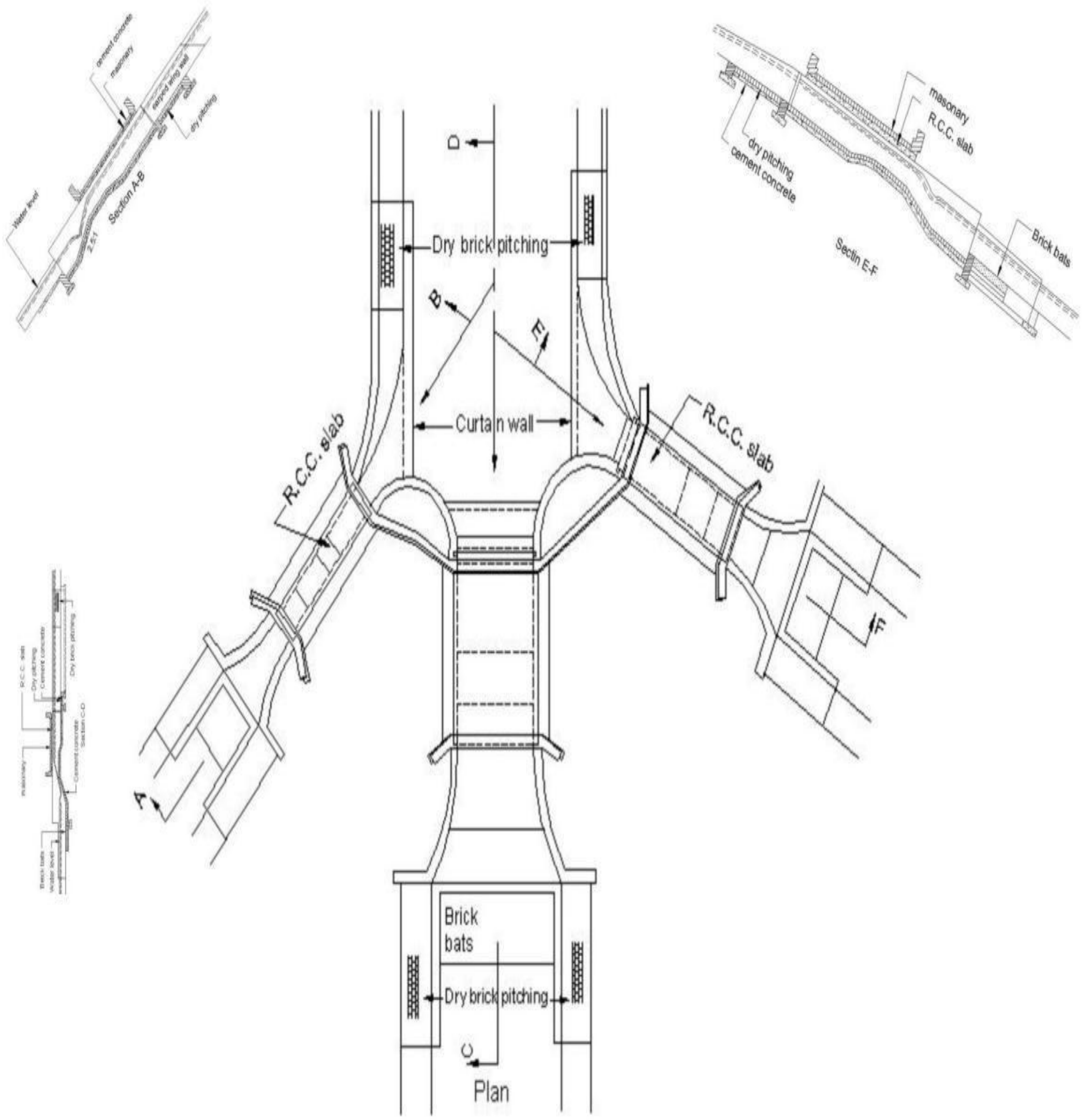


FIGURE 2. Proportional distributor structure with two offtakes

Proportional distributors may have only one offtake as shown in the typical plans shown in Figure 3. Generally, all offtakes should be designed to bifurcate at 60^0 or 45^0. The crest of all the offtakes and the flume in the parent channel should be at the same level and at least 0.15m above the downstream bed level of the highest channel. The parent channel flume may have provisions for a stop log insertion for emergency closures.

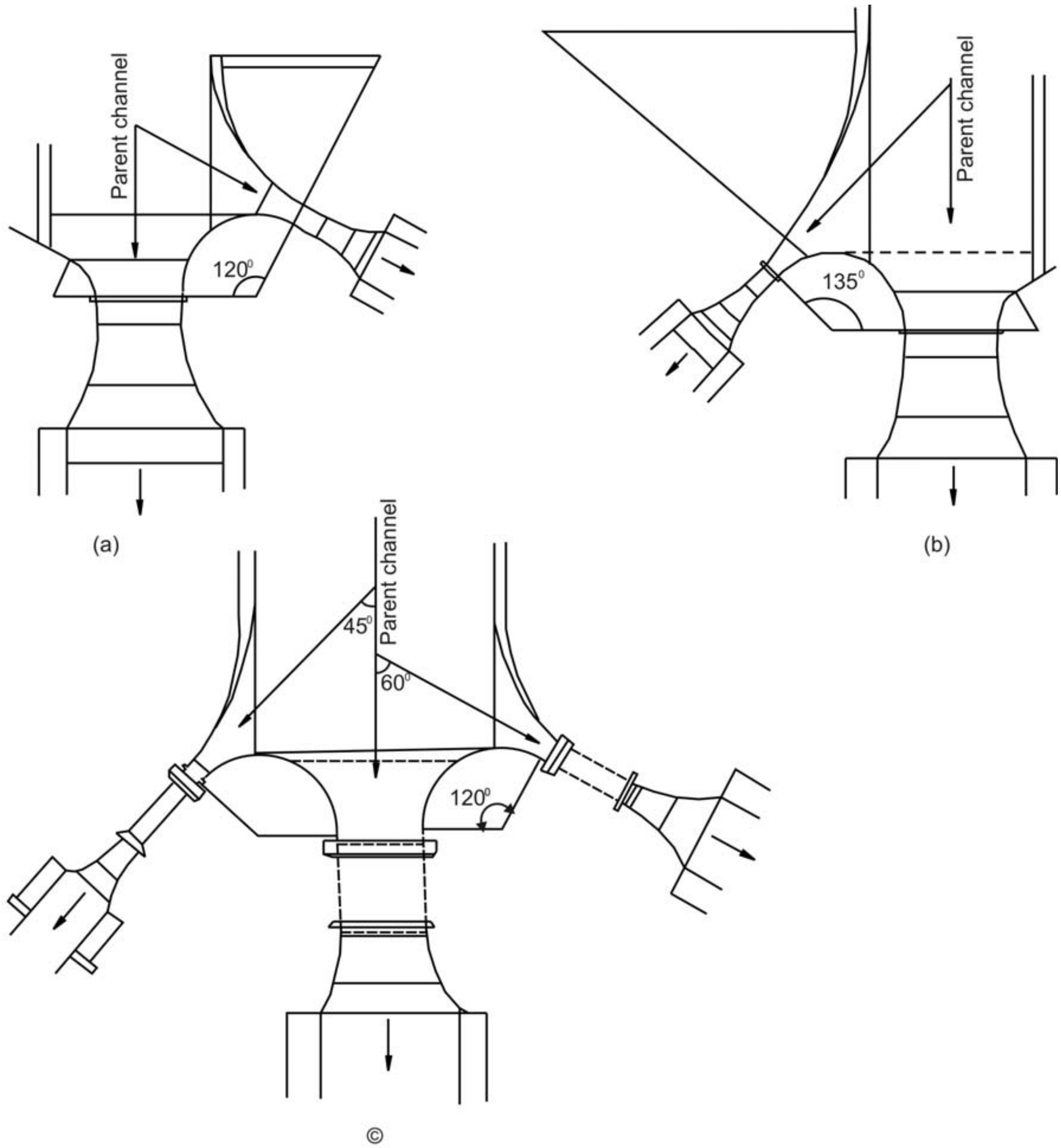


FIGURE 3. PROPORTIONAL OFFTAKES WITH (a) ONE OFFTAKE ON LEFT SIDE (b) ONE OFFTAKE ON RIGHT SIDE (c)ONE OFFTAKE ON EITHER SIDE

3.10.2 Canal outlet

Canal ***outlets***, also called ***farm turnouts*** in some countries, are structures at the head of a ***water course*** or ***field channel***. The supply canal is usually under the control of an irrigation authority under the State government. Since an outlet is a link connecting the government owned supply channel and the cultivator owned field channel, the requirements should satisfy the needs of both the groups.

Since equitable distribution of the canal supplies is dependent on the outlets, it must not only pass a known and constant quantity of water, but must also be able to measure the released water satisfactorily. Also, since the outlets release water to each and every farm watercourse, such structures are more numerous than any other irrigation structure. Hence it is essential to design an outlet in such a way that it is reliable and be also robust enough such that it is not easily tampered with. Further the cost of an outlet structure should be low and should work efficiently with a small working head, since a larger working head would require higher water level in the parent channel resulting in high cost of the distribution system. Discharge through an outlet is usually less than 0.085 cumecs.

Various types of canal outlets have been evolved from time to time but none has been accepted as universally suitable. It is very difficult to achieve a perfect design fulfilling both the properties of 'flexibility' as well as 'sensitivity' because of various indeterminate conditions both in the supply channel and the watercourse of the following factors:

- Discharge and silt
- Capacity factor
- Rotation of channels
- Regime condition of distribution channels, etc.

These modules are classified in three types, which are as follows:

(a) Non-modular outlets

These outlets operate in such a way that the flow passing through them is a function of the difference in water levels of the distributing channel and the watercourse. Hence, a variation in either affects the discharge. These outlets consist of regulator or circular openings and pavement. The effect of downstream water level is more with short pavement.

(b) Semi-modular outlets

The discharge through these outlets depend on the water level of the distributing channel but is independent of the water level in the watercourse so long as the minimum working head required for their working is available.

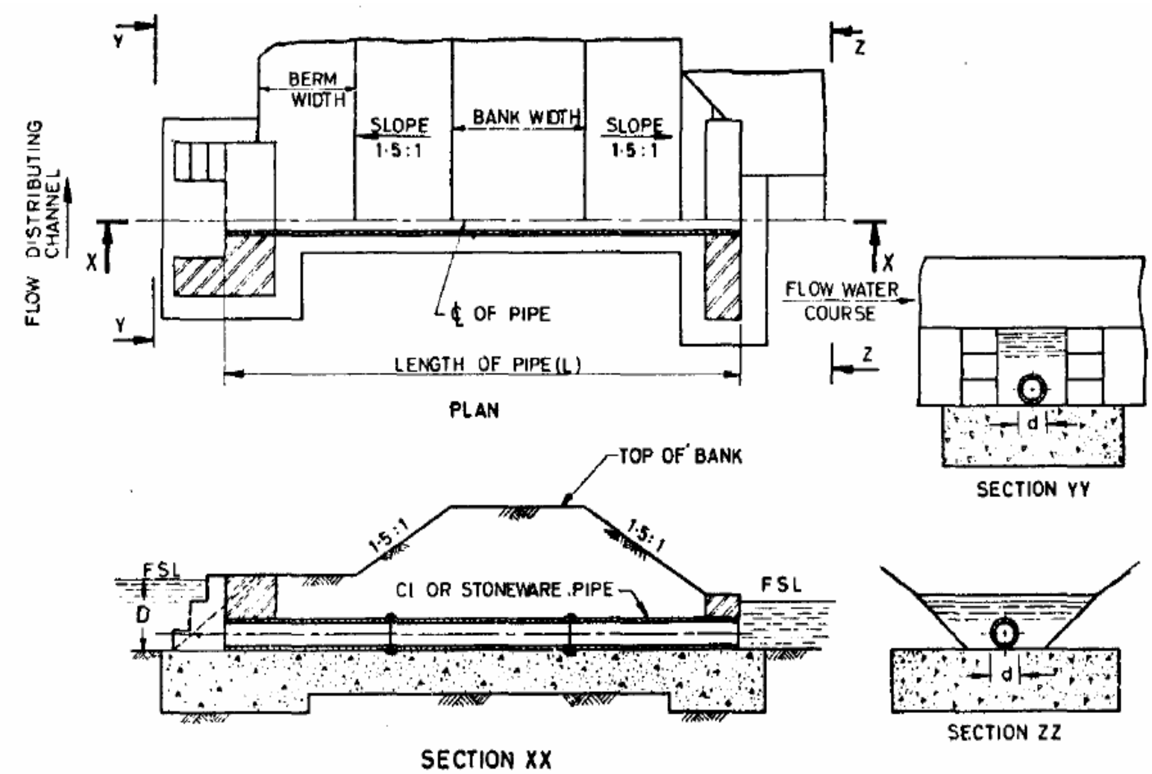
(c) Module outlets

The discharge through modular outlets is independent of the water levels in the distributing channel and the watercourse, within reasonable working limits. This type of outlets may or may not be equipped with moving parts. Though modular outlets, like the Gibb's module, have been designed and implemented earlier, they are not very common in the present Indian irrigation engineering scenario.

The common types of outlets used in India are discussed in the next sections.

3.10.3 Pipe outlets

This is a pipe with the exit end submerged in the watercourse (Figure 4). The pipes are placed horizontally and at right angles to the centre line of the distributing channel and acts as a non-modular outlet.



Discharge through the pipe outlet is given by the formula:

$$Q = CA\,(2gH)^{1/2} \qquad (1)$$

In the above equation, ***Q*** is the discharge; ***A*** is the cross sectional area; ***g*** is the acceleration due to gravity; ***H*** is difference in water levels of supply channel and watercourse and ***C*** is the coefficient of discharge which depends upon friction factor (***f***), length (***L***) and diameter of the outlet pipe (***d***) related by the formula:

$$C = \frac{1}{2 \times 10^5} \sqrt{\frac{d}{f\left(L + \frac{1.5d}{400\,f}\right)}} \qquad (2)$$

The coefficient ***f*** is the fluid friction factor and its value may be taken as 0.005 and 0.01 for clear and encrusted iron pipes respectively. For earthenware pipes, ***f*** may be taken as 0.0075. All other variables are in SI units, that is, meters and seconds.

It is a common practice to place the pipe at the bed of the distributing channel to enable the outlets to draw proportional amount of silt from the supply channel. The entry and exit ends of the pipe should preferably be fixed in masonry to prevent tampering. Since the discharge through this type of outlet can be increased by lowering the water surface level of the watercourse (thus increasing the value of H in the discharge equation), it is possible for the irrigator to draw more than fair share of water. A pipe outlet may also be designed as a semi-modular outlet, that is, one which does not depend upon the water level in the watercourse by allowing it to fall freely in to the watercourse (Figure 5).

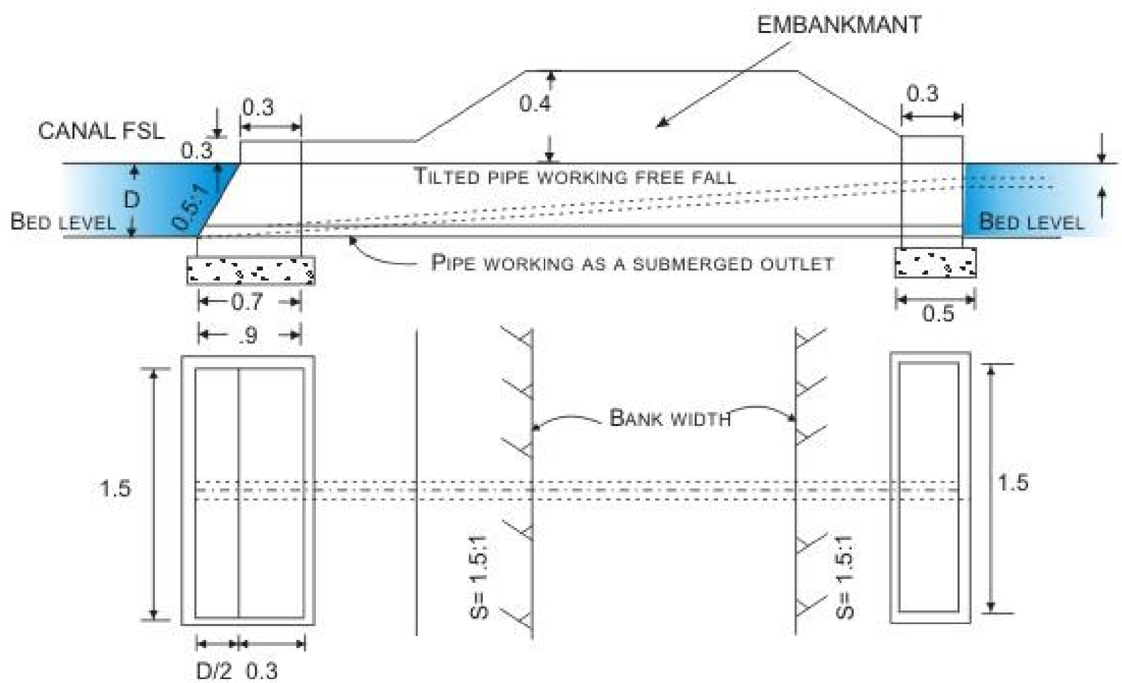


FIGURE 5. Pipe outlet with exit above water- course FSL

Pipe outlets require minimum working head and have higher efficiency. It is also simple and economical to construct and is suitable for small discharges. However, these outlets suffer from disadvantages like the coefficient discharge which varies from outlet to outlet and at the same outlet at different times apart from the possibility of tampering in the non-modular type.

3.10.4 Open flume outlets

This is a smooth weir with a throat constricted sufficiently long to ensure that the controlling section remains with in the parallel throat for all discharges up to the maximum (Figure 6). Since a hydraulic jump forms at the control section, the water level of the watercourse does not affect the discharge through this type of outlet. Hence this is a semi-modular outlet.

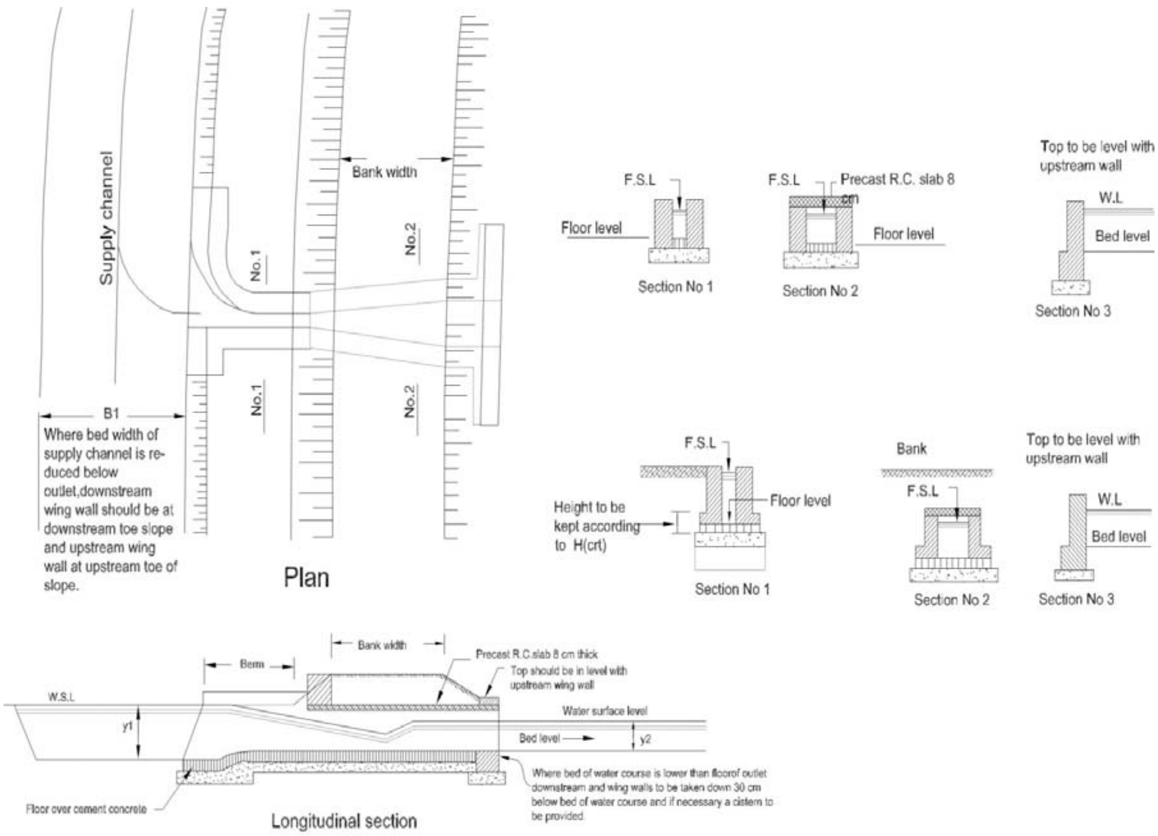


FIGURE 6. OPEN FLUME OUTLET

This type of structure is built in masonry, but the controlling section is generally provided with cast iron or steel bed and check plates. The open flumes can either be deep and narrow or shallow and wide in which case it fails to draw its fair share of silt. Generally, this type of outlet does not cause silting above the work, except when supplies are low for a considerable length of time. The silt which gets accumulated gets washed away during high supplies.

The open flume outlet is also cheaper than the Adjustable Proportional Module (APM), discussed below. The discharge formula for the open flume outlet is given as:

$$Q = C\,B_t\,H^{3/2} \qquad (3)$$

Where ***Q*** (given in l/s) is related to the coefficient of discharge, ***C***, as given in the table below; B_t is the width of the throat in cm; and ***H*** is the height of the full supply level of the supply channel above the crest level of the outlet in cm.

B_t (cm)	***C***
6 to 9	0.0160
> 9 to12	0.0163
> 12	0.0166

The minimum head required to drive the outlet is about 20 percent of H.

3.10.5 Adjustable Proportional Module (APM)

There are various forms of these outlets but the earliest of them is the one introduced by E.S. Crump in 1992. In this type of outlet, a cast iron base, a cast iron roof block and check plates on either are side are used to adjust the flow and is set in a masonry structure (Figure 7). This outlet works as a semi-module since it does not depend upon the level of water in the watercourse.

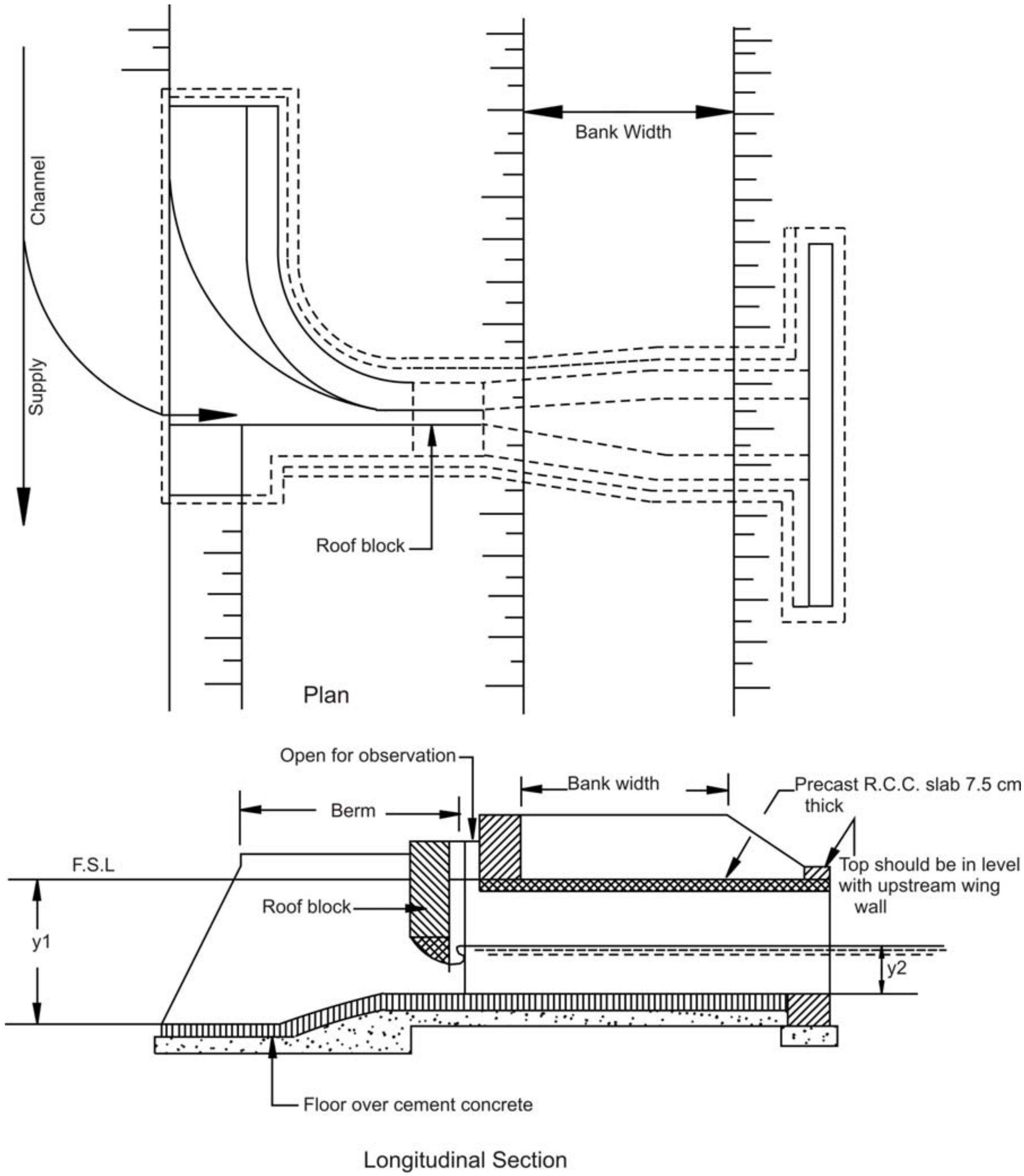


FIGURE 7. PLAN AND SECTION OF ADJUSTABLE PROPORTIONAL MODULE

The roof block is fixed to the check plates by bolts which can be removed and depth of the outlet adjusted after the masonry is dismantled. This type of outlet cannot be easily tampered with and at the same time be conveniently adjusted at a small cast.

The roof blocks may also be built of reinforced concrete. The face of the roof block is set 5 cm from the starting point of the parallel throat. It has a lamniscate curve at the bottom with a tilt of 1 in 7.5 in order to make the water converge instead of a horizontal base which would cause it to diverge. The cast iron roof block is 30cm thick.

As such, the APM is the best type of outlet if the required working head is available and is the most economical in adjustment either by raising or lowering the roof block or crest. However, it is generally costlier than the other types of outlets and also requires more working head.

The discharge formula for this type of weir is given as:

$$Q = C\, B_t\, H_1 (H_2)^{1/2} \qquad (4)$$

Where ***Q*** (given in l/s) is related to the coefficient of discharge, C, which is taken equal to around 0.0403; $\boldsymbol{B_t}$ is the width of the throat in cm; $\boldsymbol{H_1}$ is the depth of head available, that is the difference between the supply channel full supply level and the outlet bed (crest) level; and $\boldsymbol{H_2}$ is the difference between the supply channel full supply level and the bottom level of the roof block (Figure 8) .

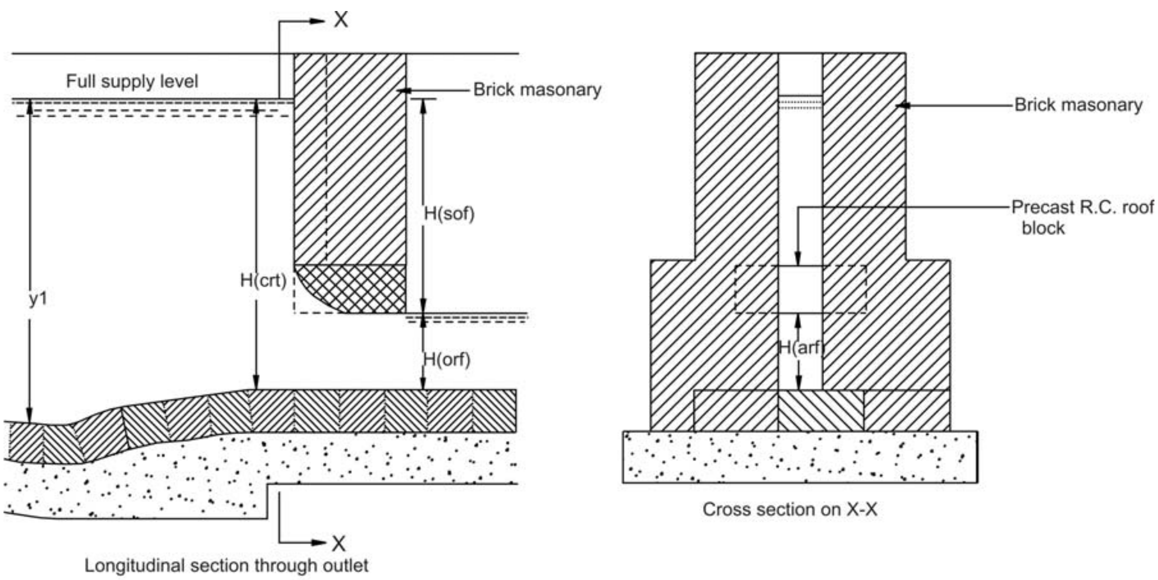
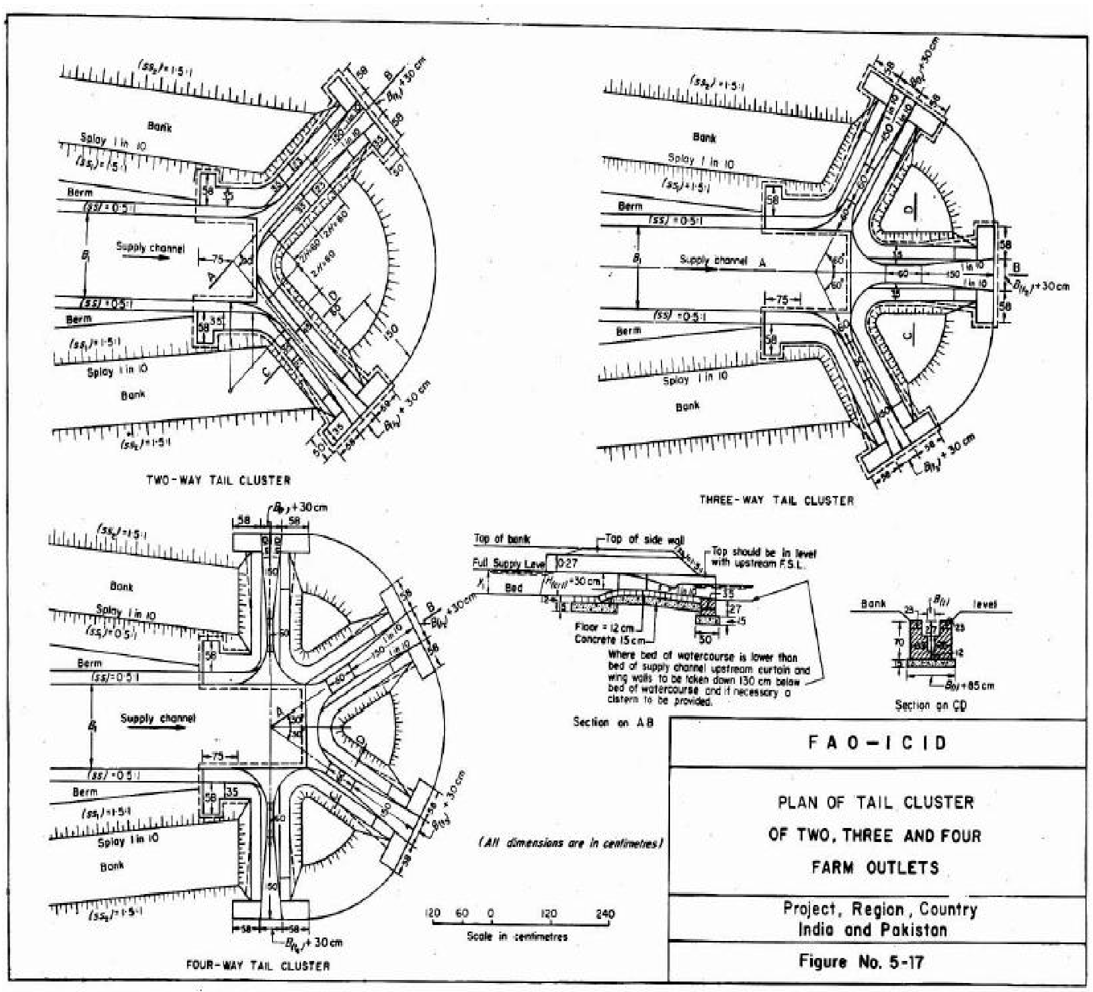


FIGURE 8.DETAILS OF BLOCK FOR ADJUSTABLE PROPORTIONAL MODULE

The base plates and the roof block are manufactured in standard sizes, which with the required opening of the orifice are used to obtain the desired supply through the outlet.

3.10.5 Tail clusters

When the discharge of a secondary, tertiary or quaternary canal diminishes below 150 l/s, it is desirable to construct structures to end the canal and distribute the water through two or more outlets, which is called a tail cluster. Each of these outlets is generally constructed as an open flume outlet (Figure 9).



3.10.6 Flow measurement in canals

The available water resources per person are growing scarcer with every passing day. Although a region may not face a net reduction in water resources, the increasing population of the area would demand increased food production and consequently, agricultural outputs. Such that an equitable distribution of water is ensured as far as possible with a command area, it is required to measure water at important points in the canal network. Measurements may also help in estimating and detecting losses in the canal.

Further, at the form level, advanced knowledge of soil properties and soil moisture / plant relationships permits irrigation systems to be designed so that water can be applied in the right amount and at the right time in relation to the soil moisture status thereby obtaining maximum efficiency of water use and minimum damage to the land. This knowledge can be utilized most effectively only by reasonably accurate measurement of the water applied.

The amount of water being delivered to a field of an irrigator should also be measured in order to make an assessment of water charges that may be levied on him. If the charge to the user of canal water is based on the rate flow, then rate-of-flow measurements and adequate records are necessary. Charges on the basis of volume of water delivered necessitate a volumetric measuring device. Ideally, water flow should be measured at intakes from storage reservoirs, canal head works, at strategic points in canals and laterals and at delivery points to the water users. The most important point for measurement is the form outlet which is the link between the management authority of the canal system and the user.

The degree of need for a measuring device at the outlet varies according to the delivery system employed. Delivery on demand usually relies up on the measurement of water as a basis for equitable distribution as well as for computing possible water charges. Where water is distributed by rotation among farmers along a lateral (or distributary or minor canal) and the where the amount of water supplied to each farmer may be different, a measuring device at the turnout is required. On the other hand, if farmers along a lateral receive water on the basis of area of land or crops irrigated measurement is not entirely necessary, but may still be desirable for other purposes, such as improvement of irrigation efficiency. Similarly in all systems based on constant flow, measurement is not entirely necessary but may be advantageous.

Where several farmers share the water of each outlet and the flow in the canal fluctuates considerably, each such outlet should be equipped with a measuring device, even if equitable distribution among outlets is practiced, so that each group of farmers will know the flow available at any one time from their respective outlet.

Amongst the methods and devices used for measuring water in an irrigation canal network, the weir is the most practical and economical device for water measurement, provided there is sufficient head available. Measuring flumes are also used in irrigation networks and their advantage are smaller head losses, reasonable accuracy over a large flow range, insensitivity to velocity of approach, and not affected much by sediment load. Propeller meters are used in many countries and are particularly suited to systems where no head loss can be permitted for water measurement and where water is sold on volumetric basis. For water measurement in small streams, particularly in field ditches and furrows and where head losses must be small, the deflection or vane meter has proved to be a useful device. Only the weir and the standing wave (hydraulic jump) type flume are discussed in this lesson as these are most commonly used.

3.10.7 Weirs

Weirs have been in use as discharge measuring devices in open channels since almost two centuries and are probably the most extensively used devices for measurement of the rate of flow of water in open channels. Weirs may be divided in to sharp and broad crested types. The broad crested weirs are commonly incorporated in irrigation structures but are not usually used to determine flow. The types of sharp crested weirs commonly used for measuring irrigation water are the following:

3.10.7.1 Sharp crested rectangular weir

A general view of this type of weir is shown in Figure 10.

WEIR SHAPE

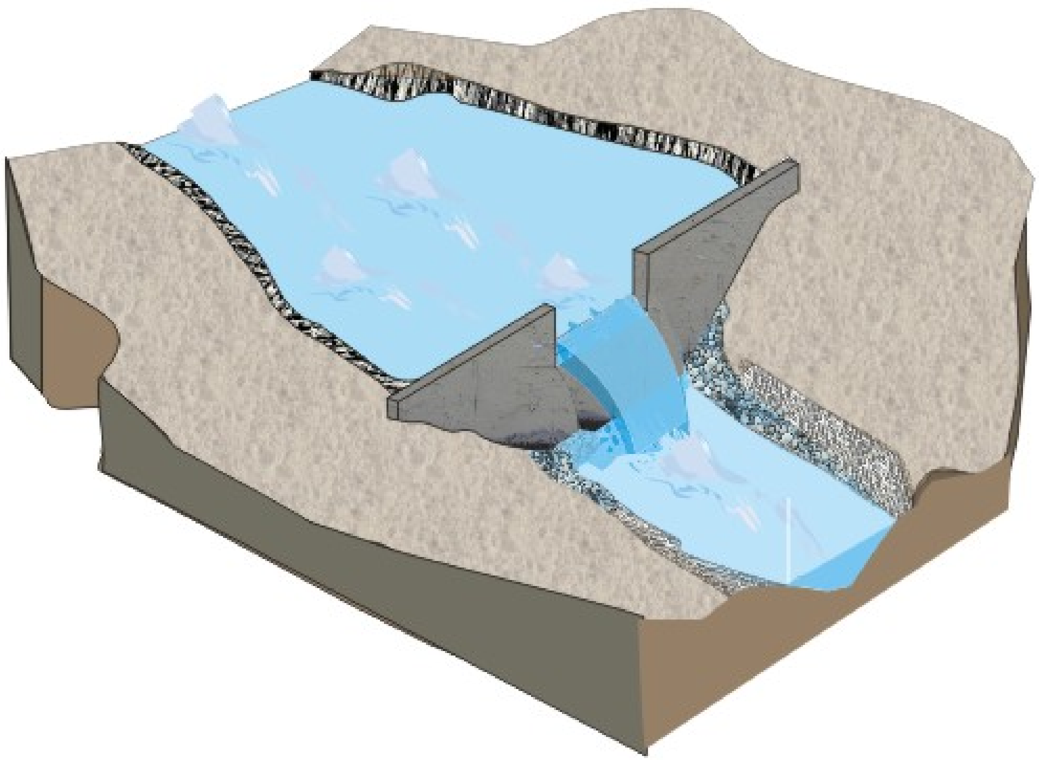


FIGURE 10. General view of a sharp created rectangular weir

Amongst the many formulae developed for computing the discharge of rectangular, sharp crested weirs with complete contraction, the most accepted formula is that by Francis and is given as:

$$Q = 1.84\ (L - 0.2H)\ H^{3/2} \qquad (5)$$

Where Q is the discharge in m^3/s; L is the length of the crest in meters; and H is the head in meters, that is, the vertical difference of the elevation of the weir crest and the elevation of the water surface in the weir pool.

3.10.7.2 Sharp crested trapezoidal (Cipolletti) weir

A general view of this type of weir is shown in Figure 11.

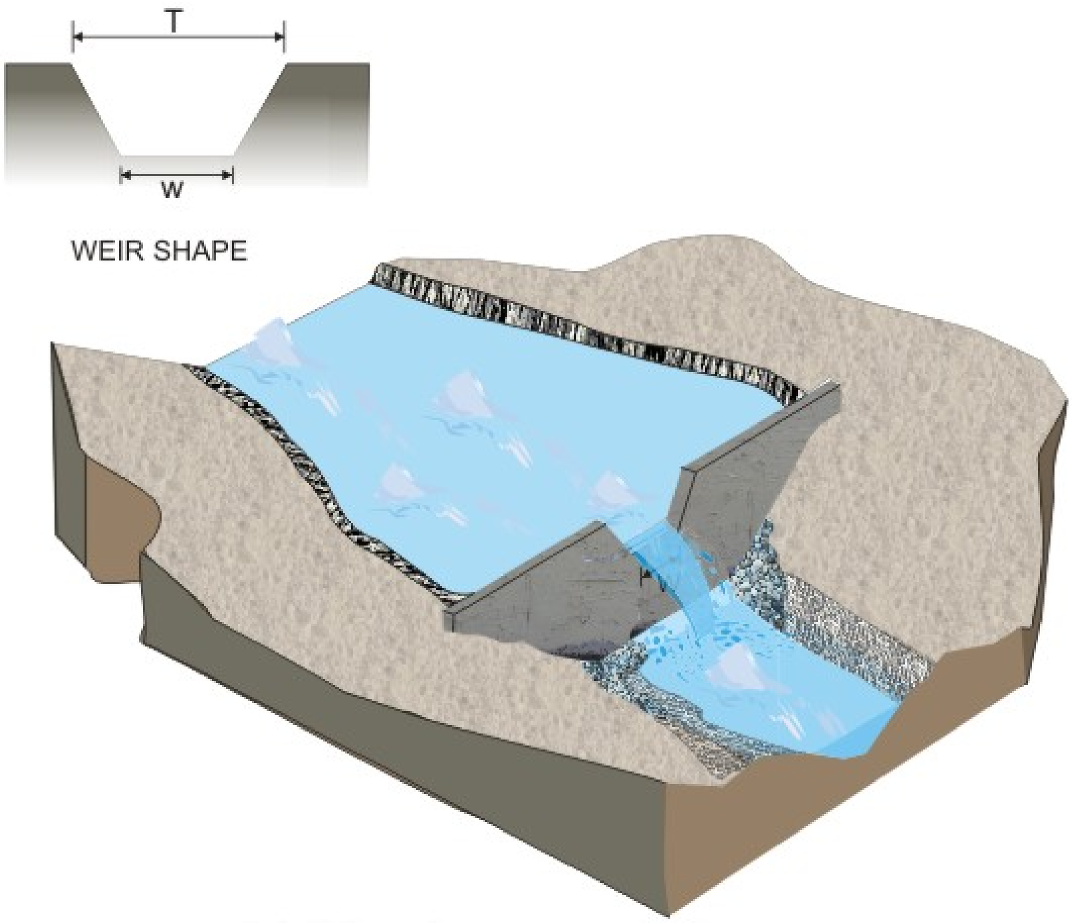


FIGURE 11. General view of a Cipolletti weir

The discharge formula for this type of weir was given by Cipoletti as:

$$Q = 1.86\ L\ H^{3/2} \tag{6}$$

Where Q is the discharge in m^3/s; L is the length of the crest in meters; and H is the head in meters. The discharge measurements using the above formula for the trapezoidal weir are not as accurate as those obtained from rectangular weirs using the Francis formula.

3.10.7.3 Sharp sided 90^0 V-notch weir

A general view of this type of weir is shown in Figure 12.

WEIR SHAPE

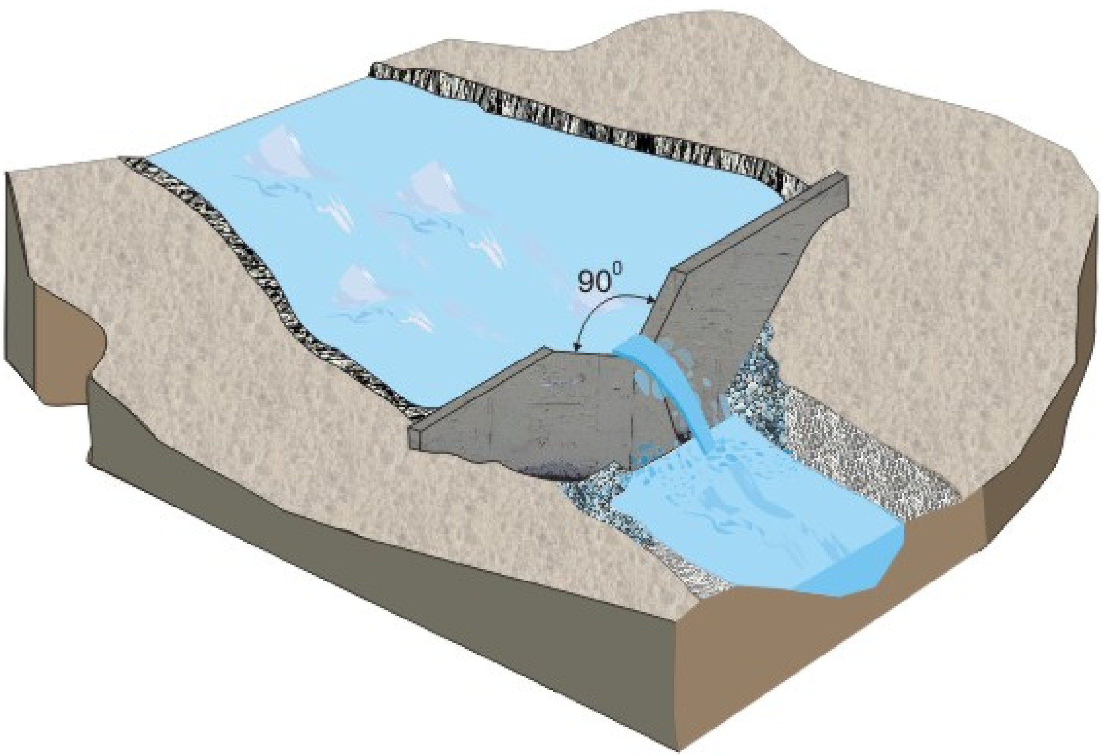


FIGURE 12. General view of 90^0 v-notch weir

Of the several well known formulae used to compute the discharge over 90^0 V-notch weirs the formula recommended generally is the following:

$$Q = 8/15\ (2gC_d)^{1/2}\ H^{5/2} \qquad (7)$$

Where Q is the discharge in m^3/s; g is the acceleration due to gravity ($9.8 m/s^2$); C_d is a coefficient of discharge; and, H is the head in meters. The value of C_d varies according to the variation of H and can be read out from (Figure 13).

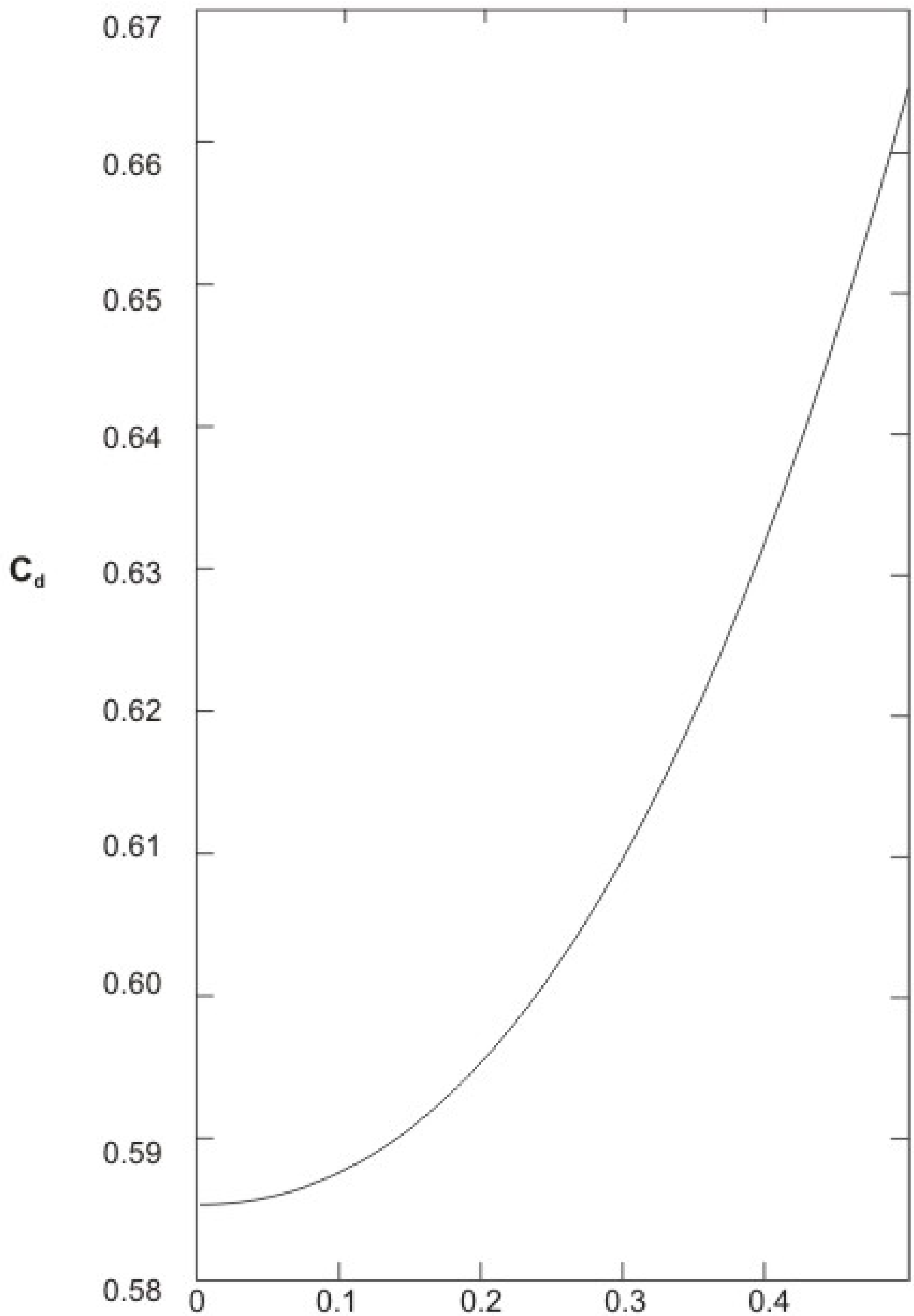


FIGURE 13. Variation of C_d for 90^0V-Notch weir with H^2/A

Each of these weirs has characteristics appropriate to particular operating and site conditions. The 90^0 V-notch weir gives the most accurate results when measuring small discharges and is particularly suitable for measuring fluctuating flows. Weirs require comparatively high heads, considerable maintenance of the weir or stilling pool and protection of the channel downstream of the crest.

3.10.8 Flumes

Flumes are flow measuring devices that works on the principle of forming a critical depth in the channel by either utilizing a drop or by constricting the channel. These two forms of flumes for flow measurement are described below.

3.10.8.1 Flume with a vertical drop

This type of structures has already been discussed in Lesson 3.9 where it was shown to be utilized to negotiate a fall in the canal bed level. One of these, the standing wave flume fall developed at the Central water and Power Research Station (CWPRS), Pune, has been standardized and documented in Bureau of Indian Standard code IS: 6062-1971 "Method of measurement of flow of water in open channels using standing wave flume-fall" and shown in (Figure 14) Because of the inherent free flow conditions the measurement of flow requires only one gauge observation on the upstream side.

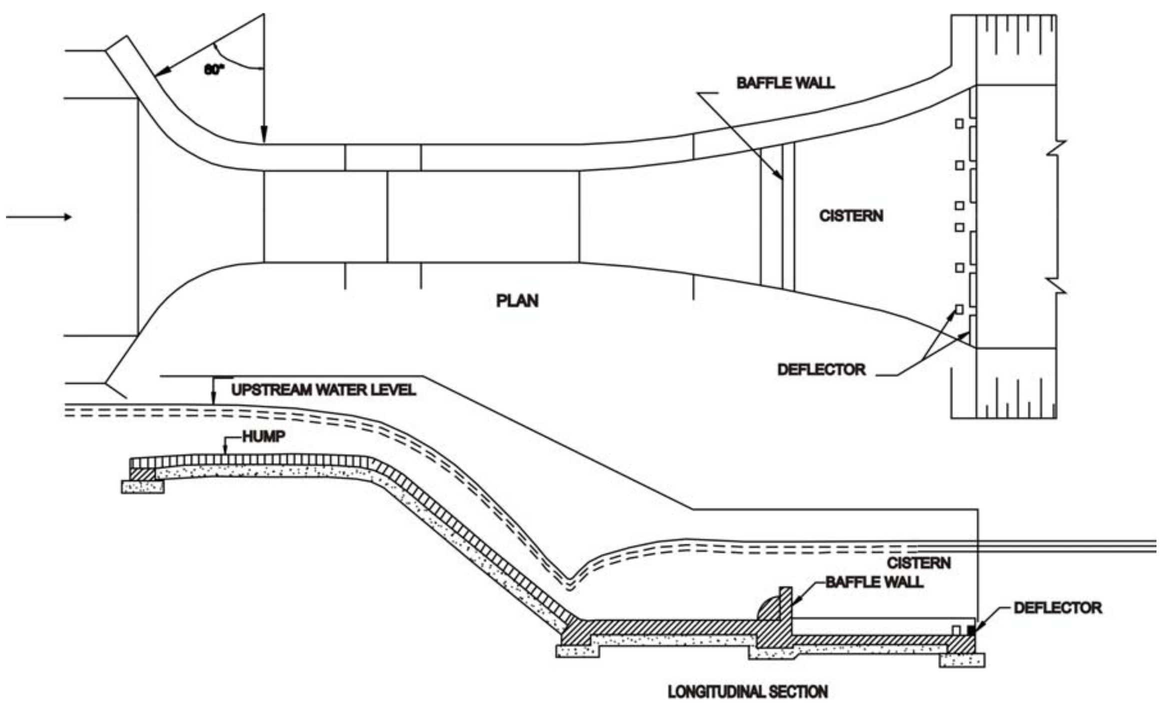


FIG.14. PLAN AND SECTION OH STANDING WAVE FLUME- FALL

The discharge equation for this structure is given by the following equation:

$$Q = 2/3\ (2g)^{1/2}\ CBH^{1.5} \qquad (8)$$

Where Q is the discharge in m^3/s; g is the acceleration due to gravity; C is the coefficient of discharge (=0.97 for $0.05 < Q < 0.3\ m^3/s$ and = 0.98 for $0.31 < Q < 1.5 m^3/s$); B is the width of the flumed section, also called the throat and H is the total head, that is, the depth of water above crest plus the velocity head.

The other type of flume type of fall is the one called the Central Design Office (CDO) Punjab type fall, which is simple and robust in construction. Up to 1m drop, a glacis is used on the downstream side (Figure 15) and if the drop exceeds 1m, the crest ends in a drop wall (Figure 16). The structure is often combined with a bridge, an intake of a third degree canal or both.

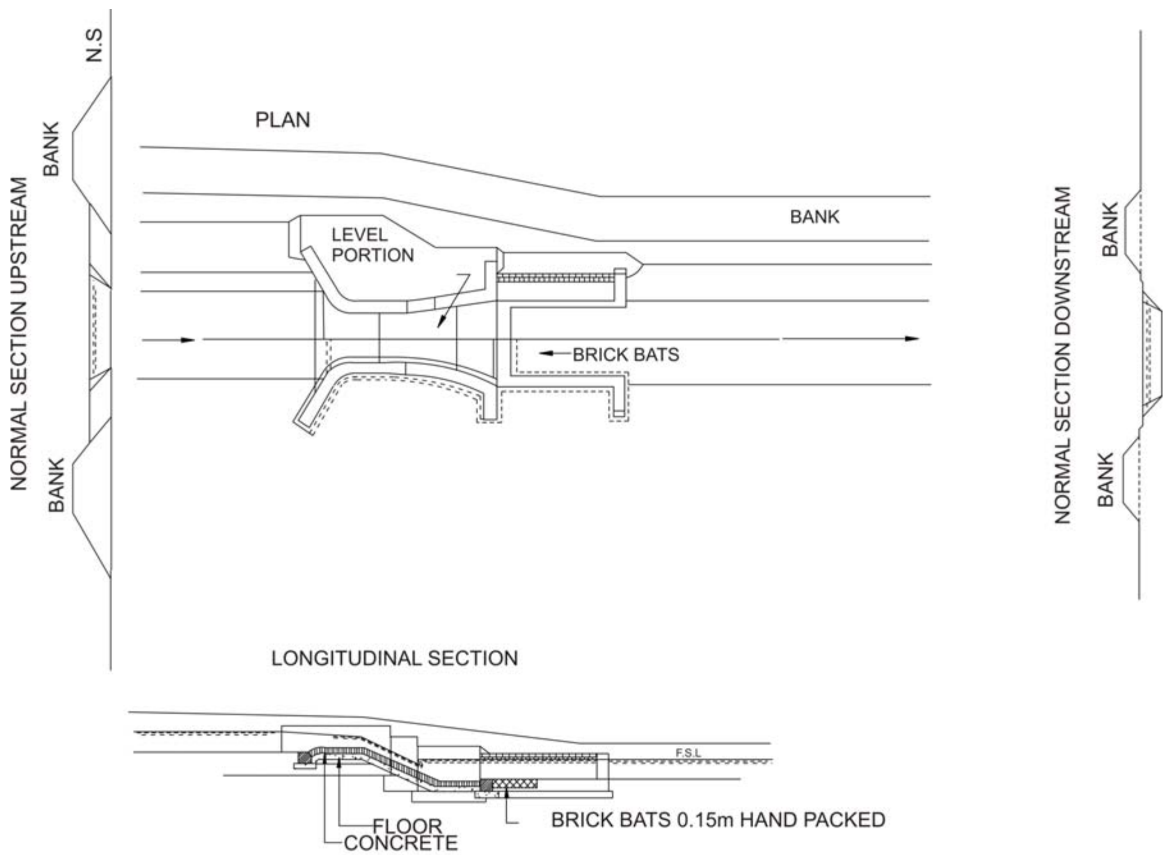


FIGURE.15. CDO PUNJAB TYPE FALL UPTO 1m DROP

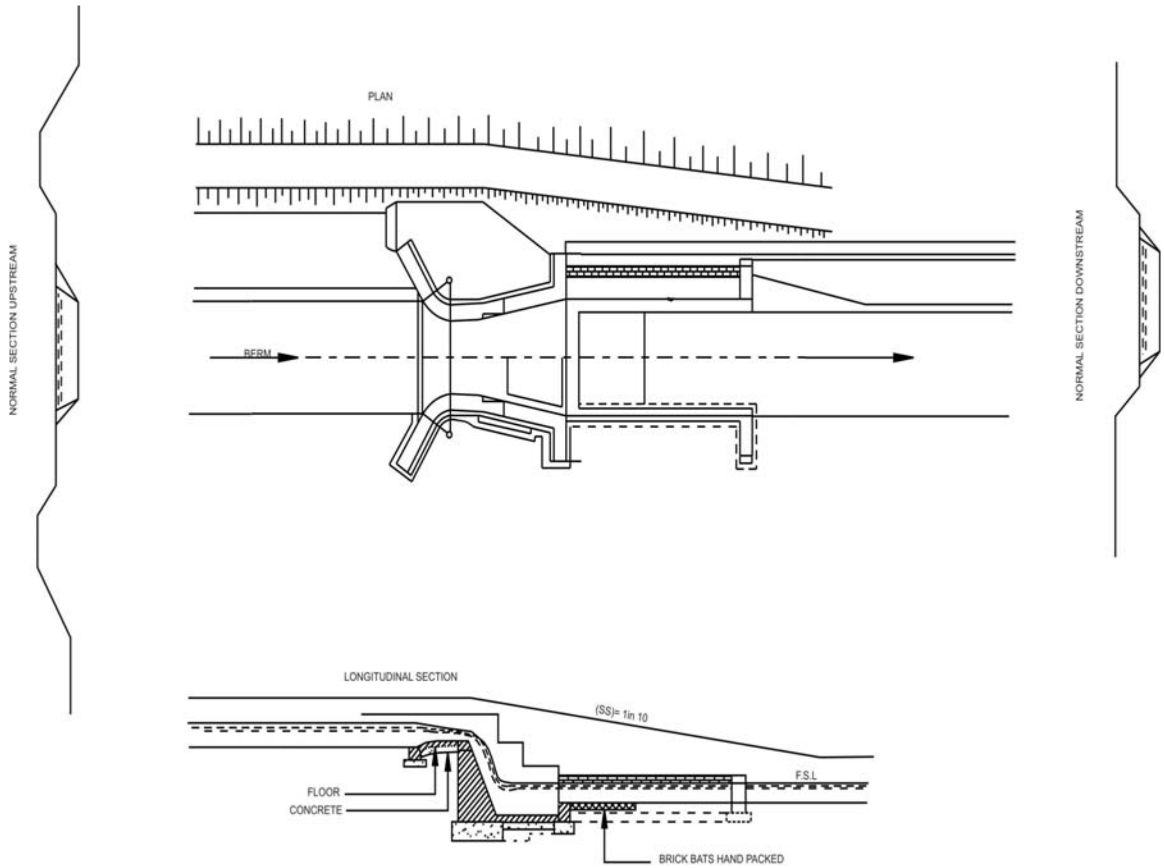


FIGURE.16. CDO PUNJAB TYPE FALL FOR GREATER THAN 1m DROP

3.10.8.2 Flume with a constricted section

This type of structures for measuring water discharge creates a free flow condition followed by a hydraulic jump by providing a very small width at some point with in the flume. These are also further divided in to two types: Long and short-throated flumes. In the former, the constriction is sufficiently long to produce flow lines parallel to the flume crest, for which analytical expression for discharge may be obtained. In the short-throated flumes, the curvature of water surface is large and the flow in the throat is not parallel to the crest of the flume. Hence, due to the non-hydrostatic pressure distribution, there is not analytically derived expression for discharge but has to be calibrated from actual measurements. However, these flumes require small lengths and are economical than long throated flumes. One of the commonly used short-throated flumes is the Parshall flume (Figure 17).

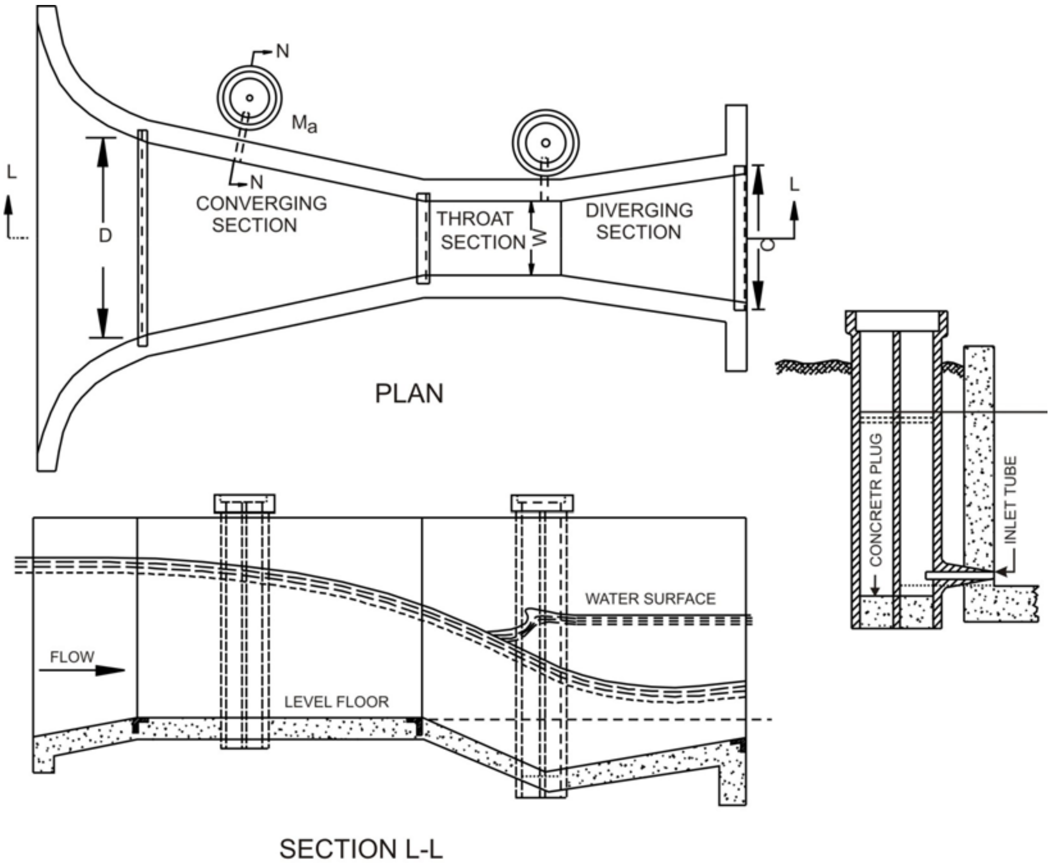


FIGURE 17. PLAN AND SECTION OF PARSHALL MEASURING FLUME

The flume consists of a short parallel throat preceded by a uniformly converging section and followed by a uniformly expanding section. The floor is horizontal in the converging section, slopes downwards in the throat, and is inclined upwards in the expanding section. The control section, at which the depth is critical, occurs near the downstream end of the contraction. There are standard dimensions of Parshall flumes which are available commercially and may be had from the reference "Design of Small Canal Structures" of USBR (1978).

One of the advantages of this type of flume is that it operates with a small head loss, which permits its use in relatively shallow channels with flat grades. For a given discharge, the loss in head through a Parshall flume is only about one fourth that required by a weir under similar free flow conditions. The flume is relatively insensitive to velocity of approach. It also enables good measurements with no submergence (that is free flow with a hydraulic jump downstream) or with submergence (that is the jump is drowned by the downstream water level) as shown in (Figure 17). The velocity of flow within the flume is also sufficient to eliminate any sediment deposition within the structure during operation. A disadvantage of the flume is that standard dimensions must be followed within close tolerance in order to obtain reasonable accuracy of measurement. Further, the flumes cannot be used close to an outlet or regulating devices. Parshall flumes can be constructed in a wide range of sizes to measure discharges from about 0.001m^3/s to 100m^3/s.

References

- FAO Irrigation and Drainage paper 26/1: Small Hydraulic Structures, Volume 1 (1982)
- FAO Irrigation and Drainage paper 26/2: Small Hydraulic Structures, Volume 2 (1982)
- Garg, S K (1996) “Irrigation engineering and hydraulic structures”, Twelfth Edition, Khanna Publishers
- IS 12331 : 1988 General Requirements for Canal Outlets
- IS 7986 : 1976 Code of practice for canal outlets
- IS: 6062-1971, “Method of measurement of flow of water in open channels using standing wave flume-fall”
- USBR (1978), “Design of Small Canal Structures”
- Varshney, R S, Gupta, S C and Gupta, R L (1993) “Theory and design of irrigation structures”, Volume II, Sixth Edition, Nem Chand Publication

Module 4

Hydraulic structures for flow diversion and storage

Lesson
1
Structures for Flow Diversion – Investigation Planning and Layout

Instructional objectives

On completion of this lesson, the student shall learn:

1. The hydraulic structures built to divert water from a river, like a barrage or a weir
2. The different steps to be followed for planning, layout, design and construction of barrages
3. The various aspects of investigation necessary for planning a diversion structure
4. How to choose the location and alignment of a proposed diversion structure
5. How to determine the characteristic dimensions of the different parts of a barrage
6. What are the appurtenant structures that have to be provided with a barrage

4.1.0 Introduction

In order to harness the water potential of a river optimally, it is necessary to construct two types of hydraulic structures, as shown in Figure 1. These are:

1. ***Storage structure***, usually a ***dam***, which acts like a reservoir for storing excess runoff of a river during periods of high flows (as during the monsoons) and releasing it according to a regulated schedule.

2. ***Diversion structure***, which may be a ***weir*** or a ***barrage*** that raises the water level of the river slightly, not for creating storage, but for allowing the water to get diverted through a canal situated at one or either of its banks. Since a diversion structure does not have enough storage, it is called a run-of-the river scheme. The diverted water passed through the canal may be used for irrigation, industry, domestic water needs or power generation.

In this lesson, we shall discuss about the planning, layout and construction aspects of diversion structures, particularly barrages. This is because a weir, which is a raised hump-like structure across the river usually associated with small shutters for flow control (Figure 2a), may be suitable for very small diversion works but for larger rivers with more flexibility on flow control, a barrage (Figure 2b) is desirable. As may be observed from the figures, a barrage is actually a gated form of a weir and the table below lists the relative merits of each of the structure over the other.

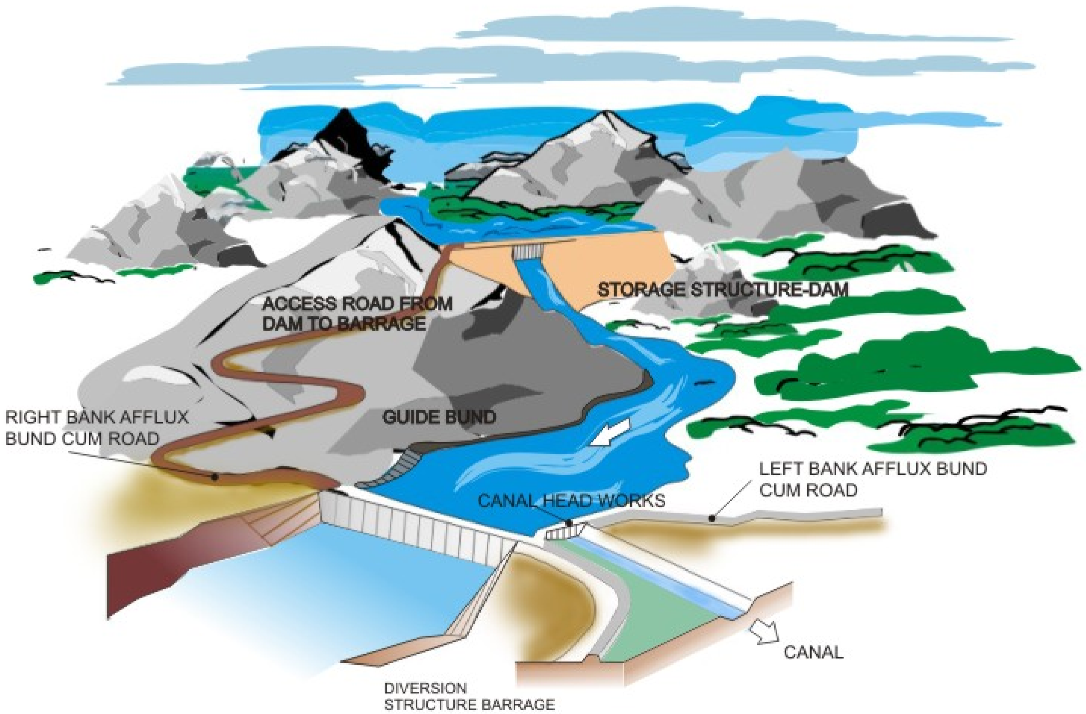


FIGURE 1. Structures for harnessing water resources potential of a river

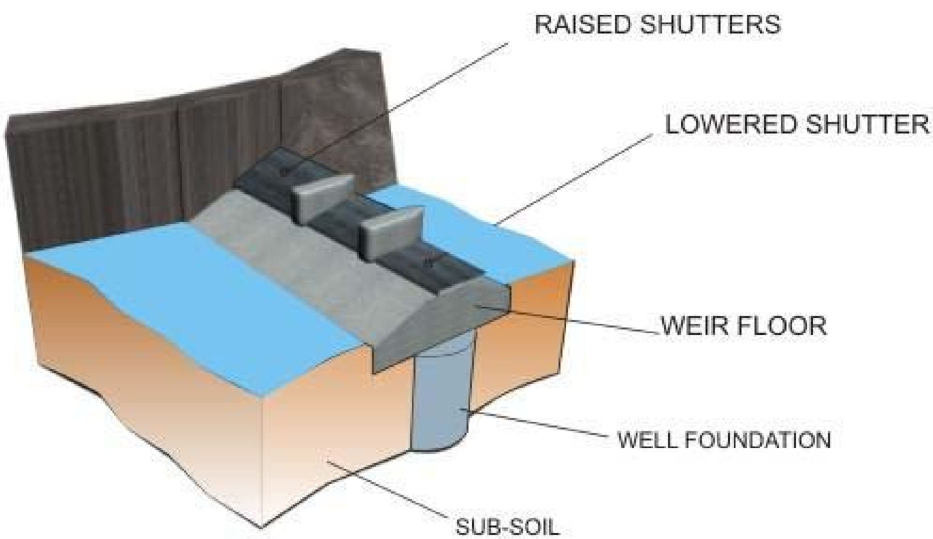


FIGURE 2A. Section through a weir (with falling shutters) showing well foundation.

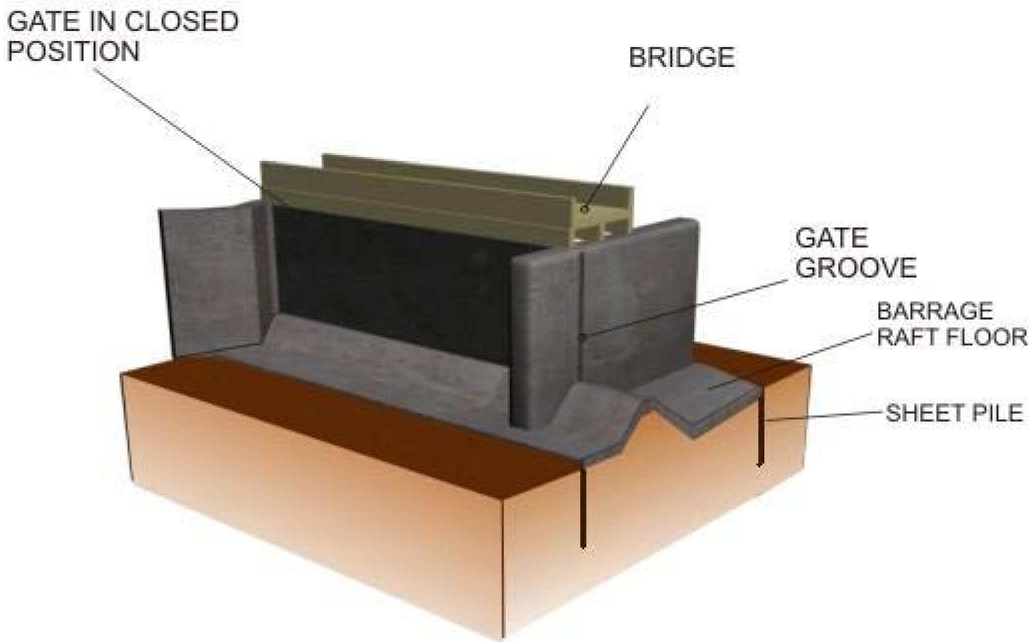


FIGURE 2B. Section through a barrage (with vertical lift gate) showing raft foundation and sheet piles.

Gate hoisting arrangement not shown.

Weir	Barrage
Low cost	High cost
Low control on flow	Relatively high control on flow and water levels by operation of gates
No provision for transport communication across the river	Usually, a road or a rail bridge can be conveniently and economically combined with a barrage wherever necessary
Chances of silting on the upstream is more	Silting may be controlled by judicial operation of gates
Afflux created is high due to relatively high weir crests	Due to low crest of the weirs (the ponding being done mostly by gate operation), the afflux during high floods is low. Since the gates may be lifted up fully, even above the high flood level.

In general, the trend in India for most of the modern water resources project involving diversion of water through a canal involves construction of a barrage, since a slightly more investment can bring in much larger benefits in the long run. Weirs may be used for very small scale hydraulic works.

In the subsequent sections of this lesson, we shall discuss only barrages and interested readers may refer to any standard textbook for details of weirs.

4.1.1 Barrages in different river regimes

A number of barrages have been constructed in this country over the past half a century or so and they may be classified as being located in the following four types of river regimes:

- Mountainous and sub-mountainous
- Alluvial and deltaic

The barrages constructed in these different types of rivers have their own advantages and disadvantages, as discussed below:

The mountainous and sub-mountainous regions are suitable for locating a diversion structure for hydroelectric power schemes due to the availability of high heads and less ***siltation*** problems. However, there could be problems at the head works (intake) of the canal due to possible withdrawal of shingles and arrangements have to be made for the elimination of these. For irrigation canals taking off from the ***head-works***, the service area (where the water would actually be used for irrigation) will start after some distance from the head-works and the idle length of the canal would be more. Further, there would be more number of drainages (hilly streams and torrents) that has to be crossed by the canal as compared to the one in the plains. It is also natural for the canal in the mountainous and sub-mountainous regions to negotiate terrain with relatively larger changes in elevation than the canals passing through alluvial or deltaic stretches of rivers. For ***power canals*** (usually called ***power channels***) the difference in elevations can be effectively utilized by generating hydro-power. In case of irrigation canals, a large number of drops have to be provided. Of course, many irrigation canal drops have been combined with a hydro-electric power generating unit, as shown in Figure 3.

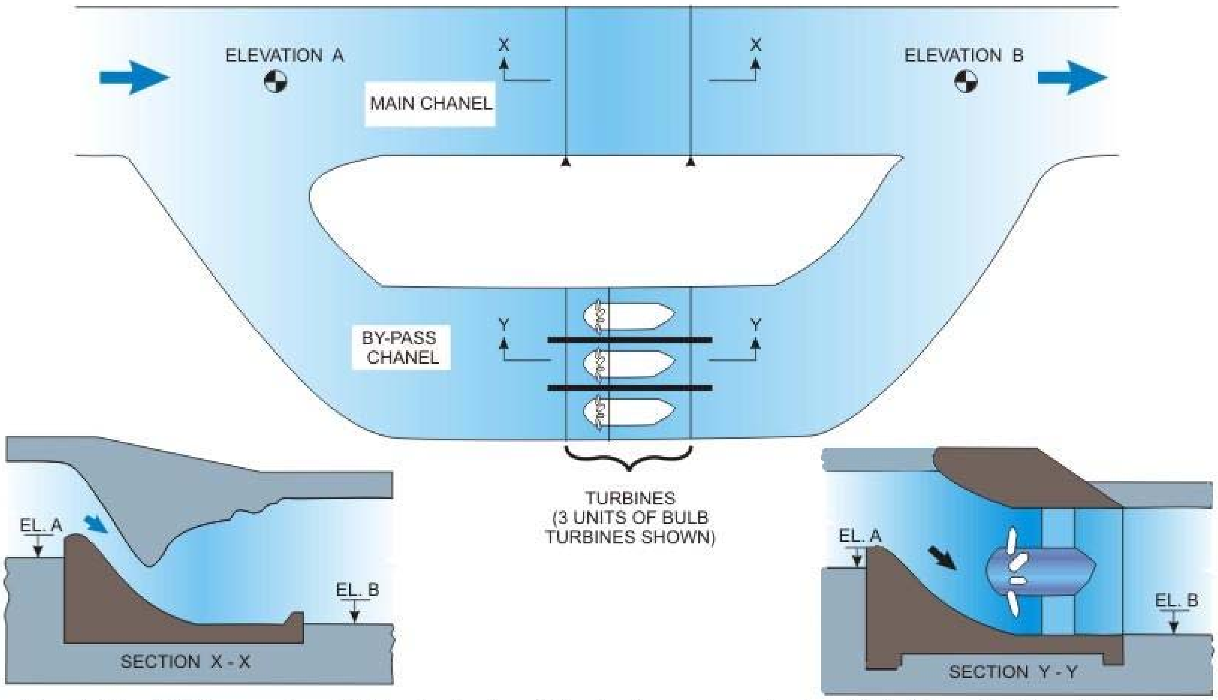


FIGURE 3. Canal Falls bye-pass channel utilising the drop in canal elevation for power generation through bulb-turbines. Flow through the Canal Fall is usually controlled by a gate to allow flow through the turbines.

4.1.2 Steps for planning, layout, design, construction and operation of barrages

It is essential for the successful working of a barrage, or any hydraulic structure for that matter, depends on a proper selection of the location, alignment, layout, design and operation of the structure. Hence, the following aspects have to be carefully looked in to, which have been discussed in detail in the subsequent sections of this lesson:

- •Site investigation and data collection
- •Location and alignment selection of the barrage axis
- •Planning, layout of the barrage and its appurtenant structures
- •Hydraulic designs
- •Structural designs
- •River training works associated with barrages
- •Head regulator for canal intake
- •Instrumentation
- •Construction
- •Maintenance and operations.

4.1.3 Site investigation and data collection

Once it has been decided to establish a barrage for flow diversion from a certain river, proper investigations should be carried out and necessary data should be collected in a systematic way . These aspects are primary to the establishment of a barrage and are necessary to avoid any delay in selecting location, layout and design of the structure. The expenditure in collecting accurate information before designs and construction forms a very small fraction of the total cost of the project, but has a great value in preparing safe and economic designs in a short span of time.

In this respect, the Bureau of Indian Standards Code IS: 7220-1991 “Criteria for investigation, planning and layout for barrages and weir” may be followed, from which the following have been extracted.

Investigations and the corresponding data are generally collected in two stages: ***primary*** and ***detailed.*** The primary investigations include the following, which are used to choose not one, but a couple of alternate sites for the proposed barrage project within some reasonable length along the river. A study of these preliminary data would help to earmark one of the few alternative sites.

Study of available maps and satellite imageries

These maps are generally the survey of India topo-sheets which are published by the agency in a particular format and scale. The survey of a region gets repeated after 30 to 40 years and, hence, it would be wise to collect not only the latest topo-sheet of the project region but also the past surveyed maps which would give an idea of the course of the river in the past. Similarly, the satellite imageries of the river not only in the recent past, but also of as many years back (such imageries are usually available since 1980s) may be collected for studying the physical behaviour of the river like lateral migration, width change, etc.

Regional and site geology

The geology of the project area helps to identify the possibility of a stable foundation of the hydraulic structure, in this case a barrage. Hence the study of this aspect with particular reference to adverse. Geological formations like faults, fractured zones, shear zones, fissures, solution cavities, seismicity, slide zones, etc. should be studied.

Study of foundation strata

Data on the physical characteristics of the riverbed soil or rock from trial pits, trenches and bore holes or from the vertical banks of the rivers should be collected in and around the project region. This data would enable the designer to determine the type of structure necessary at each possible site and hence an economical design may be proposed.

Study of available hydrological data

For correct assessment of the water potential at a certain site, it is essential that the available hydrological data, such as rainfall records in the catchment, river gauges and discharges, peak flows etc. must be studied. Primarily, an assessment of the available 10-daily and monthly runoff and peak flow should be assessed at the location of the river where the barrage is proposed to be built.

Assessment of water needed for diversion

The amount of water that needs to be diverted considering the basic requirement (agricultural, industrial or domestic) and any future increment thereof should be carefully assessed. This would enable the designer to establish well-proportioned canal headworks for intaking water into the main canal and fix the necessary levels on either side of the works required for conveying the required amount of water.

Effect of the barrage on environment and ecology

It is necessary to avoid any adverse effect on the environment, to study the fallout of locating a barrage across the river. Possible erosion of banks and river meandering on the upstream and downstream of the proposed project site an account of construction and operation of the barrage may be investigated.

Limitations on water withdrawal

In most of the rivers in India, the amount of water may not be sufficient at least during some seasons to satisfy all the potential demands. In fact, the demand of the lower river reaches, also called riparian rights, has to be honoured before deciding on the quantity of water that is proposed to be withdrawn. A system of water laws, interstate treaty on sharing of water, etc. already enacted has to be recognized. Further, a careful evaluation is to be made of the human socio-economic factors in the area, their present state, their trends, and to satisfy the corresponding needs and requirements of the society.

Availability of construction material

The construction material that is available readily should be assessed which helps the designer to plan the type of material to be used for constructing the barrage.

Communication to the site of work

While the choice of the final site for locating the structure should be made mainly from considerations of engineering and geology, due consideration should also be given for communication works for easy accessibility and economic transportation of materials to the site of work.

The above considerations furnish the general investigations and data requirement that is needed for selecting the possible sites for the location of the barrage.

Some of the viable alternative sites may be eliminated based on the data of topography, environmental, geology, and foundation, etc. Once a particular site has been chosen from amongst a few plausible options, a detailed investigation is carried out which would help in the hydraulic and structural design aspects of the barrage and its appurtenant structures. If gauge and discharge observations are not available for the site earmarked, it should be immediately started. The following list mentions the detailed investigations that are to be done in order to collect necessary data.

Detailed topographical survey

The survey of India contour maps (often called topo-sheets) are generally drawn to the scale of 1 in 50000 or 1 in 20000 with the contour intervals in the range of 20m in the former and 5m in the latter. Clearly, this accuracy is not enough while designing a hydraulic structure, especially a barrage, whose height itself may be in the range of 5 to 10 meters, and the variation of water level in the pond much less than that. Hence, a detailed survey of the project area may have to be done in scales of at least 1 in 5000 with contours not more than at 0.5m interval. Of course, the contours need not be done above some height, say 2.5 to 3 meters, above the high flood level. The contour plan shall extend up to about 5km on the upstream and downstream of the site and up to an adequate distance on both the flanks up to which the effect of pond is likely to extend.

Apart from the detailed elevation contours, the cross-sections of the river have to be taken at the axis of the barrage at the proposed site and at regular intervals, say 100m, up to about 2km upstream and 1km downstream of the site. The cross sections may be spaced at 5 to 20m apart depending upon the topography of the river. In the deep channel portion of the river, the cross levels may be taken closer.

Hydro-meteorological data

This aspect of data collection is very important for the two entirely different aspects of studies for a river diversion structure. The first is to assess the amount of high flood (called the ***design flood***) that is likely to pass through the barrage for a given probability of occurrence. This would enable the designer to provide sufficient spillway capacity for the barrage. The design flood may be analysed by a study of rainfall records of as many meteorological stations as possible in the vicinity of the site and applying the unit hydrograph analysis. If peak flow data for many years are available, then a flood frequency study may also be made. The second aspect relates to the minimum available discharge (or runoff of the river at project site) that may be diverted. Hence this evolves from a study of the low flows, and estimates of the dependable yield. If the data available is inadequate, a correlation could be established for utilizing the long-term data available for a nearby site of the river.

Sediment concentration data

For planning sediment exclusion devices at the head-works and in the canal system and to evolve a suitable gate regulation for satisfactory sediment passing down the barrage, it is necessary to have data on the sediment load carried by the river for as such period as possible. It is especially required for the flood season when the sediment carried would be more. If the quantity of sediment brought by the river is excessive, the pond levels have to be fixed carefully taking the sediment data into account. This is especially important when the ***pondage*** (the capacity of the pool behind the barrage) is proposed to be provided to meet diurnal power fluctuations also.

Pond survey

The area that is going to be submerged up to normal pond level or within the afflux bunds that shall be acquired, has to be surveyed for working out rehabilitation strategy and compensation amounts. If some forest land is getting submerged, then permission of the Department of Forests and Environment, government of India has to be acquired.

Study of navigation and fish

Data regarding the type of boats and ships passing through the river has to be collected in order to assess the possibility of providing navigation locks (Figure 4). Data regarding the quantity of migratory fish also needs to be collected in order to study the feasibility of providing a fish ladder (Figure 5).

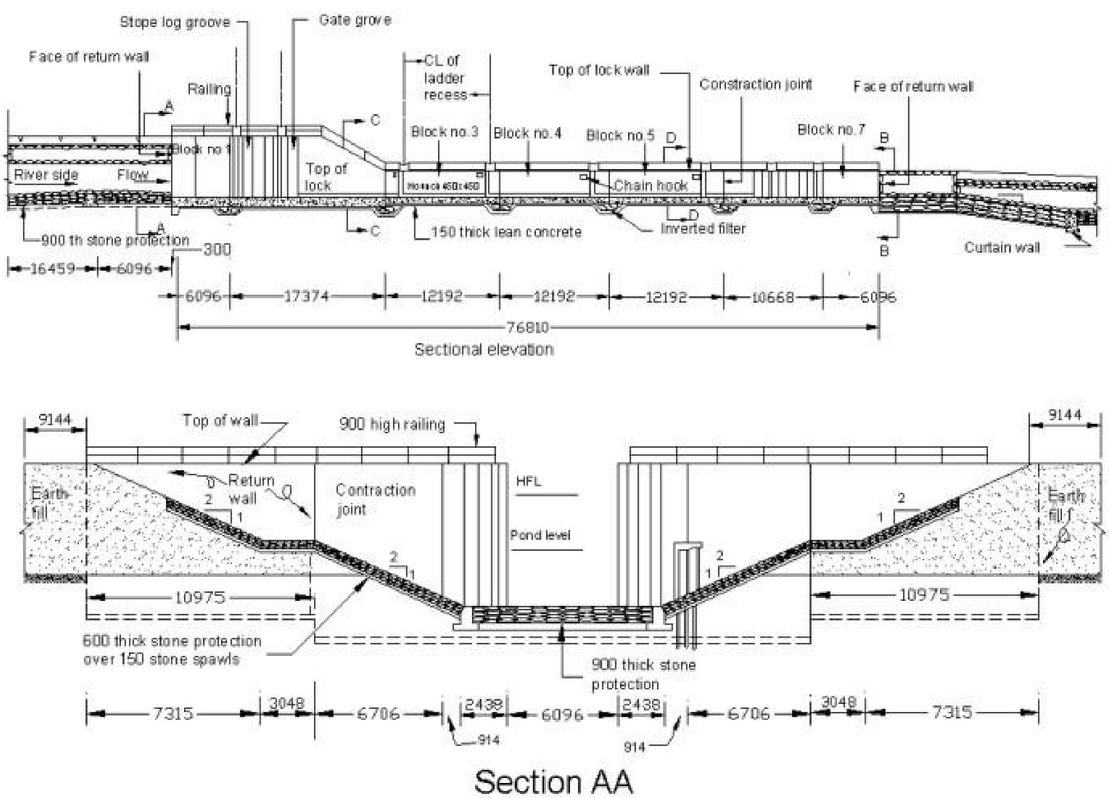


Figure 4. A typical navigation lock : longitudinal section and cross section

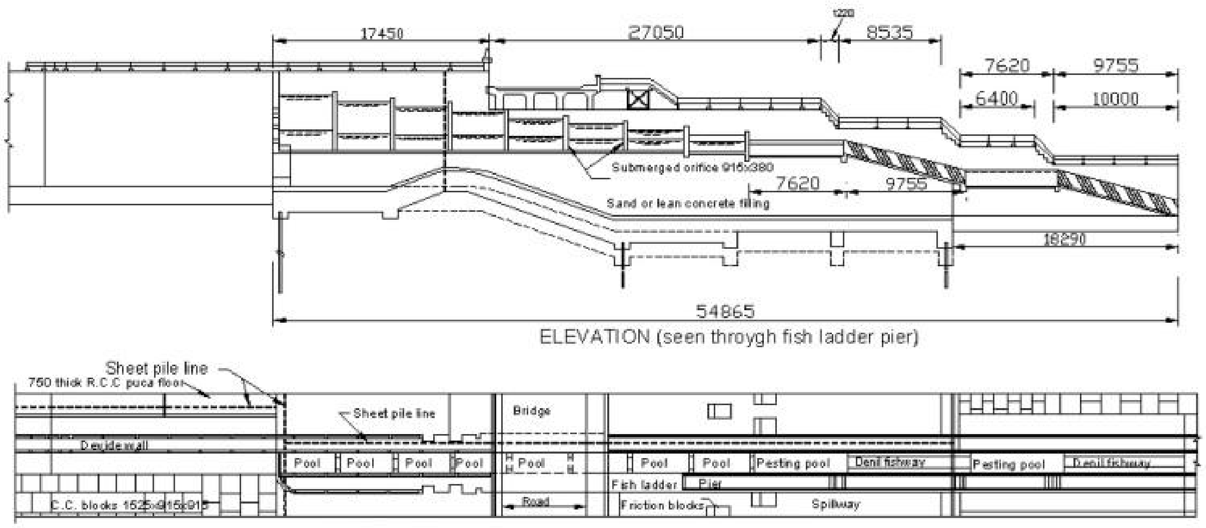


FIGURE 5. A typical fish ladder - section and plan

Study for power generation

Since a barrage causes heading up of water, there is always a possibility of utilizing the difference in water levels between the upstream pool and the downstream river level to generate hydropower. The difference in water levels is higher during the non-monsoon periods, when the total river flow is less and, consequently, the water level of the natural river downstream is quite low but the gates of the barrage help to keep the pool level high. Bulb turbines (as shown for canal falls power house, Figure 3) which can utilise head difference between one to fifteen meters can be installed in some bays of the barrage to generate power. Theoretical investigation for minimum available 10-daily flows in 50%, 75% and 90% of the year and the normal differential head available in different months have to be studied to assess the power generating potential of a barrage power house.

Study for provision of a rail or a road bridge across the barrage

The requirement of connectivity between the two sides of the river at the point where a barrage is being proposed to be built may lead to decisions regarding provisions of a rail or a road bridge across the barrage. The volume of rail or road traffic would help to determine single or double lanes for the respective modes of transport.

4.1.4 Location and alignment selection

The location for a barrage should be decided on considerations of suitability for the main structure and its appurtenant works, like silt removing devices and intake for canals (also called canal head regulators). An ideal location would be that which satisfies the requirements of all the three components. Some of the points that have to be kept in mind in selecting an appropriate location for a barrage are as follows:

- The canal head regulators (or head-works, as they are called) intending to divert water to a canal for irrigation has to be planned such that full command may be achieved by a barrage or weir of reasonable height. The combined cost of construction of head-works and that of the canal from the barrage up to the point where the water is first used for irrigation should be small.

- Sometimes, a favourable location for fixing the site for a barrage and canal head-works may have to be abandoned due to large quantities of rock excavation required.

- The river reach at the proposed location should be straight, as far as possible, so that velocities may be uniform and the sectional area of the river fairly constant. The banks should preferably be high, well defined and non-erodible. This will ensure a more or less straight flow to the barrage from the upstream. If such a site is available, it may need very small or practically no guide bunds. In case of high banks, the country side will not be submerged during high floods and a considerable saving in the cost of flood protection embankment may be effected.

- For barrages to be located in alluvial river reaches with meandering tendencies, the nodal points have to be ascertained. ***Nodal point*** is the portion of a meandering river which is more or less fixed in space (Figure 6). A nodal point may be decided by superimposing the survey maps or corresponding satellite imageries of the river for as many years as possible.

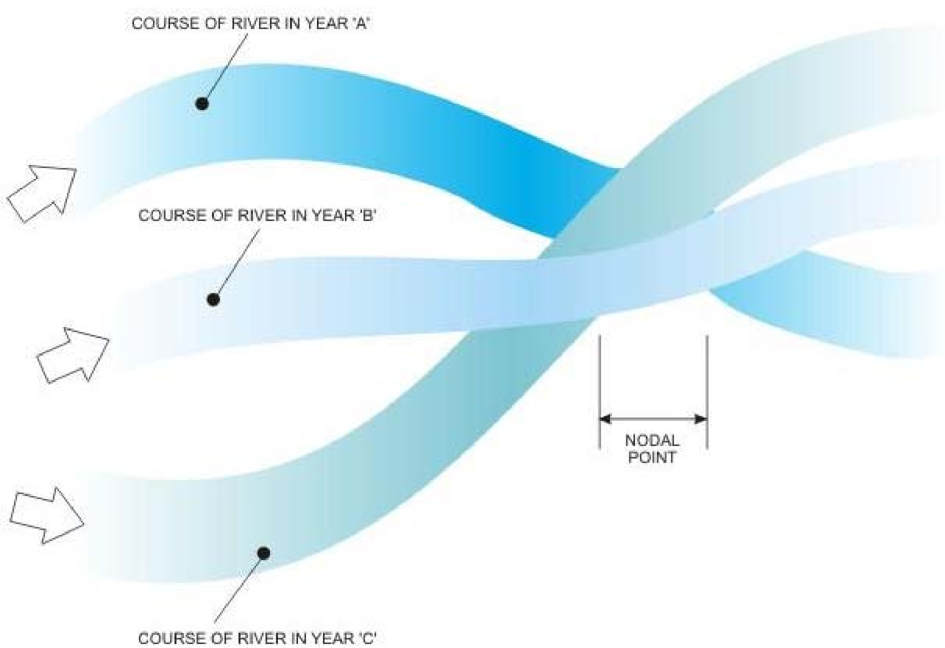


FIGURE 6. Definition of a nodal point for a meandering river

- For locating a barrage in a curve of a river, the off-taking canal may be located in the downstream end of the concave bank, which would help in drawing less sediment in to the canal. If it is a necessity to locate an off-taking canal on the convex bank (as may be required for irrigating an area on this side of the river), then proper silt excluding devices have to be designed since a convex bank of a curved river is prone to sediment accumulation.

4.1.5 Planning and layout

A barrage, by definition, is a weir structure fitted with gates to regulate the water level in the pool behind in order to divert water through a canal meant for irrigation, power generation, flow augmentation to another river, etc. By following the general guidelines mentioned in section 4.14, the location and alignment of the barrage axis and that of the canal headworks may be decided but the other details, like the width of the barrage and headworks, levels of weir crests, lengths of weir floors, river training works, pond level etc. have to be finalized based on the hydraulic conditions and geologic characteristics of the river bed and banks of the site. This section is devoted to these planning and layout concepts of a barrage project consisting of the main structure and its appurtenant works.

The planning part decides the various parameters necessary for designing the structures. Further, planning is also necessary for chalking out a construction program. The major planning aspects are as follows:

Design flood

The diversion structure has to be designed in such a way that it may be able to pass a high flood of sufficient magnitude (called the design flood) safely. It is assumed that when the design flood passes the structure all the gates of the structure are fully open and it acts like a weir across the river with only the obstruction of the piers between the abutments. The abutments are the end walls at two extremes of the structure and the length in between the two is termed as the waterway. Naturally, a high design flood would necessitate a longer waterway. In general, a design flood of 1 in 50 years frequency has been recommended for design of all items except free board for which a minimum of 500 year frequency flood or the Standard Project Flood has been recommended as per Bureau of Indian Standard Code IS: 6966 (Part1) - 1989 “Hydraulic Design of Barrages and Weirs – guidelines”, some of the barrages built in the past have considered very high design floods, as may be seen from the data given below:

Barrage across river	Design flood frequency
Gondak	1in 220 years
Godawari	1in 200 years
Kosi	1 in 600 years
Sone	1 in 70 years

Though a high design flood may ensure safety of the structure against large floods, there is a consequent adverse affect related to sediment deposition in the pool. This results from the fact that since a design flood is expected to pass once in that many years, with a full gate opening, the intervening years having lesser magnitude of floods would see the gates of the barrage being operated to raise the pond level at the desired elevation. Naturally, this would result in a slower velocity in the pool and a consequent deposition of suspended sediments. If sediment deposition continues for many consecutive years, they tend to form large mounds, called shoals, within the pool, not far upstream from the barrage bays. This phenomena, which has been noticed in many of the large barrages of India, like Farakka, Mahanadi, etc., can cause not only reduction in the pool volume but more importantly, may cause obstruction to the free flow of the river that is approaching the barrage. This results in what is called the washing of bays, with the flow through the bays directly downstream of the shoals being reduced and the excess flow passed through the other bays. As a result, it causes inclination of the approaching flow to the barrage which may cause other undesired phenomena. It has been observed that barrages with large shoal formations just upstream have flow inclinations to the extent of 60^0 or more to a normal through the barrage axis.

Afflux

If the flood in the river is less than the design flood, then some of the gates would be fully opened but the remaining opened to such an extent which would permit the maintaining of the pond level. However, when a design flood or a higher discharge through the barrage structure, all the gates have to be opened. Nevertheless, the structure would cause a rise in the water level on the upstream compared to level in the downstream at the time of passage of a high flood (equal to or more than the design flood) with all the gates open. This rise in water level on the upstream is called afflux. The amount of afflux will determine the top levels of the guide bunds and marginal bunds, piers, flank walls etc. Naturally a smaller waterway would result in larger afflux and vice versa. Hence, reduction in water way may cause in lowering the cost of the barrage structure but may result in higher afflux and a resulting larger height of bunds and piers.

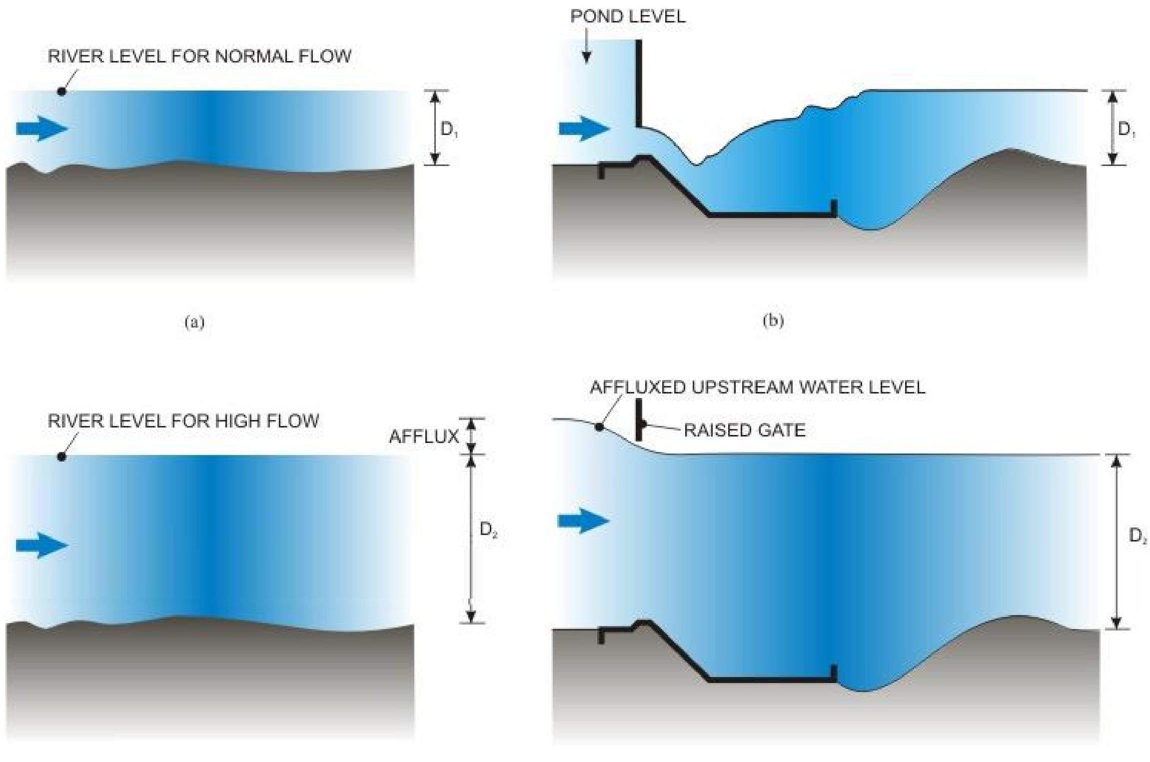


FIGURE 7. Afflux explained. (a) River at normal flow:Depth almost same as on the downstream of a ponded flow as in (b). (c) River at flood level: Downstream depth almost the same as that of the original river ; upstream depth higher than downstream depth, the difference being called Afflux as in (d).

Free Board

Once the permissible afflux is decided, the necessary water way can be accordingly worked out and the upstream water level estimated for the design flood. Over the gauge-discharge curve on the downstream side and estimated on the upstream, sufficient Free Board has to be provided so that there is no overtopping of the components like abutments, piers, flank walls, guide bunds, afflux bunds etc. The Free Board to be provided depends on the importance of the structure generally, 1.5 to 2 m Free Board above the affluxed water level on the upstream and above the high flood level on the downstream is provided. A freeboard is provided over an affluxed water level due to a flood with 1 in 500 year frequency.

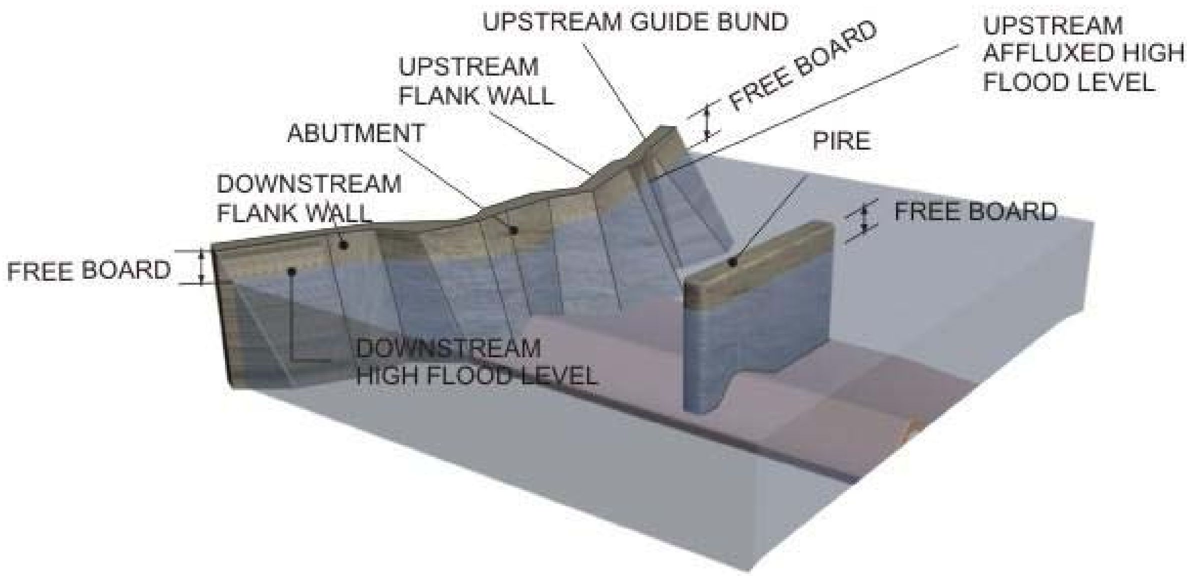


FIGURE 8. Free board has to be kept above the respective high flood levels, on the upstream considering afflux and on the downstream without afflux

Pond Level

Pond level is the level of water, immediately upstream of the barrage, which is required to facilitate withdrawal of water into the canal with its full supply. The pond level has to be carefully planned so that the required water can be drawn without difficulty. By adding the energy losses through the head regulator to the Full Supply Level of the canal at its starting point just downstream of the canal head-works, the pond level is evaluated.

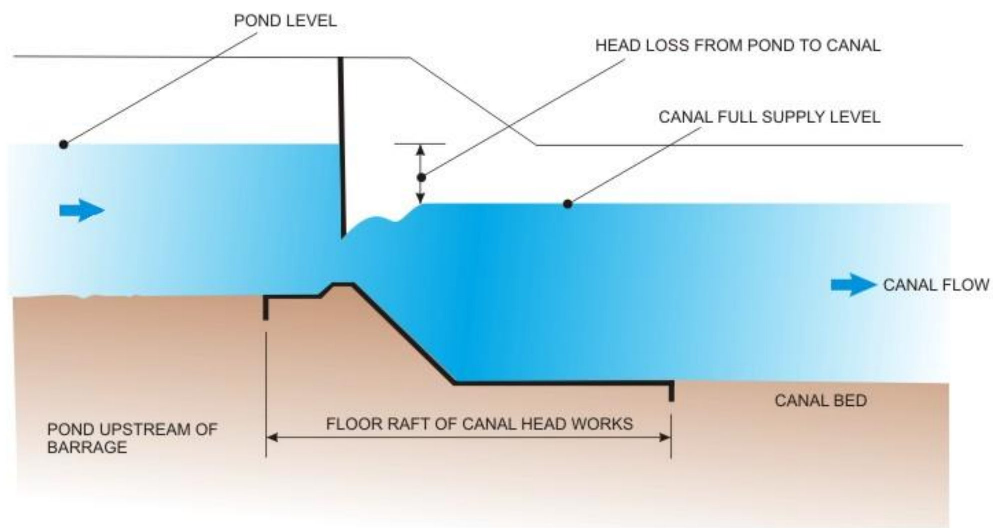


FIGURE 9. Section through a canal head regulator. Pond level should be equal to the canal full supply level added to the head loss through the head works while delivering the canal flow

The provision of a high pond level with an elevation almost equal to the high flood level or above has to be planned very carefully since such a provision is likely to induce shoal formation on the upstream. This has happened in the Durgapur Barrage on river Damodar.

Waterway

As discussed earlier, waterway, or the clear opening of a barrage to allow flood flow to pass has a bearing on the afflux. Hence, a maximum limit placed on the afflux also limits the minimum waterway. Many a times, the Lacey's stable perimeter for the highest flood discharge is taken as the basis of calculating the waterway. However, it should be remembered that Lacey's formula is based on studies of canals in the alluvial regime and may not be quite correct for large rivers, and also for rivers in boulder or clayey reaches. Nevertheless, application of the Lacey's waterway would require the following calculations as given in Bureau of Indian standard Code IS: 6966-1989 "Guidelines for hydraulic design of barrages and weirs: Part 1 Alluvial Reaches".

$$\boldsymbol{P} = 4.83\, \boldsymbol{Q}^{1/2} \qquad (1)$$

Where Q is the design flood discharge in m^3/s for the 50 year frequency flood. In the case of rivers in bouldery reaches, the width available at the site is limited by the firm banks. For meandering rivers in alluvial reaches, a factor is usually multiplied with the perimeter obtained by Lacey's formula, which is called the looseness factor, as given below

Silt Factor, ***f***	Looseness Factor
<1.0	1.2 to 1.0
1.0 to 1.5	1.0 to 0.6

Silt factor f is calculated by knowing the average particle size d_{50} is in mm of the soil from the following relationship

$$f = 1.76\,(d_{50})^{1/2} \qquad (2)$$

By limiting the waterway, and consequently increasing the velocity and discharge per unit width, the shoal formation in the pond upstream of the barrage can possibly be minimized. However, it has an adverse effect also since increase in the intensity of discharge, requires longer solid apron and deeper sheet piles due to higher expected scour depths. Nevertheless, the performance of many barrages has led to the general observation that high looseness factor, more than about 1.0, results in shoal formation in the upstream pool. Hence many recent barrages have been designed with a low looseness factor, nearing 0.5. However, there is a need for a systematic study to evolve a scientific analysis for evaluating the waterway.

A restricted waterway for a barrage is obtained by the use of guide bunds, approach and afflux embankments in Figure 10.

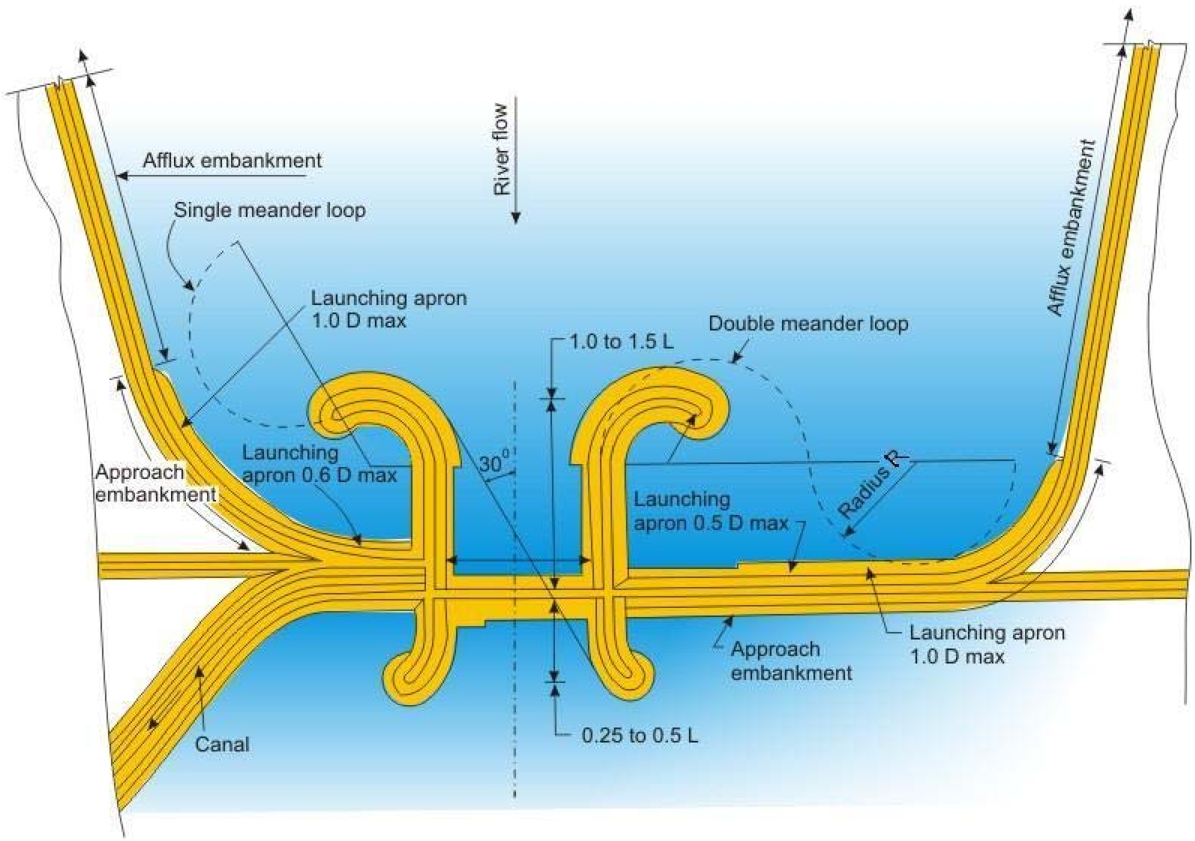


FIGURE 10 . Typical layout of river training works for restricting waterway of barrage

A brief discussion of the above works, called river training works, is given in the following section.

River training works

The river training works for barrages are required to achieve the following;

1. Prevent out flanking of the structure
2. Minimize cross flows through the barrage
3. prevent flooding by the river lands upstream
4. provide favourable curvature of flow at the head regulator from the point of sediment entry into the canal, and
5. guide the river to flow axially through the barrage or weir

As was seen in the section on waterway, it is necessary at many instances to narrow down and restrict the course of the river through the barrage and it is achieved by the use of the river training works. Proper alignment of guide bunds is essential to ensure satisfactory flow conditions in the vicinity of the barrage. In case of wide alluvial banks, the length and curvature at the head of the guide bunds should be kept such that the worst meander loop is kept away from either the canal embankment or the approach embankment. If the alluvial bank is close to the barrage, the guide bunds may be connected to it by providing suitable curvature, if necessary. If there is any out-crop of hard strata on the banks, it is advisable to tie the guide bunds to such control points. Two typical guide bund layouts are shown in Figure 11.

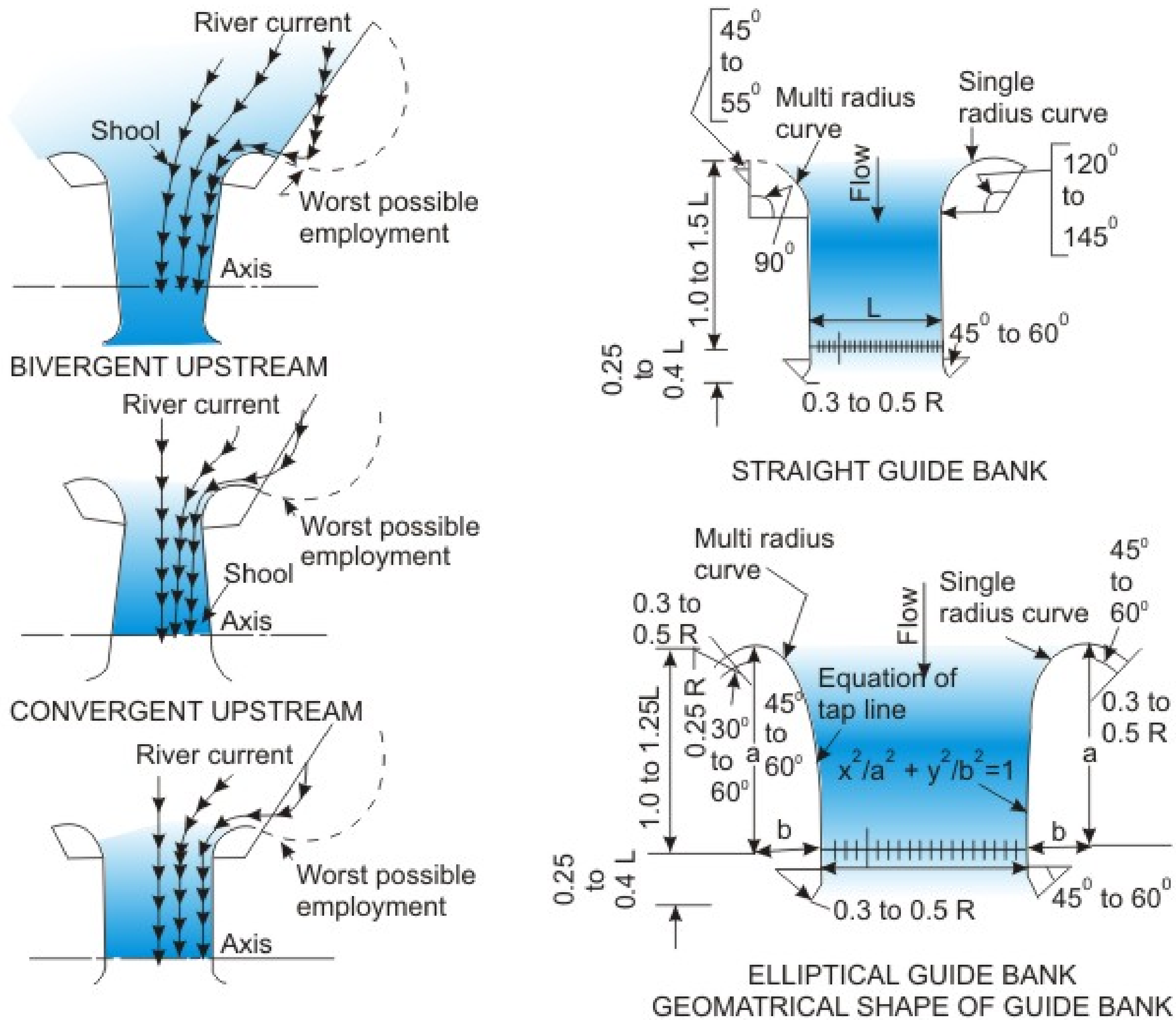


FIGURE 11. Typical layouts of guide banks

The dimensions given in Figure 11 are preliminary, and model studies have to be carried out for fixation of final sizes for any particular project depending upon the prevalent site conditions.

Crest levels of spillway and undersluice bays

The bays of a barrage are in the shape of weirs or spillways and the crest levels of these have to be decided correctly. Some of the bays towards the canal end of the barrage are provided with lower crest (Figure 12) in order to:

- Maintain a clear and well defined river channel towards the canal head regulator
- To enable the canal to draw silt free water from surface only as much as possible
- To scour the silt deposited in front of the head regulator

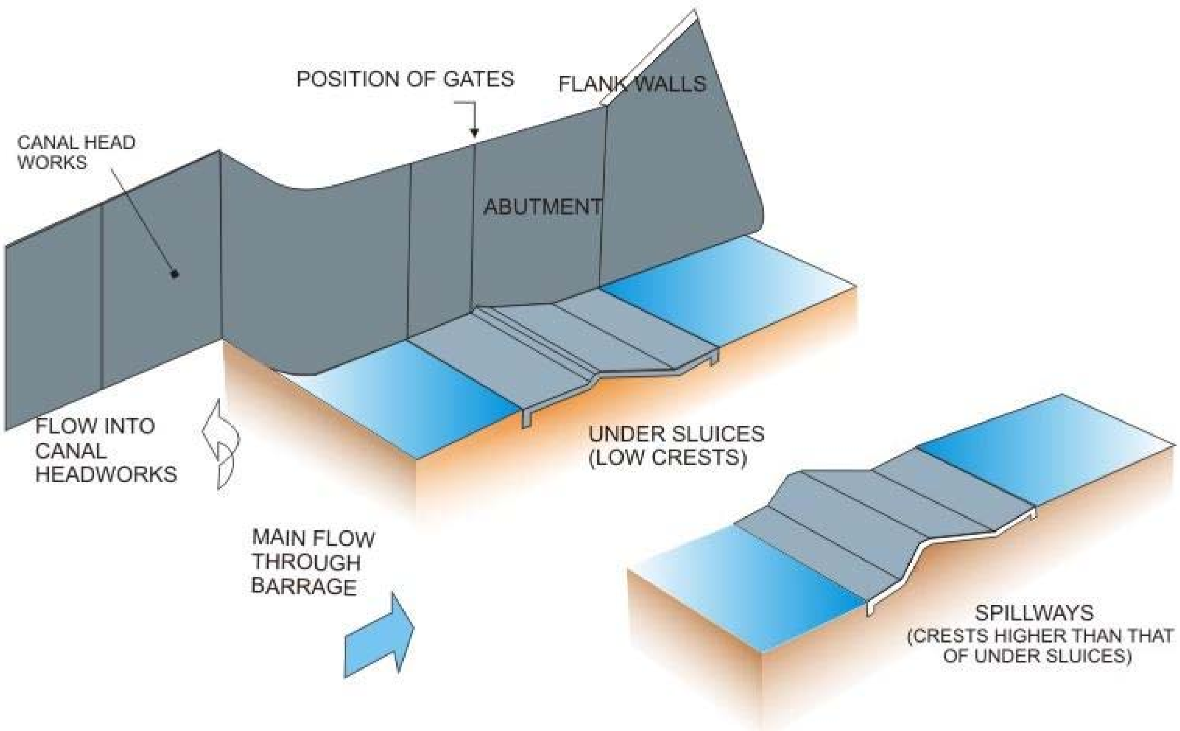


FIGURE 12. Relative shapes of under sluice bays (towards the canal end) and spillway bays of a barrage. Piers, gates and walls have not been shown.

The set of undersluice bays withlow crest elevations are separated from the set of spillway bays witha small weir hump by a long wall, called the ***divide wall***.

The layout of a barrage and its appurtenant structures can be seen from a typical plan view shown in Figure 13. The important components of a barrage are discussed below:

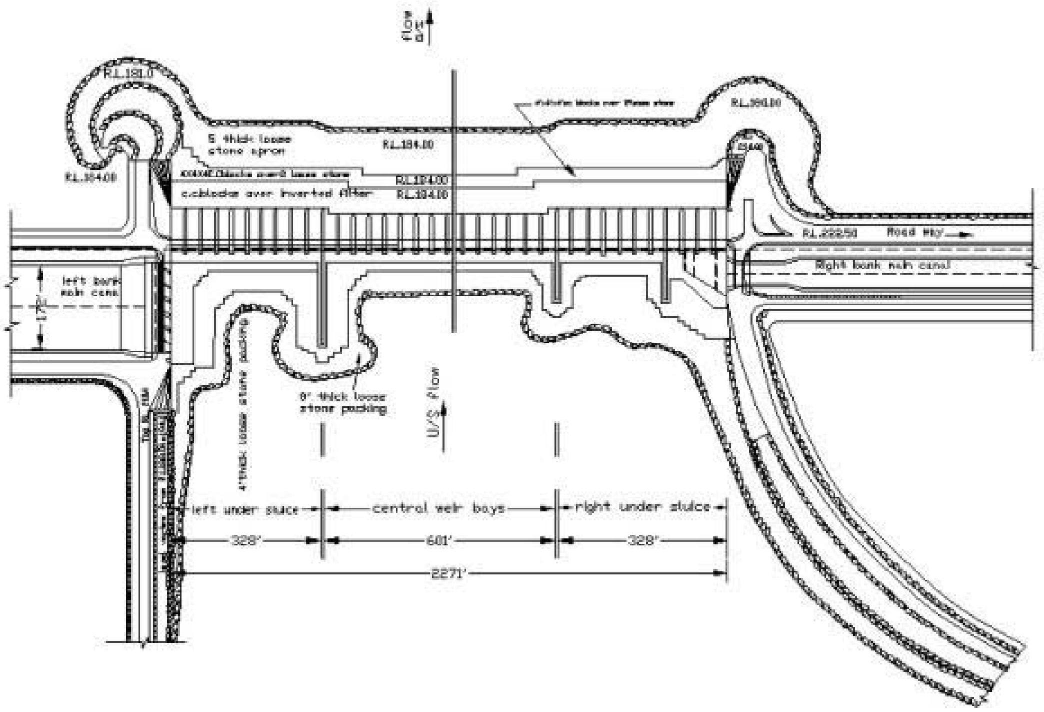


FIGURE 13. A typical layout of a barrage and its appurtenant structure.
This example shows canal off-taking from either end

Spillway bays

This is the main body of the barrage for controlling the discharges and to raise the water level to the desired value to feed the canals. It is a reinforced concrete structure designed as a raft foundation supporting the weight of the gates, piers and the bridge above to prevent sinking into the sandy river bed foundation. A typical section of a spillway bay is shown in Figure 14.

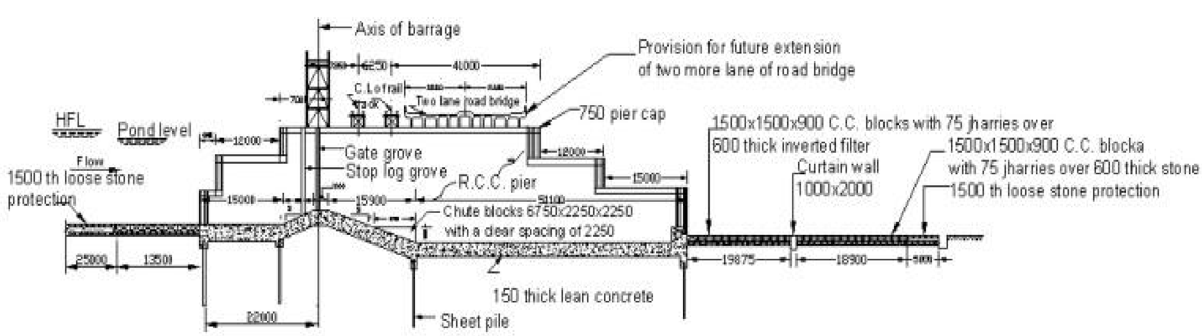


FIGURE 14. Typical section of spillway of a barrage

Undersluice bays

These low crested bays may be provided on only one flank or on bothflanks of the river depending upon whether canals are taking-off from one or both sides. The width of the undersluice portion is determined on the basis of the following considerations.

- It should be capable of passing at least double the canal discharge to ensure good scouring capacity
- It should be capable of passing about 10 to 20 percent of the maximum flood discharge at high floods
- It should be wide enough to keep the approach velocities sufficiently lower than critical velocities to ensure maximum settling of suspended silt load.

Undersluices are often integrated withRCC tunnels or barrels, called ***silt excluders***, extending up to the widthof the Canal Head Regulator, as can be seen from Figure 13. These tunnels are provided in order to carry the heavier silt from a distance upstream and discharge it on the downstream, allowing relatively clear water to flow above from which the Canal Head Regulator draw its share of water.

Typical sections of undersluices with and without silt excluder tunnel are shown in Figures 15 and 16.

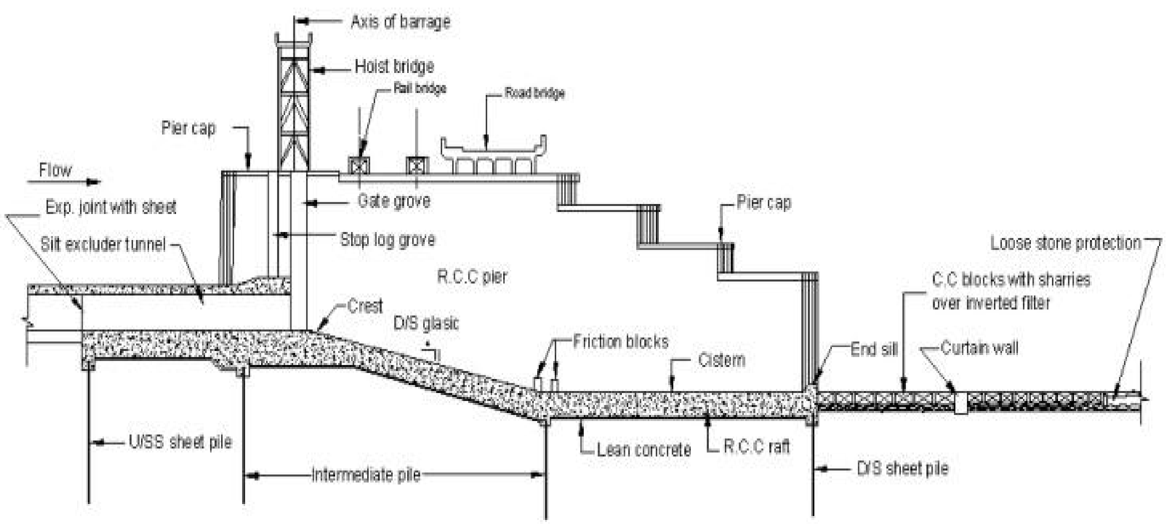


FIGURE 15. Typical section of undersluice with silt excluder tunnels

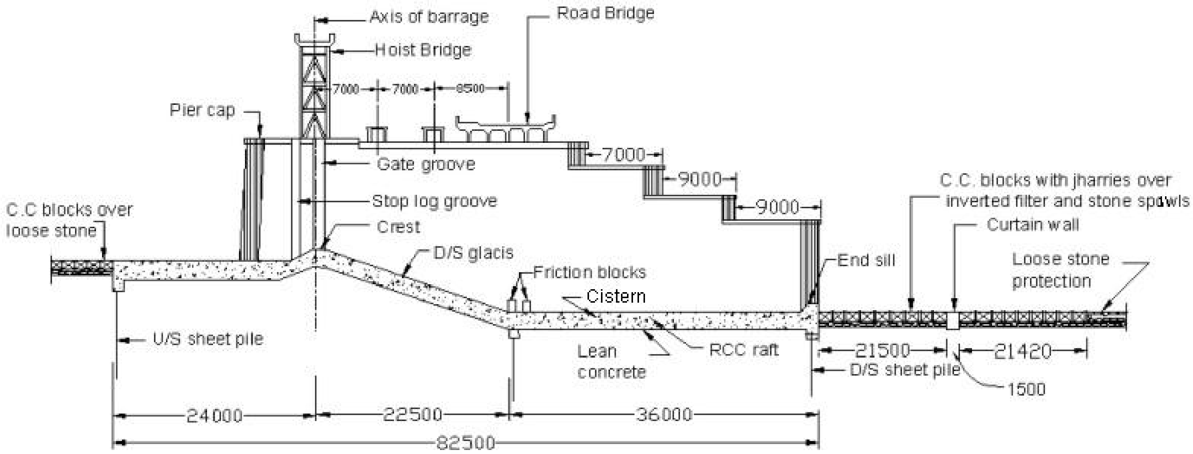


FIGURE 16. Typical section of undesluice without silt excluding tunnels

River –sluice bays

River sluices are a set of sluices similar to the undersluices located in between the undersluice and spiilway bays and separated from them by means of divide walls. These are generally provided in long barrages (that is, in wide rivers) for simplifying the operation of gates during normal floods and to have better control on the river. The section of river-sluice bays would generally be similar to that of undersliuce bays without silt excluding tunnels.

Cut-off

Cut-offs are barriers provided below the floor of the barrage both at the upstream and the downstream ends. They may be in the form of concrete lungs or steel sheet-piles, as observed from the figures 14, 15 and 16. The cut-offs extend from one end of the barrage up to the other end (on the other bank). The purpose of providing cutoff is two-folds as explained below.

During low-flow periods in rivers, when most of the gates are closed in order to maintain a pond level, the differential pressure head between upstream and downstream may cause uplift of river bed particles (Figure 17a). A cutoff increases the flow path and reduces the uplift pressure, ensuring stability to the structure (Figure 17b).

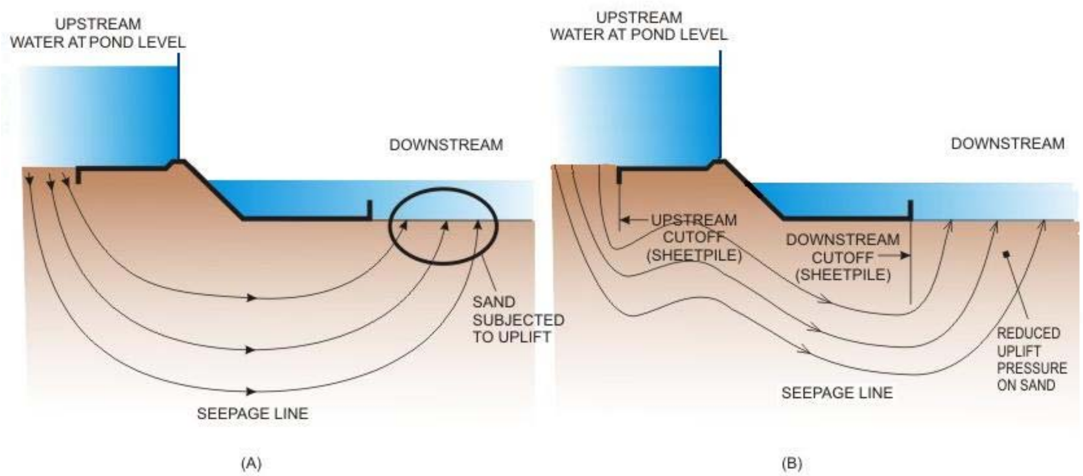


FIGURE 17. Downstream riverbed uplift and its prevention. (A) Differential pressure head causing sand uplift ; (B) Cutoffs to reduce uplift pressure by increasing the seepage path.

During flood flows or some unnatural flow condition, when there is substantial scour of the downstream riverbed, the cutoffs or sheet piles protect the undermining of the structures foundation (Figure 18).

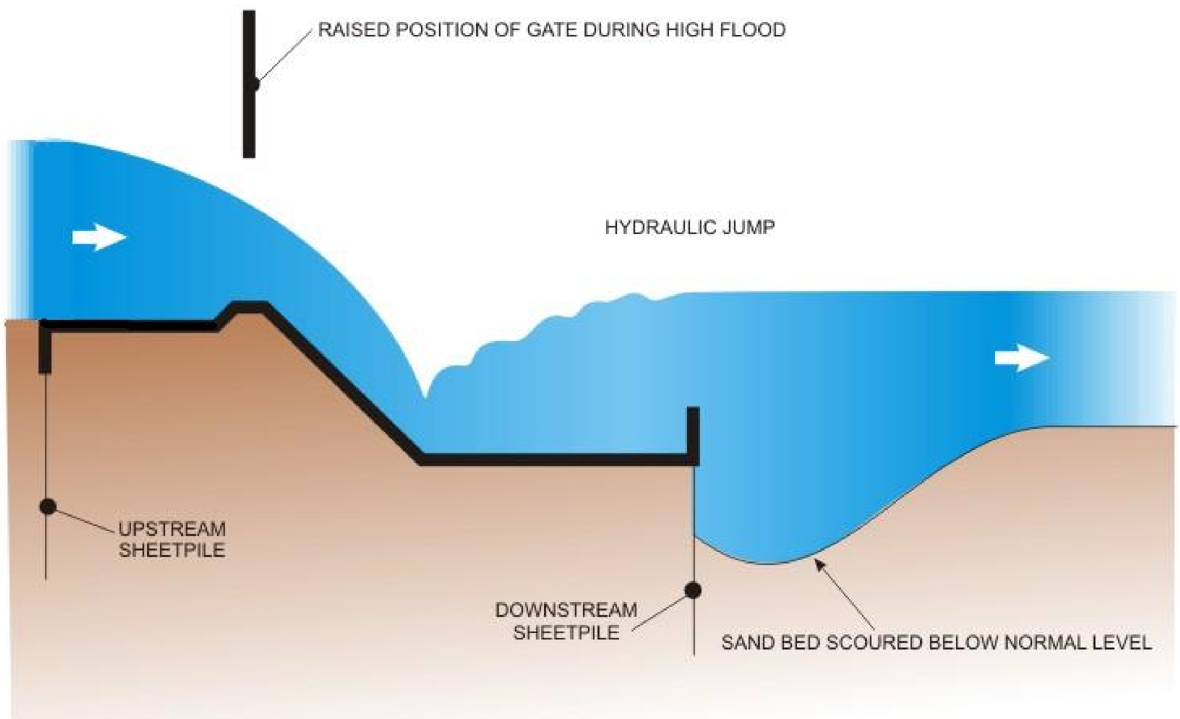


FIGURE 18. Riverbed scour resisted by sheet pile protects the foundation barrage floor.

Pier

Piers are provided between each bay. The gates operate through the groove provided in the piers. Usually, there are two sets of grooves, the upstream being called the Stop Log groove and the downstream one being called the Main Gate or Service gate groove. The piers are constructed usually monolithic with the floor (raft), and extend usually from the upstream end to the downstream end solid floor of the spillway (or under sluice/river sluices), as may be observed from Figure 2B. The piers have to be high enough to hold the gates clear off the maximum flood level while making ample allowance for passing any floating debris under the raised gates.

Divide wall

The divide wall is much like a pier and is provided between the sets of undersluice or river sluice or spill bays. The main functions of a divide wall:

- It separates the turbulent flood waters from the pocket in front of the canal head.
- It helps in checking parallel flow (to the axis of the barrage) which would be caused by the formation of deep channels leading from the river to the pocket in front of the sluices.

The length of the divide wall on the upstream has to be such as to keep the heavy action on the nose of the divide wall away from the upstream protection of the sluices and also to provide a deep still water pond in front of the canal head regulator.

A typical section of a divide wall is shown in Figure 19.

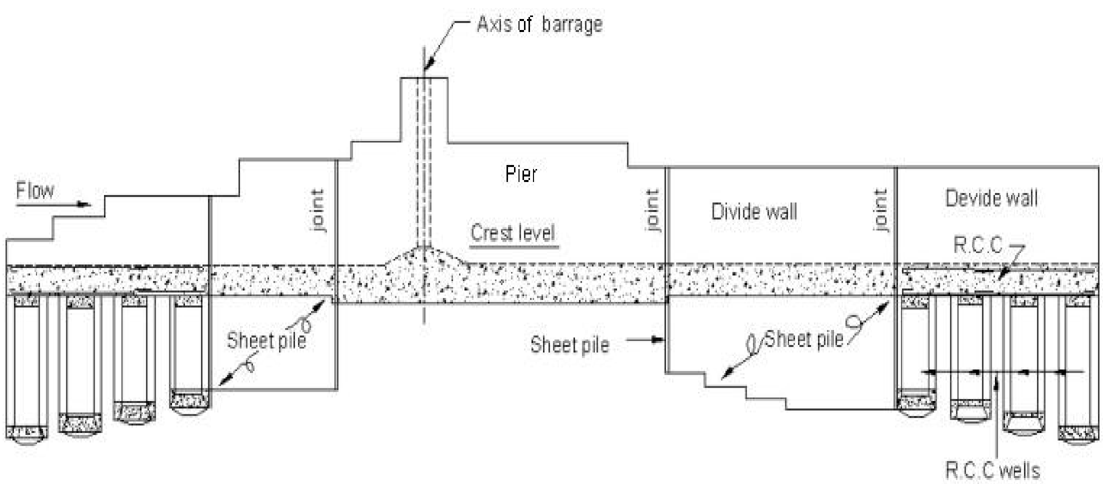


FIGURE 19. **Typical section through a divide wall**

Abutment

The abutments form the end structures of the barrage and their layout depends upon the project features and topography of the site. The lengthof the abutment is generally kept same as the lengthof the floor. The top of the abutment is fixed withadequate free board over the upstream and downstream water levels.

Flank wall

In continuation of the abutments of the diversion structure, flank walls are provided bothon the upstream and downstream sides on boththe banks. The flank walls ensure smoothentry and exit of water and away from the diversion structure. The flank walls laid out in a flare withvertical alignment close to the abutment and a slope of 2H:1V or 3H:1V on the other end, as may be observed from the layout of the barrage shown in Figure 13.

Return wall

Return walls are generally provided at right angles to the abutment either at its end or at the flank wall portion, and extends into the banks to hold the bank or back-filling earth in place.

Guide bunds

The requirement of narrowing down and restricting wide alluvial river courses to flow axially through the barrage necessitates the use of guide bunds, as shown in Figure 10.

Afflux bunds

Afflux bunds are components of the diversion structures wherever necessary to protect important low lying properties adjacent to the structures from submergence due to affluxed high floods.

Silt excluding devices

As shown in the layout of a barrage (Figure 13), the silt excluding tunnels carry heavy silt down the river below the undersluices.

Navigation Lock

Since inland or river water navigation is economically more attractive for larger cargo, navigation facilities can be combined withthe barrage projects. This includes the provision of a navigation channel withnavigation locks suitably incorporated to allow passage of crafts to move from upstream to downstream and vice versa (Figure 4).

Fish pass

Some barrages require providing special structures to allow migratory fishes to flow up and down the river through structures called Fish Passes or Fish Locks (Figure 5).

Canal Head Regulator

The water that enters a canal is regulated through a Head Regulator. A typical cross section through a regulator is shown in Figure 9. As it is desirable to exclude silt as much as possible from the head regulator, the axis of the head regulator is laid out at an angle from 90^0 to 110^0 to the barrage axis as recommended in Bureau of Indian Standards code IS : 6531(1972) "Criteria for design of canal head regulators". A typical layout of a head regulator is shown in Figure 20.

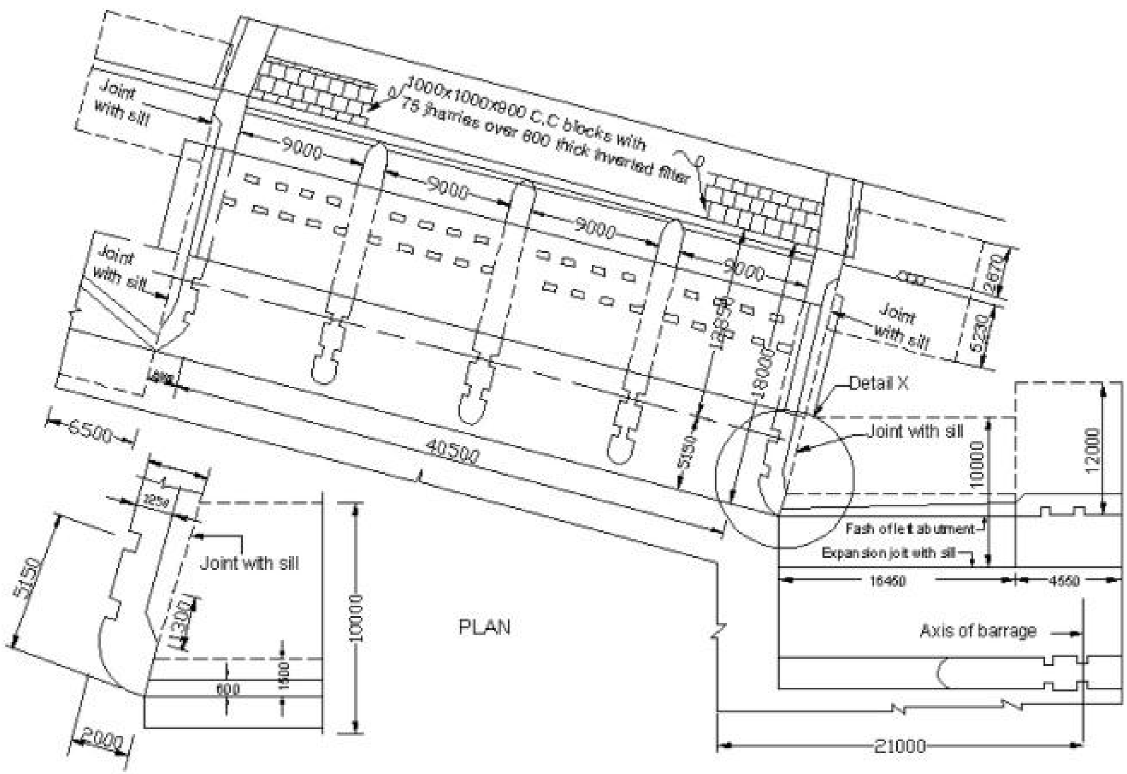


FIGURE 20. Typical layout of a canal head regulator

4.1.6 Finalisation of barrage layout through model studies

It may be realised that a being a structure spanning across a river, may cause enormous changes to the river hydraulics and morphology. Much of this is dynamic, since the floods every year would generally be of different magnitude or duration and accordingly the gate operation would be different each time. Hence the planning and layout decided from the general principles discussed earlier in this lesson may only be taken as a guideline. The final position, location, layout, alignment of each component of the structure and in relation to each other has to be done through model studies. The Bureau of Indian Standard code IS 14955: 2001 "Guidelines for hydraulic model studies of barrages and weirs" lays down the basic principles of model studies and could be followed to finalise the layout of a particular barrage project.

Module 4

Hydraulic Structures for Flow Diversion and Storage

Lesson
2
Design of the Main Diversion Structure of a Barrage

Instructional objectives

On completion of this lesson, the student shall learn:

1. The sub-surface consideration for the design of a barrage
2. Steps for computing seepage pressure and exit gradient for barrage
3. Procedure to determine the waterway of a barrage
4. Computation step for determining barrage cistern level
5. The different protection works for barrages

4.2.0 Introduction

The design of any hydraulic structure comprises of two steps:

- Hydraulic design, to fix the overall dimensions and profiles of the structure, and
- Structural design, where the various sections are analysed for stresses under different loads and reinforcement or other structural details are worked out.

The dimensions fixed by hydraulic design through available empirical formulae are further refined by testing a scale model of the structure in a hydraulic model testing laboratory. The structural design uses the hydraulic forces expected from the given hydraulic parameters (evolved through the hydraulic design) and produces a detailing that will keep the structure safe against those forces and loadings. In this lesson, the hydraulics of barrages and canal head regulator is presented. The structural design is discussed in Lesson 4.2.

For barrages, there are two different sets of hydraulic conditions. The first is due to sub-surface or seepage flow conditions that occurs due to a water level difference on the upstream and down stream of a barrage and is the maximum when the gates of a barrage are mostly closed as during the low flow period of the river. The other is due to surface flow conditions which occur while the barrage gates are open during floods. In this section we shall discuss each of these hydraulic conditions for the main diversion structure of a barrage to evaluate the forces generated by them. The hydraulic conditions of canal head regulators are also quite similar, with seepage flow dominating during gate closed condition and free flow during gate open condition.

4.2.1 Hydraulic design for sub-surface flow

The sub-surface flow below a barrage causes two definite instability problems, as listed below and illustrated in Figure 1.

1. Uplift forces due to the sub soil pressure that tends to lift up the barrage raft floor, and

2. Upward rising seepage forces through the river bed just down stream of the solid apron causes sand particles to erupt upwards and tends to 'piping' failure of the foundation.

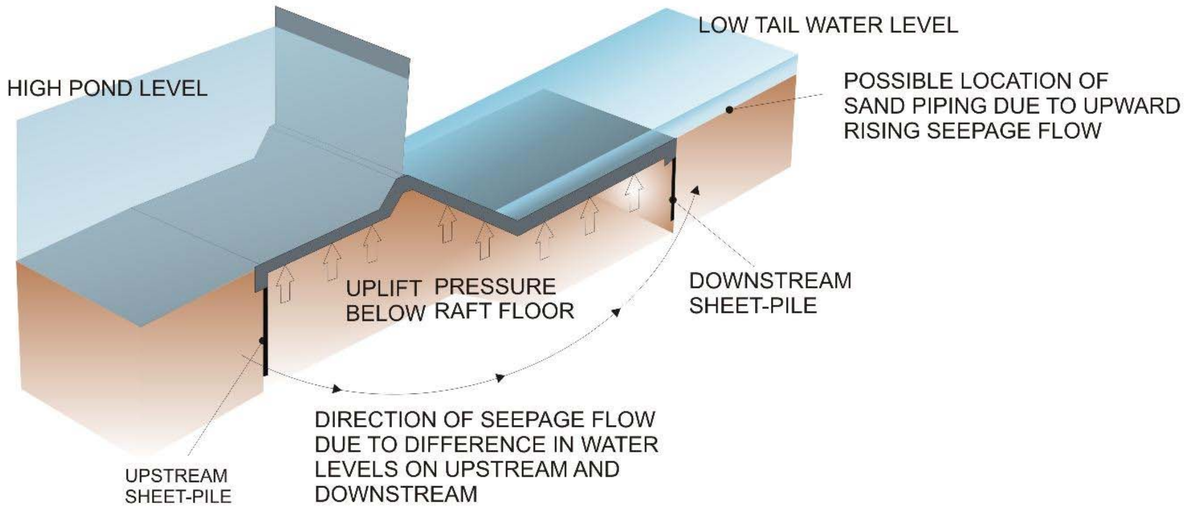


FIGURE 1. Effect of sub-surface flow below barrage floor

Seepage forces would be the most dominating for gates closed condition, but would also exist during some cases of full flow conditions, as shown in Figure 2.

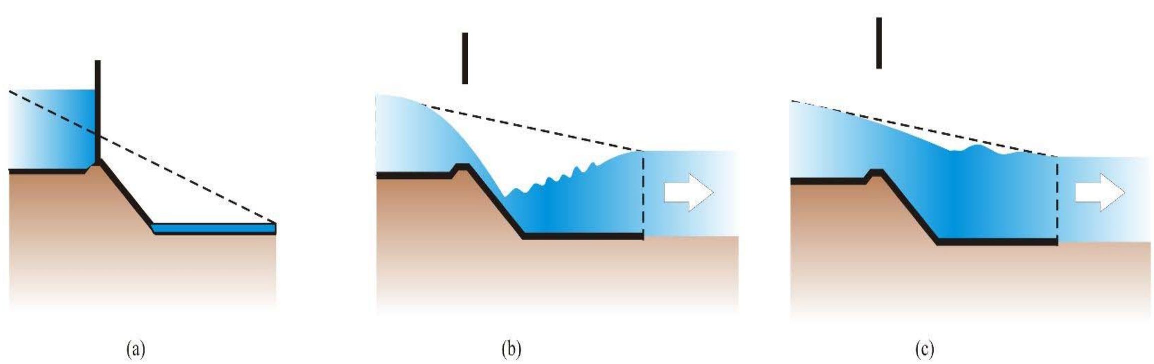


FIGURE 2. Seepage line gradient changes (a) steepest during no flow ; (b) Average during medium flood ; and (c) Almost none during high floods.

It may be noticed that during the flow conditions, a part of the uplift forces due to seepage flow is negated by hydraulic pressure of the water on the downstream. Under the closed gates condition, the downstream water depth is rather small.

In order to evaluate the uplift forces due to seepage flow, it might be worthwhile to recall the mechanism of such flow, as seen from Figure3, the distribution of sub-surface pressure, that is, the pressure of the water held within the pores of the soil is such that it varies from a maximum along the upstream river bed to a minimum at the downstream river bed. The pressure head differential between the upstream and downstream is expressed as a percentage and denoted by ϕ. A comparison of pressure distribution below the barrage floor from Figs. 3(a) and 3(b) indicate that the introduction of sheet piles reduce the pressure below the barrage raft floor. Infact, the seepage paths increase due to the introduction of sheet piles, consequently reducing the gradient of sub-surface pressure.

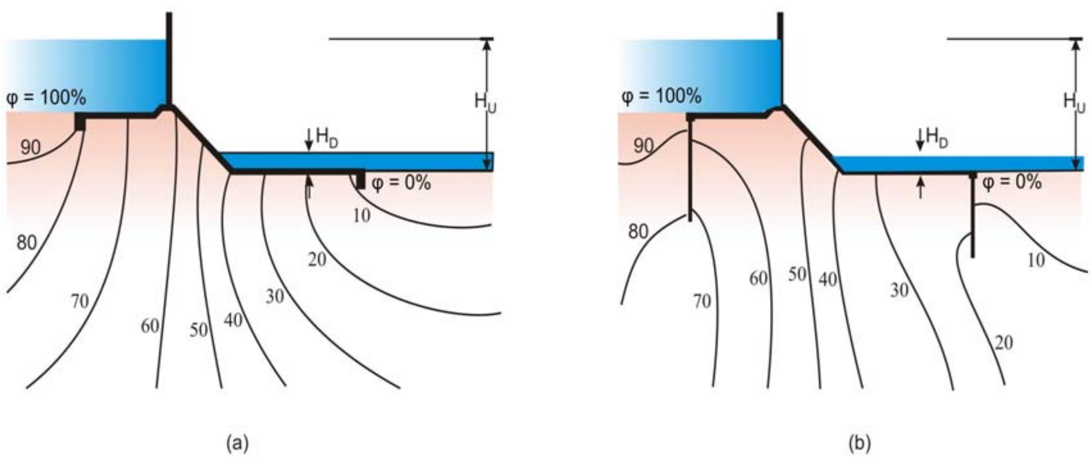


FIGURE 3. Distribution of equipotential lines (a) Barrage floor without sheet piles ; (b) Barrage floor with sheet piles at upstream and downstream ends

It may be noted from the figure that the pressure at any location of a certain equipotential line is given by the following expression:

$$p_\phi = \rho g H_D + \frac{\phi}{100}\rho g(H_U - H_D) = \rho g\left[\left(1-\frac{\phi}{100}\right)H_D + \frac{\phi}{100}H_U\right] \quad (1)$$

In equation (1), H_U is the head of water on the upstream pool above datum and H_D is the head of tail water above datum.

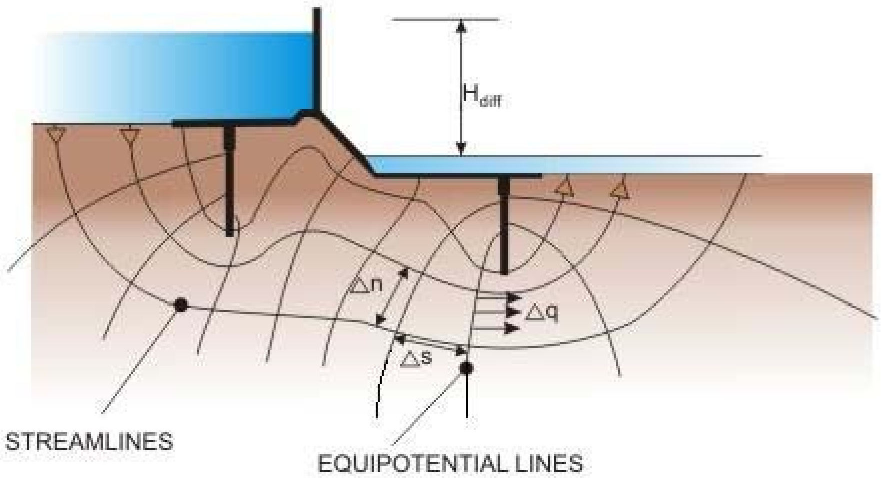


FIGURE .4 Streamlines and equipotential lines below barrage floor and sheetpiles

If a flow net is constructed using both sub-surface equipotential lines as well as stream lines (Figure 4), an estimate may be made of the seepage discharge as given below.

Assuming that a flow channel is designated by the space between two adjacent streamlines, (Figure 4) then the flow through all such flow channels may be considered equal and amounting to, say, $\Delta \mathbf{q}$ m^3/s per metre width. If there are $\mathbf{N_f}$ flow channels, then the total seepage flow **q** would be given as below:

$$\mathbf{q = N_f \ \Delta q} \tag{2}$$

The quantity Δq is governed by Darcy's law is

$$\mathbf{\Delta q = k \ \Delta h / \Delta \ s \quad \Delta \ n} \tag{3}$$

In the above equation **k** is the coefficient of permeability, $\Delta \mathbf{h}$ is the potential drop between two equipotential lines, $\Delta \mathbf{s}$ is the potential length along the stream line of flow net 'square' and $\Delta \mathbf{n}$ is the length normal to the stream line and pressures the other length of the 'square'. $\Delta \mathbf{s}$ and $\Delta \mathbf{n}$ are nearly equal and $\Delta \mathbf{h}$ is equal to $\mathbf{H_{diff} / N_d}$ where $\mathbf{H_{diff}}$ is the head difference between upstream pool and downstream tail water level and $\mathbf{N_d}$ is the number of equipotential drops between the upstream and the downstream river bed. Hence,

$$\mathbf{q = N_f \ k \ H_{diff} / N_d = k \ H_{diff} \ N_f / N_d} \tag{4}$$

The above equation enables the computation of the seepage quantity **q**.

The seeping water below the barrage exerts a dynamic pressure against the river bed particles through whose voids the water is moving. This may be estimated by considering a small cylindrical volume of length **Δl** and cross sectional area **ΔA** in appropriate units. The seepage force on this small volume arises due to the difference in pressure on either side of the cylindrical volume.

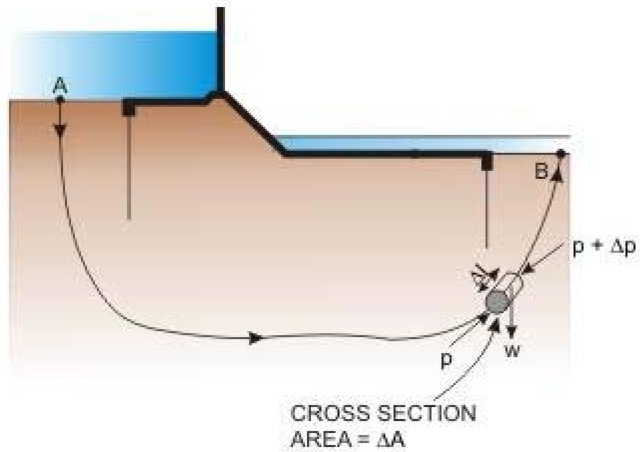


FIGURE 5. Forces on an infinitesimal cylindrical volume aligned along a streamline

In Figure 5, these pressures are shown as **p** on the upstream and **p + Δp** on the downstream sides of the infinitesimal volume. Of course, the higher pressure being on the upstream side, **Δp** would be negative. An expression for the seepage force **ΔF** acting on the cylindrical elementary volume may be expressed as:

$$\Delta F = p.\ \Delta A - (p + \Delta p).\Delta A \qquad (5)$$

This expression yields

$$\Delta F = -\Delta p.\ \Delta A \qquad (6)$$

Thus, the seepage force per unit volume of soil is given as:

$$\Delta F / (\Delta A.\Delta l) = -\Delta p / \Delta l = -\rho\ g.\Delta H/\Delta l \qquad (7)$$

Where ΔH is the difference in head of water on either side of the small volume. Obviously ΔH is negative, since pressure head drops in the direction of flow, and thus the quantity on the right side of the equation would turn out to be positive.

At the exit end, where the stream line meets the river bed surface (B in Figure 5), the seepage force is directed vertically upwards and against the weight of the volume of solid held in the soil. If the seepage force is great enough, it would cause sand-boiling, with the ejection of sand particles causing creation of pipe-like voids through the river bed, on the other hand, the river bed particles at the entry point (A in Figure 5) do not face such a problem, since both the seepage force and the particle weight are both directed vertically downward.

In order to provide safety against piping-failure at the exit end, the submerged weight (**w**) of the solid should be at least equal to the seepage force. This may be expressed as:

$$\mathbf{w = (1\text{-}n)(\rho_s\text{-}\rho)\ g \geq \text{-}\rho g\ \Delta H/\Delta l} \qquad (8)$$

In the above, **w** is the submerged weight of the solids assuming a void ratio **n**. ρ_s and ρ stand for the density of the solids and water, respectively. The equation then simplifies to

$$\mathbf{\text{-}\Delta H/\Delta l \leq (1\text{-}n)\ (G\text{-}1)} \qquad (9)$$

Where **G** is the relative density of the soil.

The quantity $\Delta H/\Delta l$ represents the hydraulic gradient of the subsurface water at the exit end of the streamline, and is also called the Exit Gradient. This should not exceed the given value in order to prevent failure by piping. Assuming **G** and **n** to be nearly equal to 2.65 and 0.4 respectively for sandy bed, the limiting value of $|\Delta H/\Delta l|$ turns out to be approximately equal to 1.0. However, it is not enough to satisfy this limiting condition. Even a slight increase in the value will upset the stability of the sub-soil at the exit end. This requires the application of a generous factor of safety in the designs, which may be considered as a precaution against uncertainties such as:

- Non- homogeneity of the foundation soil
- Difference in the packing and pore space
- Local intrusion of impervious material like clay beds or very porous material
- Faults and fissures in sub-soil formation, etc.

The factor of safety to be applied would take care of all these uncertainties besides covering cases where due to retrogression of bed levels or flood scour, the upper part of the piles at the downstream is exposed. According to the Bureau of Indian Standards Code IS 6966 (part 1): 1989 "Hydraulic design of barrages and weirs- guidelines", the following factors of safety may be considered according to the variation of river bed material:

Sub-soil material	Factor of safety
Shingle	4 to 5
Coarse sand	5 to 6
Fine sand	6 to 7

4.2.2 Computation of seepage pressure and Exit gradient:

With the advent of numerical computational tools and computers with high precision speeds, numerical solution of the Laplace equation representing sub-surface flow has become quite common these days to evaluate the above parameters. However, analytical solutions have been derived by a group of engineers and scientists in India comprising of A.N. Khosla, N.K. Bose and M. Taylor and presented in simple analytical forms and graphs. These can be used to arrive at a quick solution to a given problem. They arrived at these equations after conducting several experiments and solving the Laplace equation under simplified conditions using the Schwartz Christoffel transformation theory. The results of their mathematical solutions have been published under publication 12 of the Central Board of Irrigation and Power titled “Design of weirs on permeable foundations”. Of course, the soil confined below a barrage conforms to a complex shape and is not readily amenable to solution using analytical formulae but still the following simple profiles have been found to be very useful for approximately arriving at the subsurface pressures of a barrage or canal head regulator floor.

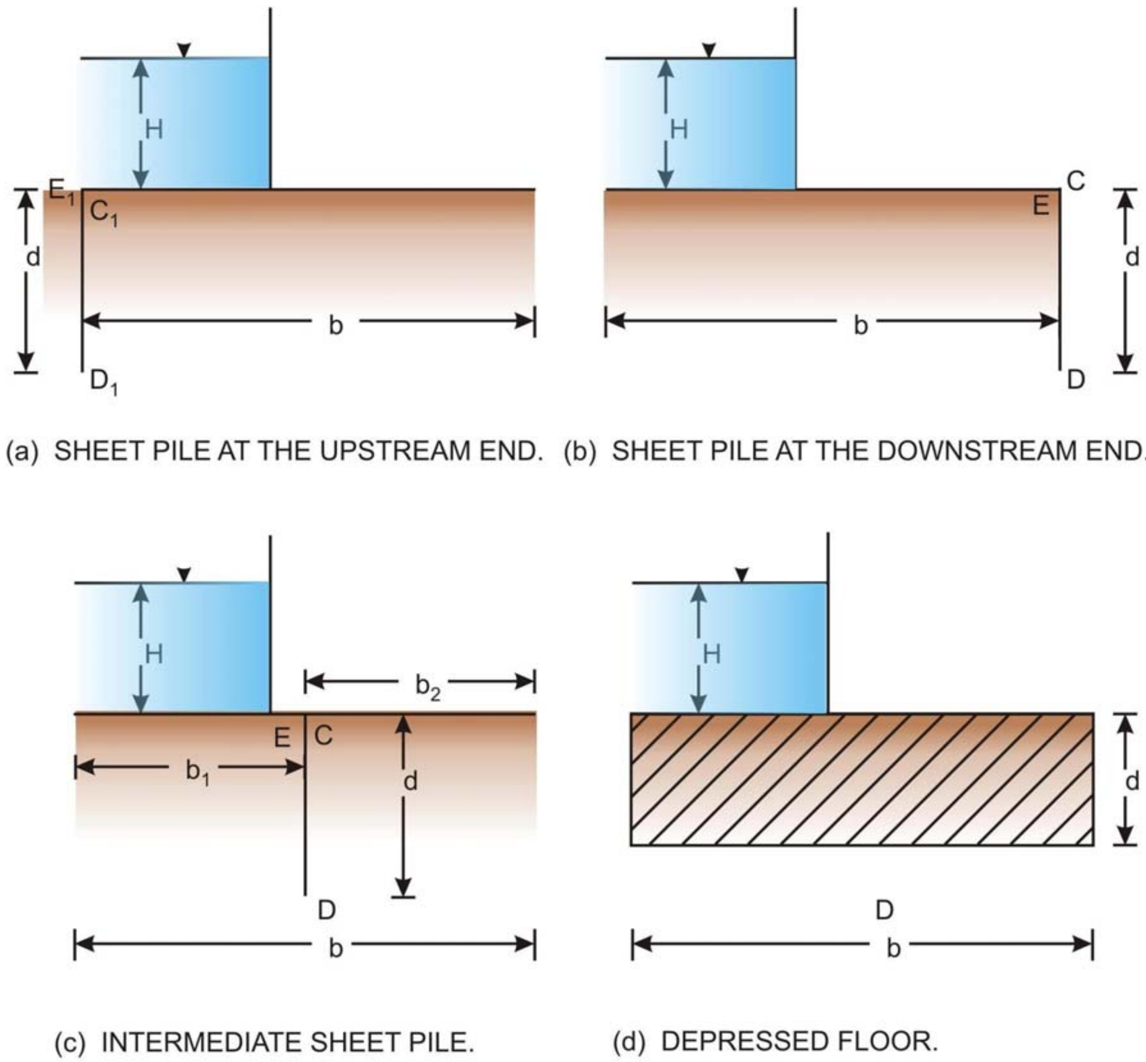


FIGURE 6. Simple standard profiles for finding sub-soil pressure at key points

- A straight horizontal floor of negligible thickness with a sheet pile at either end [Figure 6(a) or 6(b)].

- A straight horizontal floor of negligible thickness with an intermediate sheet pile [Figure 6(c)].

- A straight horizontal floor depressed below the bed but with no sheet pile [Figure 6(d)].

The solution for these simple profiles has been obtained in terms of the pressure head ratio (or percentage) at key points as shown in the figures.

These key points are the junction points of the sheet pile with floor incase of floors of negligible thickness and at the corners of the base at the upstream and down stream end incase of depressed floor. The analytical expressions of each of the above cases are given as under:

- For sheet piles at either upstream end [Figure 6(a)] or the down stream end [Figure 6(b)].

$$\phi_E = (1/\pi) \cos^{-1}[(\lambda-2)/ \lambda] \quad (10)$$

$$\phi_D = (1/\pi) \cos^{-1}[(\lambda-1)/ \lambda] \quad (11)$$

$$\phi_{C1} = 100 - \phi_E \quad (12)$$

$$\phi_{D1} = 100 - \phi_D \quad (13)$$

$$\phi_{E1} = 100 \quad (14)$$

where $\lambda = (1/2)[1+\sqrt{(1+\alpha^2)}]$
and $\alpha = (b/d)$

- For sheet piles at the intermediate point [Figure 6(c)]

$$\phi_E = (1/\pi) \cos^{-1}[(\lambda_1-1)/ \lambda_2] \quad (15)$$

$$\phi_D = (1/\pi) \cos^{-1} [\lambda_1/ \lambda_2] \quad (16)$$

$$\phi_C = (1/\pi) \cos^{-1}[(\lambda_1+1)/ \lambda_2] \quad (17)$$

In the above equations, **λ1** and λ_2 are given as:

$$\lambda_1= (1/2)[\sqrt{(1+\alpha_1^2)} - \sqrt{(1+\alpha_2^2)}] \quad (18)$$

$$\lambda_2= (1/2)[\sqrt{(1+\alpha_1^2)} + \sqrt{(1+\alpha_2^2)}] \quad (19)$$

where $\alpha_1 = (b_1/d)$

$\alpha_2 = (b_2/d)$

- For the case of a depressed floor ,

$$\phi_D' = \phi_D - (2/3)[\phi_E - \phi_D] + (3/ \alpha^2) \quad (20)$$

$$\phi_D' = 100 - \phi_D$$

Where ϕ_D and ϕ_E are given as

$$\phi_D = (1/\pi)\cos^{-1}[(\lambda-1)/\lambda] \qquad (21)$$

$$\phi_E = (1/\pi)\cos^{-1}[(\lambda-2)/\lambda] \qquad (22)$$

The above expressions may also be determined from the graph shown in Figure 7.

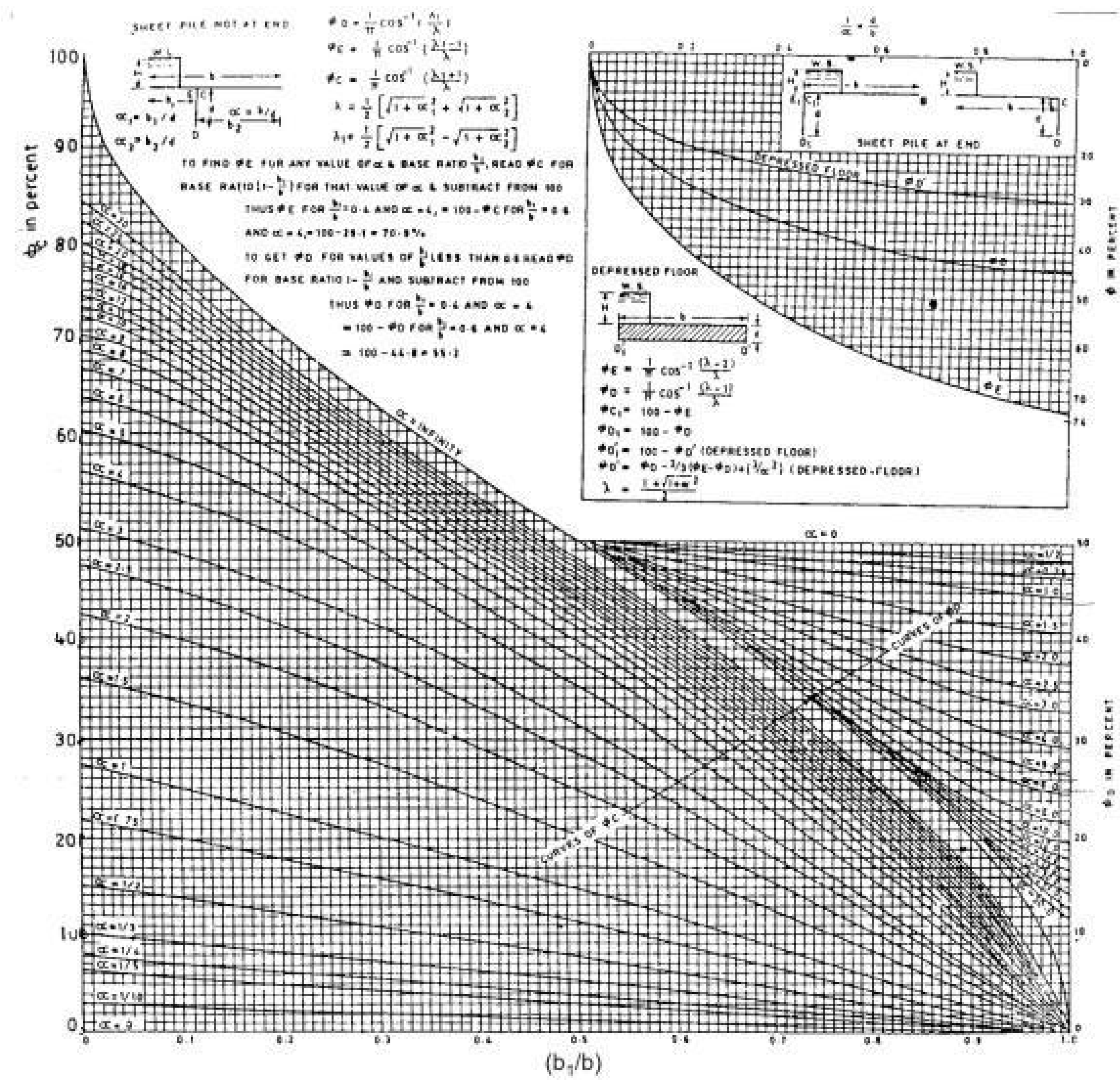


FIGURE 7. Curves given by Khosla, Bose and Taylor for estimation of uplift

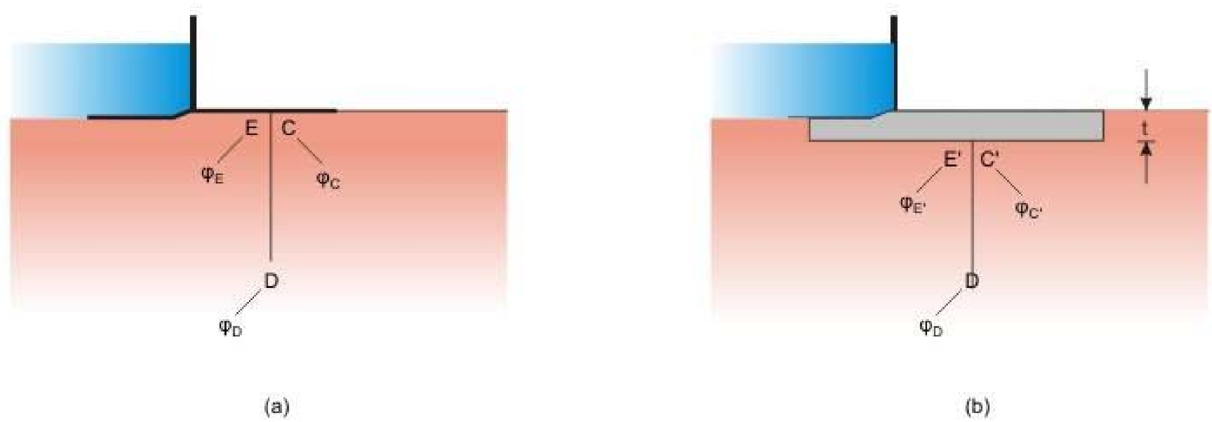
The uplift pressures obtained by the analytical expressions or graphical methods need to be corrected for the following more realistic conditions:

- Floor raft with sheet piles at either end actually has a floor thickness that can not be considered as negligible compared to the sheet pile depths.
- A floor raft may have sheet piles both at the upstream as well as the downstream ends, which might interfere one with the other.
- The floor of a modern barrage is not horizontal throughout.

Some formulas have, therefore, been suggested for incorporating the necessary corrections which are expressed as follows:

Correction for floor thickness

Figure 8 illustrates the correction to the evaluated values at key points E and C that is applied considering a floor thickness ***t***.



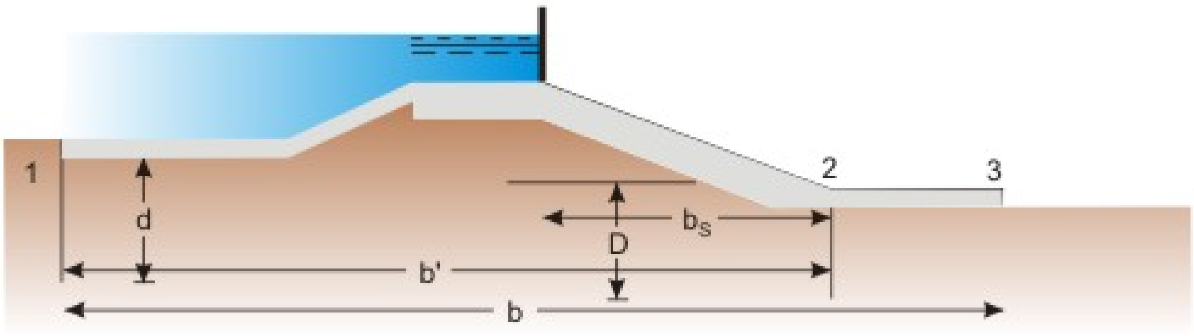
$$\varphi_{E'} = \varphi_E - \frac{\varphi_E - \varphi_D}{d + t}\, t$$

$$\varphi_{C'} = \varphi_C - \frac{\varphi_D - \varphi_C}{d + t}\, t$$

FIGURE 8. Correction to uplift for floor thickness (a) φ_D , φ_E and φ_C as obtained from simplified formulae ; (b) Corrected values $\varphi_{E'}$ and $\varphi_{C'}$ at the same location considering thickness of floor

Correction for mutual interference of sheet piles

Figure 9 gives the amount correction C (in percent) for interference of one sheet pile on the other.



CORRECTION FOR MUTUAL INTERFERENCE OF SHEET PILES.

$$C = 19\sqrt{\frac{D}{b'}}\left(\frac{d+D}{b}\right)$$

WHERE

C IS THE CORRECTION TO BE APPLIED AS PERCENTAGE OF HEAD.

b' IS THE DISTANCE BETWEEN THE TWO PILE LINES (SEE SKETCH).

b_s = SLOPING LENGTH OF FLOOR.

D = DEPTH OF THE PILE LINE , THE INFLUENCE OF WHICH HAS TO BE DETERMINED ON THE NEIGHBOURING PILE OF DEPTH d , D IS TO BE MEASURED BELOW THE LEVEL AT WHICH INTERFERENCE IS DESIRED.

d = DEPTH OF PILE ON WHICH THE EFFECT OF PILE OF DEPTH D IS SOUGHT TO BE DETERMINED.

b = TOTAL FLOOR LENGTH.

FIGURE 9. Correction factor for mutual interference of sheet piles.

Correction for slope of the floor

The correction coefficient for taking care of inclined floor is given in Figure (10). The correction coefficients may also be read from the following table:

Values of slope correction

Slope(V:H)	Correction of percent of pressure
1:1	11.2
1:2	6.5

1:3	4.5
1:4	3.3
1:5	2.8
1:6	2.5
1:7	2.3
1:8	2.0

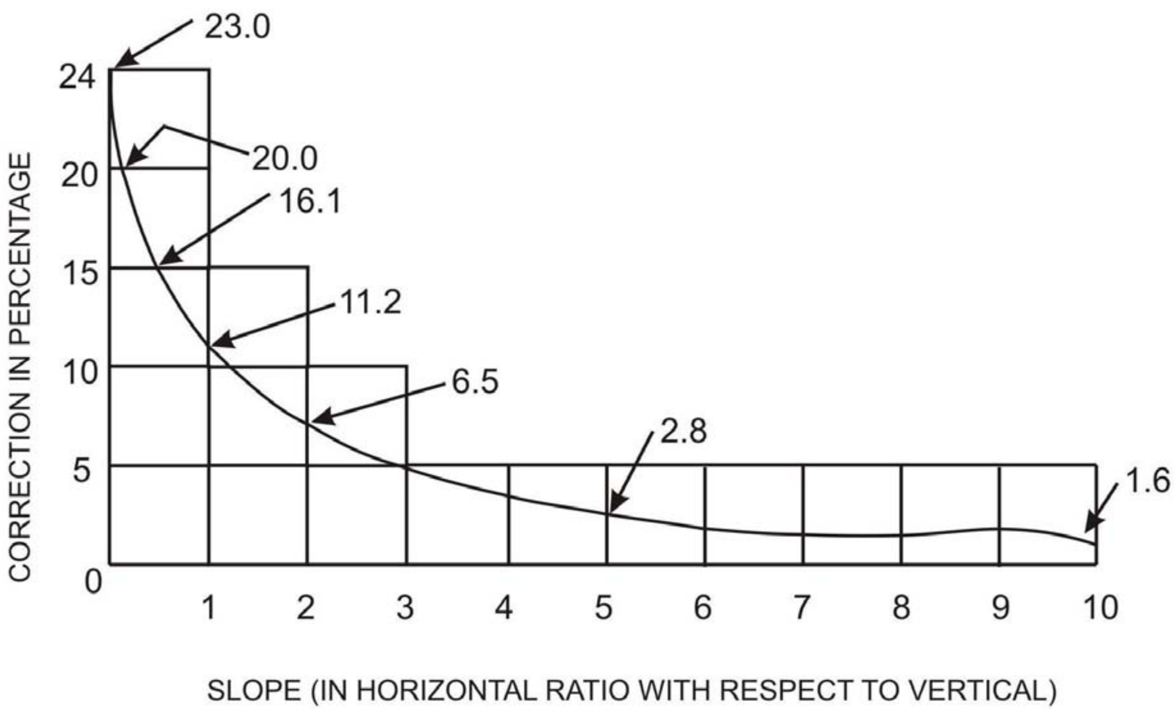


FIGURE 10. Correction coefficient for sloping floor.

The corrections will have to be taken positive for down slopes and negative for up-slopes taken in the direction of flow. The corrections are applicable to the key points of the pile line fixed at the beginning or the end of the slope, for example the pile line 2 at its end E for the floor and sheet-pile shown in Figure 9.

Corresponding to the downstream sheet pile [Figure 6 (b)], the ***exit gradient***, denoted as $\mathbf{G_E}$, is given below:

$$G_E = (H/d)\ (1/\pi \sqrt{\lambda}) \qquad (23)$$

The above equation or its equivalent graphical form shown in Figure11 gives a valve of G_E equal to infinity if there is no downstream sheet pile (d=0). It is, therefore, essential that a downstream sheet pile should invariably be provided for any barrage floor. As mentioned earlier, the calculated exit gradient must not be allowed to exceed the critical value of that of the soil comprising the riverbed material.

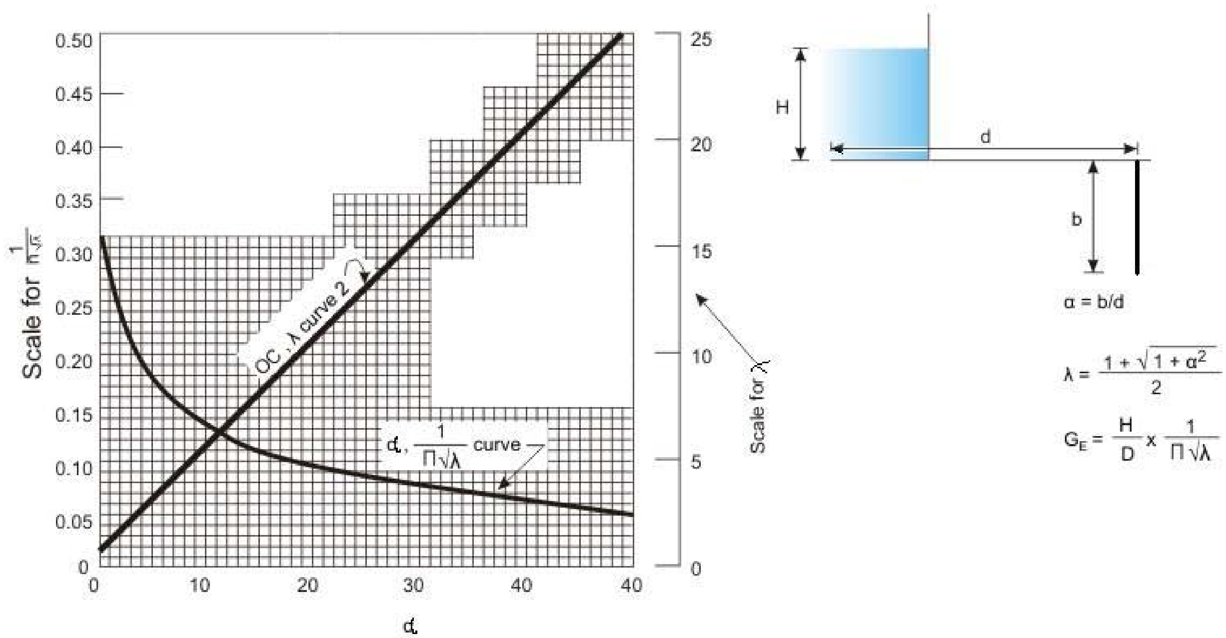


FIGURE 11. Curves for estimating Exit Gradient.

4.2.3 Hydraulics of barrage surface flow

A barrage constructed across a river has to pass floods of different magnitudes each year and the gates have to be operated in such away that the water level in the pool is kept at least at the Pond Level (PL). A very high flood would require opening of all the gates to provide an almost obstruction-less flow to the flood. For smaller floods, the gates may not have to be opened fully to provide unobstructed flow. The gates of all the bays of a barrage are not usually opened uniformly, but is opened more towards the side where more flow is to be attracted due to certain site-specific reasons. Nevertheless, the requirement of maintaining pond level means that as the flood rises in a river, more and more gate opening is provided till such time when the gates are fully open.

The corresponding upstream stage-discharge curve shown in Figure 12(a) shows that up to a river discharge of Q_0, the water level behind the barrage is maintained at Pond Level. At any higher discharge, the stage discharge curve is similar to that of the normal river downstream [Figure 12(b)] but with an afflux.

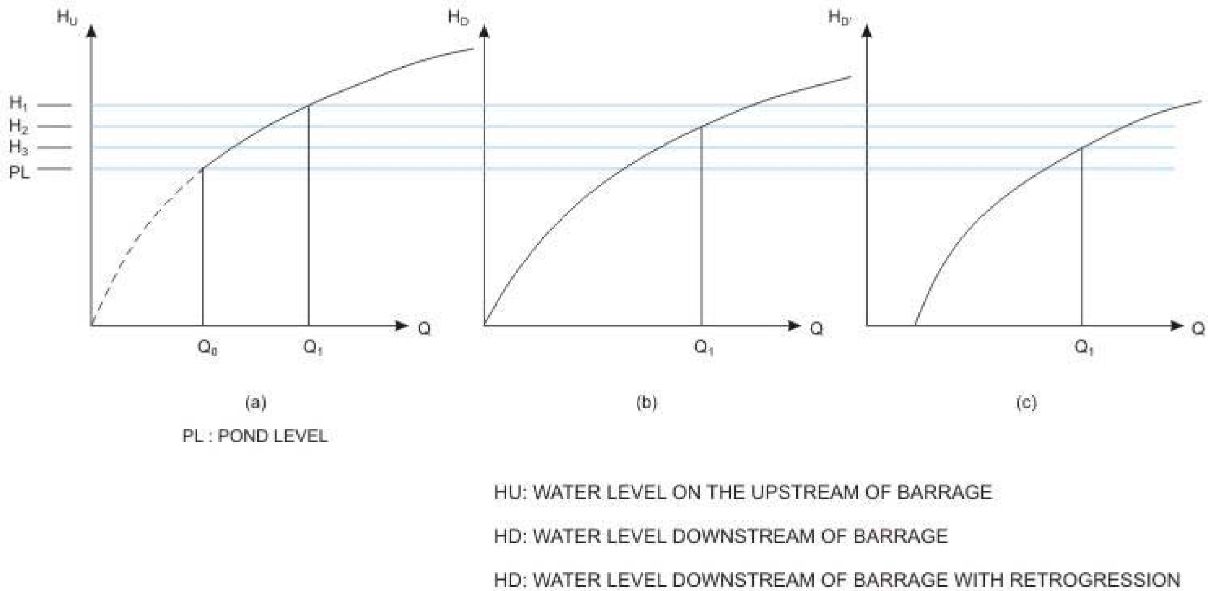


FIGURE 12. Stage (water - level) versus discharge curves. (a) Upstream of barrage ; (b) Downstream of barrage (c) Downstream of barrage , with retrogression.

Hence, at any discharge Q greater than Q_0, the water level behind the barrage (H_u) is higher than that downstream of the barrage (H_D). In some rivers the construction of a barrage causes the downstream riverbed to get degraded to progressively up to a certain extent, a phenomenon that is called Retrogression, which has been found to be more pronounced in alluvial rivers carrying more silt or having finer bed material and having steep slope. IS 6966 (part 1): 1989 “Hydraulic design of barrages and weirs-guidelines” recommends a retrogression of 1.25 to 2.25 m for alluvial rivers at lower river stages depending upon the amount of silt in the river, type of bed material, and slope. As a result of retrogression, low stages of the river are generally affected more compared to the maximum flood levels. At the design flood, the reduction of stages due to retrogression may be within 0.3 to 0.5m depending upon whether the river is shallow or confined during floods. Figure 12(c) shows a typical retrogressed water stage versus discharge and for the same discharge Q_1, the corresponding water level ($H_{D'}$) would be much lower than the upstream water level (H_u).

The above discussion implies that for the same flood discharge, a non-retrogressed river may exhibit submerged flow phenomenon [Figure 13(a)] compared to a free flow condition [Figure 13(b)] expected for a retrogressed condition. As a consequence, there would be a difference in scour depths in either case. Nevertheless, IS 6966 [part 1]: 1989 recommends that for non cohesive soils, the depth of scour may be calculated from the Lacey's formula which is as follows:

$$\mathbf{R=0.473(Q/f)^{1/3}} \quad \text{when looseness factor is more than 1} \tag{24}$$

or

$$\mathbf{R=1.35(q^2/f)^{1/3}} \quad \text{when looseness factor is less than 1} \tag{25}$$

Where

R=depth of scour below the highest flood level (in meters).
Q=high flood discharge in the river (in m^3/s)
f=silt factor which may be calculated knowing the average particle size m_r(in mm), of the soil from the following relationship

$$\mathbf{f=1.76\sqrt{d_{50}}} \tag{26}$$

q=intensity of flood discharge is in m^3/s per meter width

The extent of scour in a river with erodible bed material varies at different places along a barrage. The likely extent of scour at various points are given in the following table:

Location	Range	Mean
Upstream cutoff (sheet pile) depth	1.0R*	
Downstream cutoff (sheet pile) depth	1.25R*	
Flexible apron upstream of impervious floor	1.25 to 1.75R	1.5R
Flexible apron downstream of impervious floor	1.75 to 2.25R	2.0R
Noses of guide banks	2.00 to 2.50R	2.25R
Noses of divide wall	2.00 to 2.50R	2.25R
Transition from nose to straight	1.25 to 1.75R	1.50R
Straight reaches of guide banks	1.00 to 1.50R	1.25R

* A concentration factor of 20 percent is to be taken into account in fixing the depth of sheet piles. These should be suitably extended into the banks on both sides up to at least twice their depth from top of the floors.

It is quite common to find layers of clay below the riverbed of alluvial rivers in which case, a judicious adjustment in the depths of upstream and downstream sheet-piles shall have to be made to avoid build up of pressure under floor.

4.2.4 Fixing dimensions of barrage components

The hydraulic calculation for a barrage starts with determination of the waterway, as discussed in Lesson 4.1. For shallow and meandering rivers, the minimum stable width (P) can be calculated from Lacey's modified formula

$$\mathbf{P=4.83\ Q^{1/2}} \qquad (27)$$

Where, the discharge **Q** is in m^3/s. For rivers with very wide sections, the width of the barrage is limited to Lacey's width multiplied by looseness factor and the balance width is blocked by tie bunds with suitable training measures. Assuming the width of each bay to be between 18m to 20m and pier width to be around 1.5 the total number of bays is worked out. The total number of bays is distributed between spillway, under-sluice and river-sluice bays. The crest levels of the different bays may be fixed up on the basis of the formulations in Lesson 4.1.

With these tentative values, the adequacy of the water way for passing the design flood within the permissible afflux needs to be checked up. Otherwise, the water way and crest levels will need to be readjusted in such a way that the permissible values of afflux are not exceeded.

The discharge through the bays of a barrage (spillway or under sluices) for an uncontrolled condition (as during a flood discharge) is given as:

$$\mathbf{Q=CLH^{3/2}} \qquad (28)$$

Where **L** is the clear water way (in meters) **H** is the total head (including velocity head) over crest (in meters) and **C** is the coefficient of discharge, which for free flow conditions [as shown in Figure 13 (b)] may be taken as 1.705 (for Broad-crested weirs/spillways) or 1.84 (for Sharp-crested weirs/ spillways). Roughly, a spillway or weir is considered to be Broad Crested if a critical depth occurs over its crest. However, with the general dimensions of a barrage spillway (with the crest width generally being kept at about 2m) and the corresponding flow depths usually prevailing, it would mostly act like a Sharp-Crested spillway. Under sluices and river sluices (without a crest) would behave as broad-crested weir. Another point that may be kept in mind is that the total head **H** also includes the velocity head $\mathbf{V_a^2/2g}$, where $\mathbf{V_a}$ is the velocity of approach and may be found by dividing the total discharge by the flow **Q** cross section area **A**. The quantity **A**, in turn, may be found out by multiplying the river width by the depth of flow, which has to be taken not as the difference of the affluxed water level and the normal river bed, but as the depth of scour measured from water surface as mention in section 4.2.3.

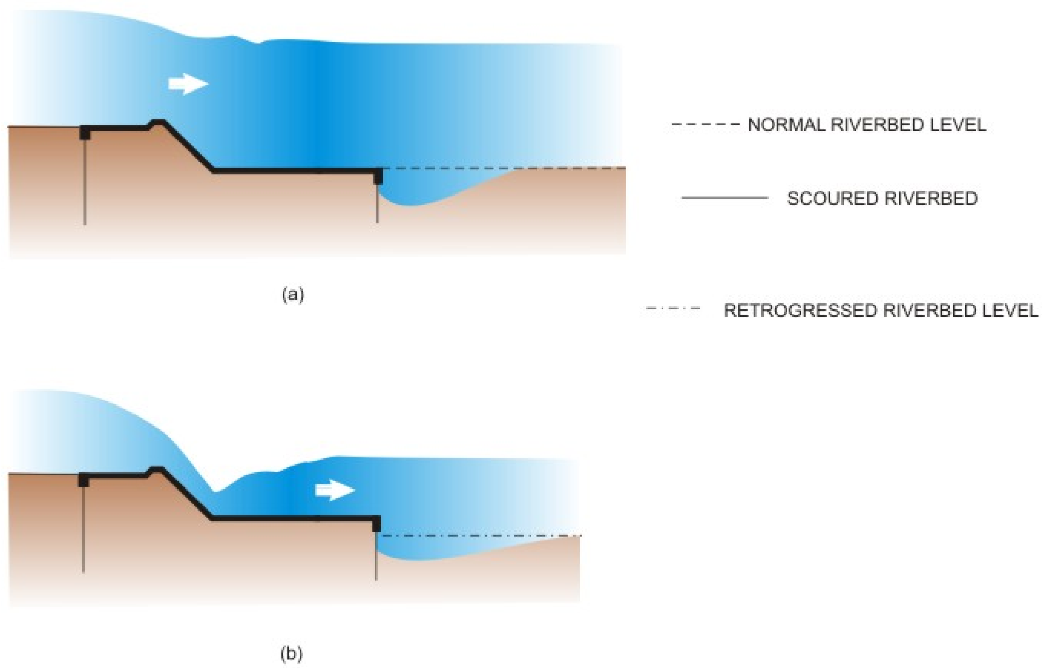


FIGURE 13. Jump formation modes in a barrage due to same discharge ; (a) Submerged jump for high tail water level ; (b) Free jump for low tail water level due to retrogression or steep river slope

It may be noticed from Figure 13 (a) that a barrage spillway/under sluice can also get submerged by the tail water. In such a case, one has to modify the discharge by multiplying with a coefficient, **k**, which is dependent on the degree of submergence, as shown in Fig .14.

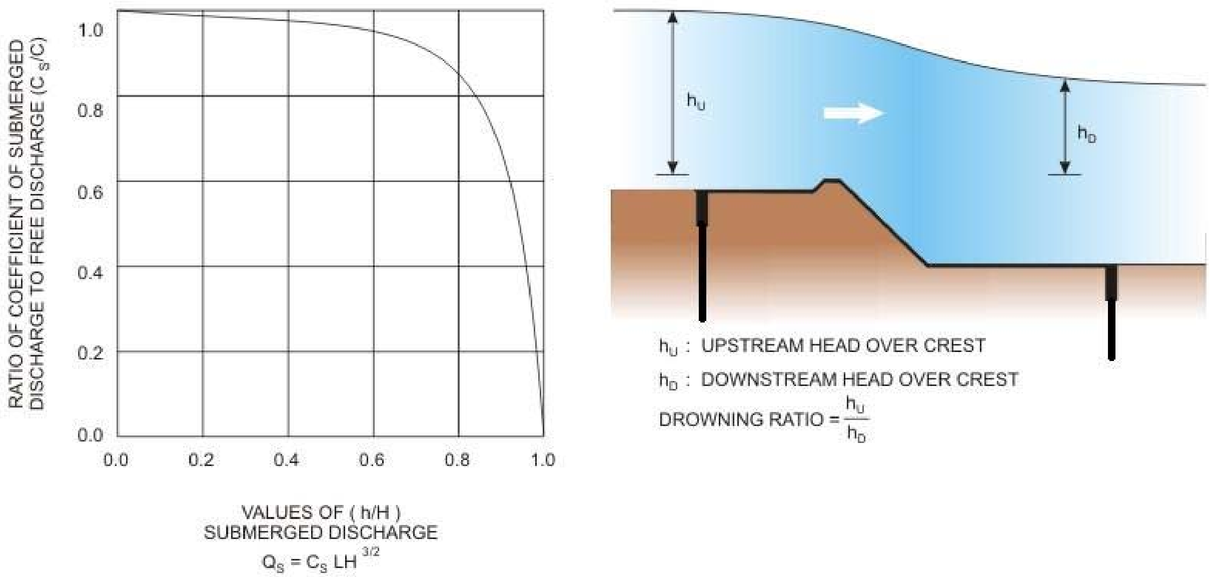


FIGURE 14. Multiplying coefficient (k) for transition from free flow to submerged flow conditions.

Since the crest levels of spillway, under sluice and river-sluice bays would be different, the discharge passing through each will have to be estimated separately and then summed up. Wherever silt excluder tunnels are proposed to be provided in the undersluice bays, the discharge through these tunnels and over them need to be calculated separately and added up.

Having fixed the number of spillway, river-sluice and under sluice bays and their crest levels, it is necessary to work out the length and elevation of the corresponding downstream floors. The downstream sloping apron extending from the crest level to the horizontal floor is usually laid at an inclination of 3H:1V, and the structure is designed in such a way that any hydraulic jump formation (during free flow condition) may take place only on the sloping apron. Thus, the worst case of low tail water level, which governs the formation of a hydraulic jump at the lowest elevation decides the location of the bottom end elevation of the slope as well as that of the horizontal floor (Figure 15). The length of the horizontal floor (also called the cistern) is governed by the length of the jump, which is usually taken as $5(D_2-D_1)$ where D_1 is the depth of water just upstream of the jump and D_2 is the depth of water downstream of the jump (Figure 15).

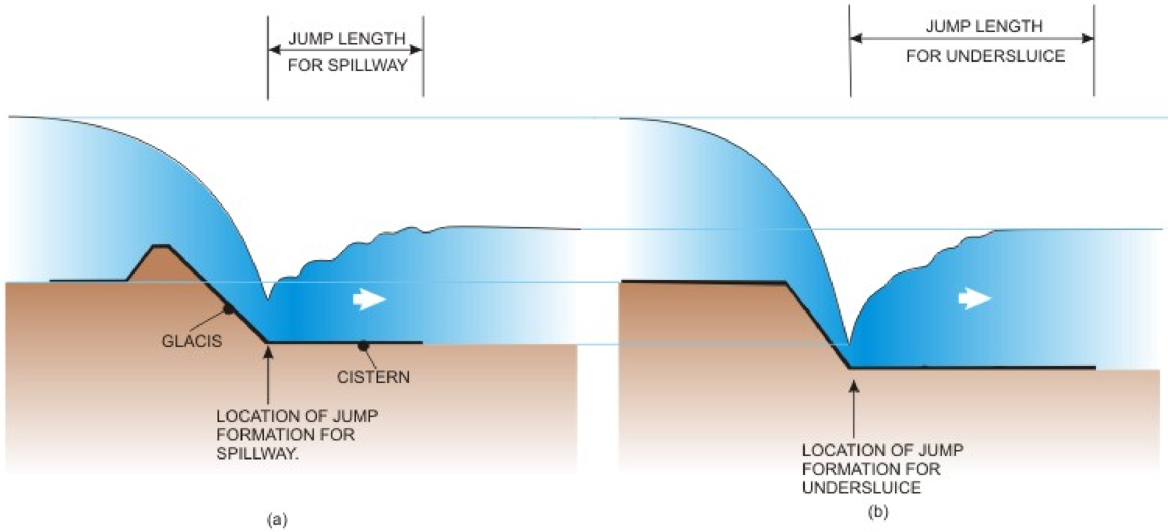


FIGURE 15. Jump formation at lowest end of Glacis for (a) Spillway bays ; (b) Underslice bays.

It may be observed from the figure that though the upstream and downstream water levels of the spillway and under sluice bays are same for a particular flow condition, the difference in crest elevations (here the under sluice portion is shown without a crest) causes more flow per unit width to pass through the undersluice bays. This results in a depressed floor for the under sluices bays compared to the spillway bays.

The cistern level and its length for the spillway, river-sluice or under sluice bays have to be determined for various sets of flow and downstream water level combinations that may be physically possible on the basis of the gate opening corresponding to the river inflow value, The most severe condition would give the lowest cistern level and the maximum length required, the hydraulic conditions that have to be checked are as follows:

1) Flow at Pond level, with a few gates opened.
2) Case 1 with discharge concentration enhanced by 20% and a retrogressed downstream riverbed level.
3) Flow at High Flood Level, with all gates opened.
4) Case 3 with discharge concentration enhanced by 20% and a retrogressed downstream riverbed level.

The determination of cistern level, either through the use of the set of curves known as Blench Curves and Montague curves have to be used, or they may be solved analytically. Here, the latter has been demonstrated and readers interested to know about graphical method, may go through any standard textbook on hydraulic structures or irrigation engineering as the following:

Asawa, G L (1995) Irrigation Engineering, New Age Publishers

The various steps followed in the determination of the cistern level by analytical methods are as follows:

1) For any given hydraulic condition, calculate the Total Energy **T.E.** (**=H+ V^2/2g**, where **H** is the water head above a datum and **V** is the average velocity) on both upstream and down stream of the barrage corresponding discharge per unit width **q**.

2) Assume a cistern level.

3) Then, the energy above crest level on the upstream, E_{f1}, is determined as :
E_{f1}=upstream T.E.L - Assumed cistern level.

4) From the known values of E_{f1} and **q** with 20% concentration, D_1 (the depth of water before jump) is calculated using the following relationship:

$$E_{fi} = D_1+q2/(2gD_1^2)$$

Here, it is assumed that there is negligible energy loss between the upstream point where E_{f1} is calculated and up to beginning of the hydraulic jump. In the above equation, g is the acceleration due to gravity.

5) Calculate the pre-jump Froude number F_{r1} using the equation :

$$F_{r1}^2 = q^2/ (gD_1^3)$$

6) From the calculated values of D_1 and F_{r1}, the post jump depth (D_2) can be calculated from the following relationship:

$$D_2= D_1/2(-1+\sqrt{1+8 F_{r1}^2})$$

7) The required cistern level for the considered case of hydraulic condition would be equal to the retrogressed down stream water level minus D_2.

8) In the first trial, the initially assumed cistern level chosen in step 2 and the calculated cistern level in step 7 may not be the same which indicates that the cistern level assumed initially has to be revised. A few trials may be required to

arrive at a final level of the cistern. The required length of the cistern is then found out as **$5(D_2-D_1)$**.

The cistern is designed with the final dimension arrived at from the hydraulic calculations mentioned above. However, the length of the horizontal floor can be reduced with a corresponding saving in cost if the normal steady level of the downstream water is obtained in a distance less than 5 times the jump height by addition of some appurtenant structures. The most common is the addition of either one or a combination of the following to the horizontal downstream floor:

1) A continuous end sill
2) Raised blocks

According to Bureau of Indian Standards Code IS:4997-1968 "Criteria for design of hydraulic jump type stilling basins with horizontal and sloping apron", there are many designs of the appurtenant structures recommended for different Froude numbers of flow over the spillway and downstream tail water level. As such, the Type1 Indian standard stilling basin (Figure 16), which is recommended for inflow Froude number less than 4.5, is suitable for the design of cisterns of barrages.

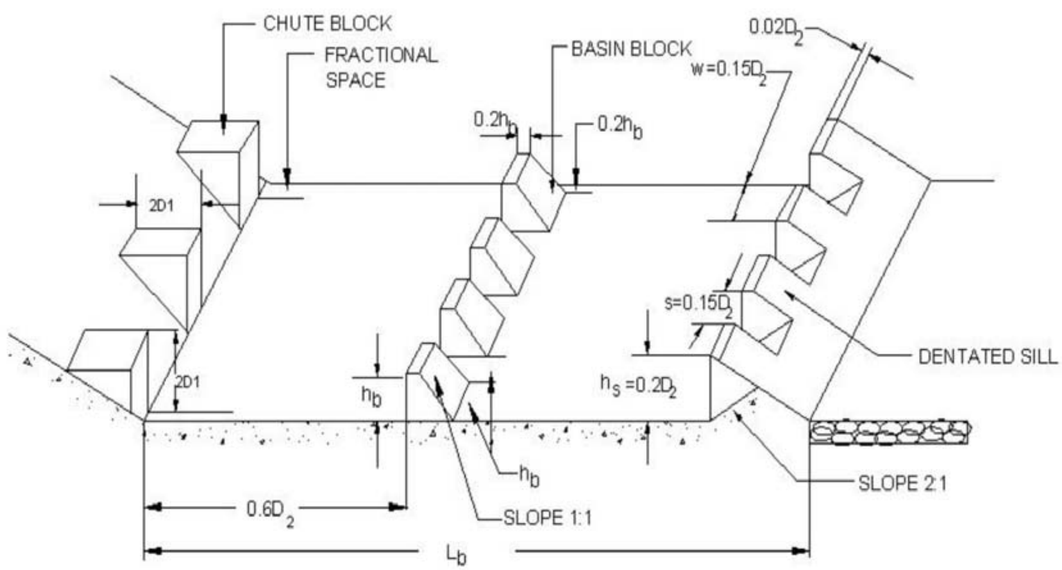


FIGURE 16. STILLING BASIN OF TYPE I RECOMMENDED FOR BARRAGE SPILLWAYS

The length of the upstream floor of the barrage may be fixed after knowing the total floor length required from sub-surface flow conditions and subtracting the crest length, glacis length and down stream cistern length as calculated from the surface flow conditions.

4.2.5 Protection works

Just upstream and downstream of the solid floor of the spillway apron, the river bed is protected by certain methods like block protection, loose stone apron, etc. as may be seen from Figure 17 showing a typical section of spillway of a barrage. These protection works are discussed below:

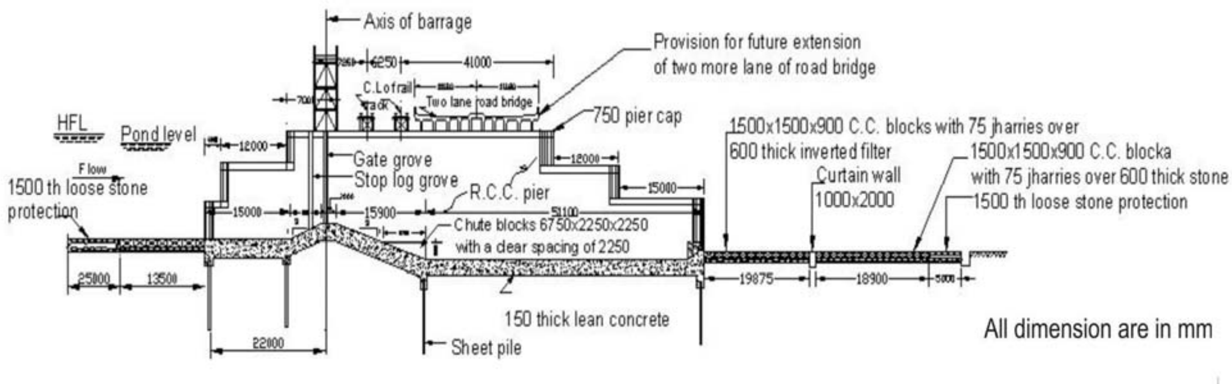


FIGURE 17. Section through a typical barrage spillway

Upstream block protection

Just beyond the upstream impervious floor, pervious protection comprising of cement concrete blocks of adequate size laid over loose stone shall have to be provided. The cement concrete blocks of size around 1.5m x 1.5m x 0.9m are generally used for barrages in alluvial rivers. The length of the upstream block protection may be kept equal to a length D, that is, the design depth of scour below the floor level (Figure 18).

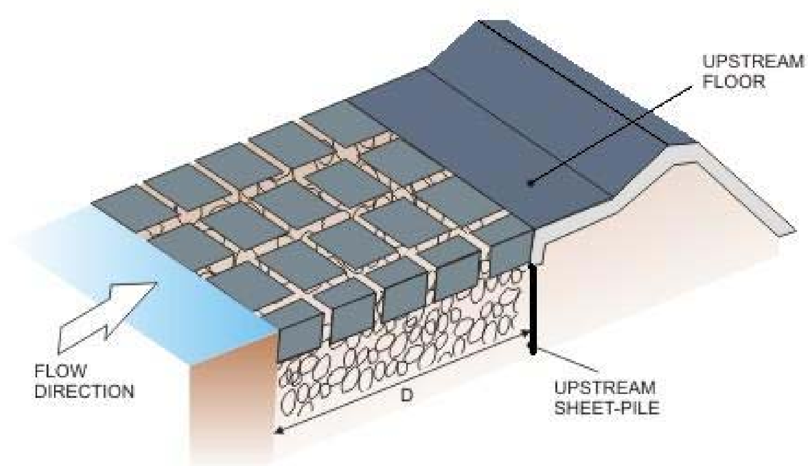


FIGURE 18. UPSTREAM BLOCK PROTECTION

Downstream block protection

Pervious block protection is provided just beyond the down stream impervious floor. It comprises of cement concrete blocks of size 1.5m x 1.5m x 0.9m laid with gaps of 75mm width and packed with gravel. The downstream block protection is laid on a graded inverted filter designed to prevent the uplift of fine sand particles upwards due to seepage forces. The filter should roughly conform to the following design criteria.

1) $$\frac{d_{15}\ of\ filter}{d_{15}\ of\ foundation} \geq 4 \geq \frac{d_{15}\ of\ filter}{d_{85}\ of\ foundation}$$

Where d_{15} and d_{85} represent grain sizes. d_x is the size such that x% of the soil grains are smaller than that particle size. Where x may be 15 or 85 percent.

2) The filter may be provided in two or more layers. The grain size curves of the filter layers and the base material has to be nearly parallel.

The length of the downstream block protection has to be approximately equal to 1.5D, where D is the design depth of cover below the floor level. Where this length is substantial, the block protection with inverted filter may be provided in part of the length and block protection only with loose stone spawls in the remaining length as shown in Figure 19.

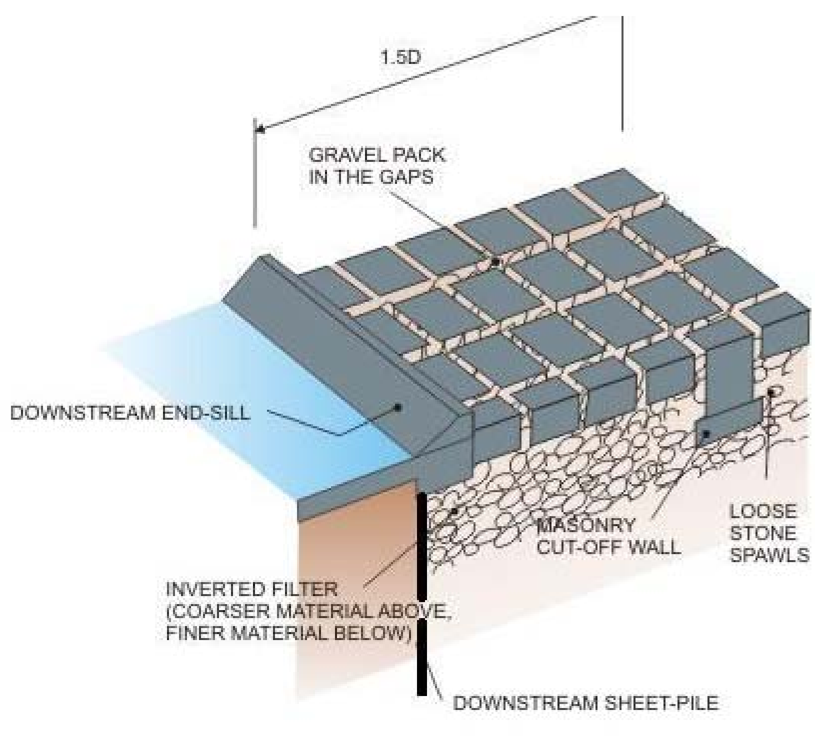


FIGURE 19. DOWNSTREAM BOLCK PROTECTION

Loose stone protection

Beyond the block protection on the upstream and down streams of a barrage located on alluvial foundation, a layer of loose boulders or stones have to be laid, as shown in Figure 20(a). The boulder size should be at least 0.3m and should not weigh less than 40kg. This layer of boulders are expected to fall below at an angle , or launch , when the riverbed down stream starts getting scoured at the commencement of a heavy flood [Figure 20(b)]. The length of river bed that has to be protected with loose stone blocks shall be around 1.5D, where D is the depth of scour below average riverbed.

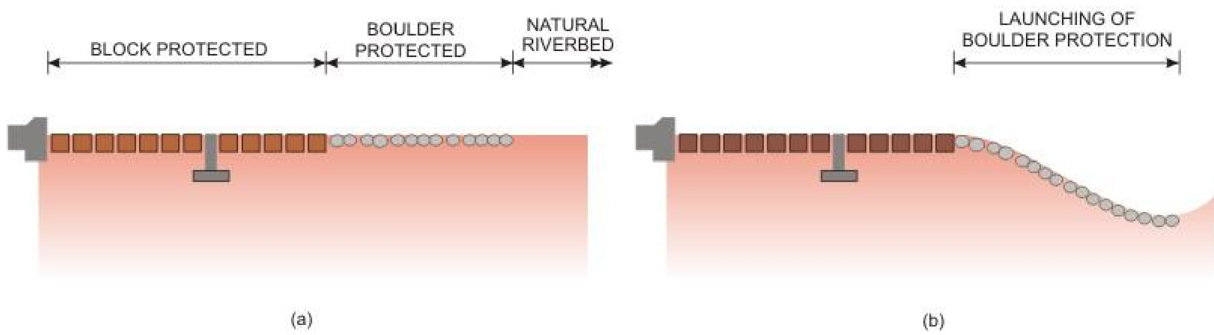


FIGURE 20. Section through downstream protection : (a) After initial laying ; (b) After scour of downstream riverbed due to passage of flood.

It may be mentioned that the loose stone protection shall have to be laid not only down stream of the barrage floor, but all along the base of guide bunds, flank walls, abutment walls, divide walls, under sluice tunnels, as may be observed from the typical layout of a barrage given in Figure 21.

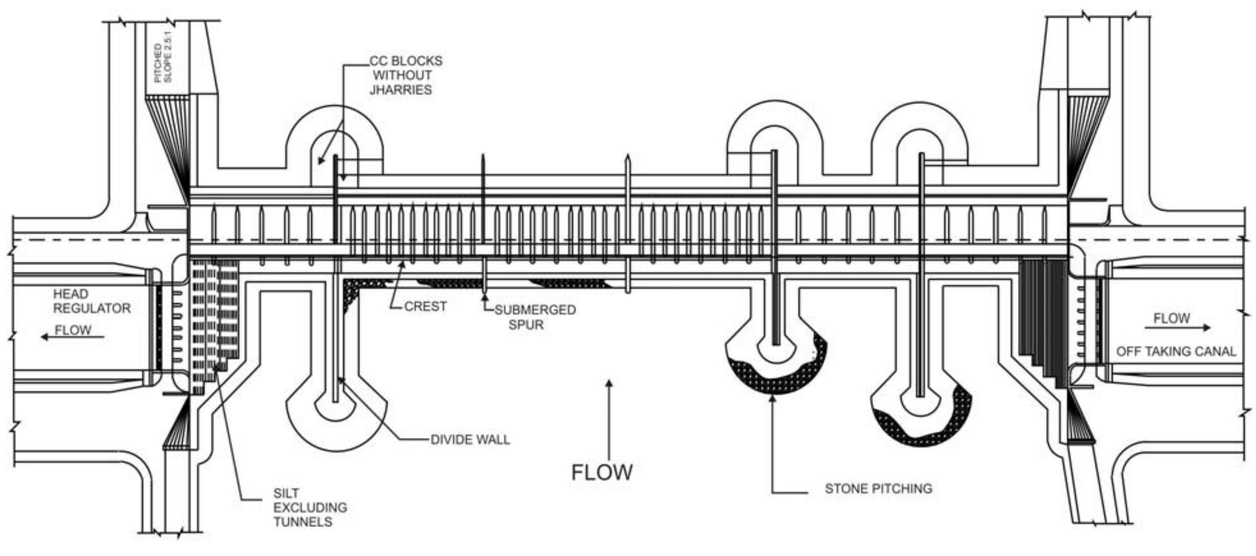


FIGURE 21. TYPICAL LAYOUT OF A BARRAGE AND ITS APPURTENANT STRUCTURES

Once the basic dimensions are fixed for all the barrage component, they are designed structurally, considering the forces estimated from the hydraulic analysis, The Bureau of Indian Standards code IS: 11130- 1984 "Criteria for structural design of barrages and weirs" specifies the recommendations in this regard, the important ones of which have been discussed below.

As the major portion of the barrage structure including the raft floor, piers, divide walls, under sluice tunnels, etc. is constructed as reinforced concrete structures, accordingly the general principles specified in IS: 456- 2000 "Code of practice for plain and reinforced concrete" shall have to be followed. Since most of the construction is likely to remain underwater, the minimum cover may be kept at 50mm for safety. Some other items, notably sheet piles, gates, gate groves, etc. have to be made of structural steel made, conforming to relevant Bureau of Indian Standards specifications. The important components of a barrage are discussed below with the specific structural requirements.

Cut-off (Sheet-pile)

The upstream and downstream cut-offs of a diversion structure may be steel sheet-piles anchored to the barrage floor by means of R.C.C. caps, or may be built of masonry or reinforced concrete. The sheet pile cut-offs are to be designed as sheet pile retaining walls anchored at the top. They shall be designed to resist the worst combination of forces and movements considering possible scour on the outer side, earth pressure and surcharge due to floor loads on the inner side, differential hydrostatic pressure computed on the basis of the percentage of pressure of seepage below floor etc. In case the effect of cut-offs is taken into account for resistance against forward sliding of the structure, the cut-offs shall also be designed to withstand the passive pressures developed. The R.C.C. pile caps shall be designed to transmit the forces and moments acting on the steel sheet piles to the barrage floor.

Impervious floor (also called solid apron)

Generally there are two types of floors, the first being called the Gravity type and the second as the Raft type. In the former, the uplift pressure is balanced by the self weight of the floor only considering unit length of the floor, where as the latter considers the uplift pressure to be balanced not only by the floor but also the piers and other superimposed dead loads considering a span as a unit. Contemporary designs of barrages have also been of the raft- type, and hence this type of construction is recommended and discussed in this session.
The thickness of the impervious floor shall be adequate to counter balance the uplift pressure at the point under consideration. The thickness of the downstream floor (cistern) shall also have to be checked under hydraulic jump conditions, in which case the net vertical force on the floor is to be found out from the difference of the vertical uplift due to sub-surface flow and the weight of water column at any point from above due to the flowing water.

The design of the raft has to be done using the theory of beams on elastic foundations and the following forces, or their worst combination has to be taken:

Beyond the block protection on the upstream and down streams of a barrage located on alluvial foundation, a layer of loose boulders or stones have to be laid, as shown in Figure 20(a). The boulder size should be at least 0.3m and should not weigh less than 40kg. This layer of boulders are expected to fall below at an angle , or launch , when the riverbed down stream starts getting scoured at the commencement of a heavy flood [Figure 20(b)]. The length of river bed that has to be protected with loose stone blocks shall be around 1.5D, where D is the depth of scour below average riverbed.

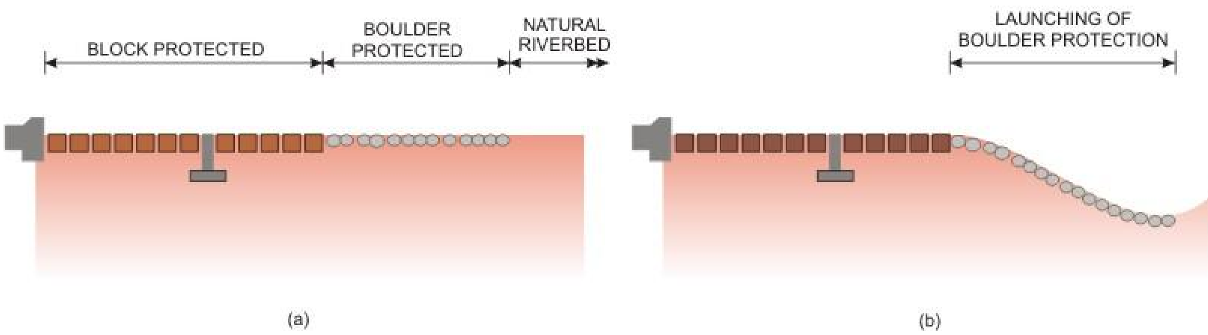


FIGURE 20. Section through downstream protection : (a) After initial laying ; (b) After scour of downstream riverbed due to passage of flood.

It may be mentioned that the loose stone protection shall have to be laid not only down stream of the barrage floor, but all along the base of guide bunds, flank walls, abutment walls, divide walls, under sluice tunnels, as may be observed from the typical layout of a barrage given in Figure 21.

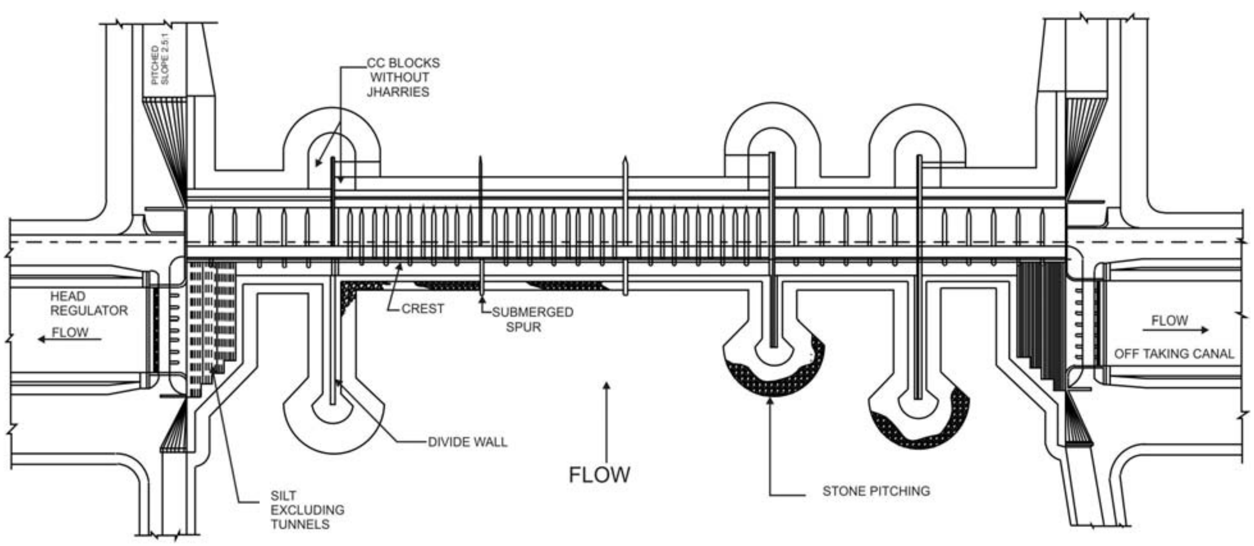


FIGURE 21. TYPICAL LAYOUT OF A BARRAGE AND ITS APPURTENANT STRUCTURES

4.2.6 Structural design of barrage components

- Buoyancy
- Wind forces
- Forces due to water current
- Differential hydrostatic pressure with the gate of one bay open and the adjacent bay closed
- Seismic forces, if any
- Hydro dynamic forces due to seismic conditions

A pier shall have to be designed as a reinforced concrete column and IS:456-2000 may be followed accordingly.

For the design of other components of a barrage project, like Divide walls, Abutments, Flank walls, Return walls, etc., IS: 11130-1984 may be followed.

4.2.7 Construction of concrete barrages

Barrages are nowadays made of reinforced concrete and designed as raft type structures which are light in weight compared to storage dams (designed as gravity-type structures). The design of barrages is done by accepting some calculated risks and hence it is important that the construction of such a structure is done with great care and there is no room for construction failure to occur. In this section, the important steps for a careful construction of barrage is explained and further details may be had from Bureau of Indian Standards Code IS:11150-1993 "Construction of concrete barrages – code of practice".

Data required for construction activities

For planning and execution of construction activities, a number of data is required, most of which would be available from the design reports. These include:

- Index map of the site
- Contour plan of the area
- Cross-sections of the river
- Bore-hole log charts
- Permeability coefficients
- Rainfall data of the location
- Flood discharges, minimum and maximum water levels
- Location and accessibility of quarry areas for coarse and fine aggregates
- Working drawings of barrage and appurtenant structures
- Sequence of construction of various blocks comprising of number of bays and abutments, etc.
- Requirements of inter-dependence of various items
- Necessary precautions to be taken
- Special features of construction, if any

Construction planning

The construction planning for any structure can be broadly classified under two heads: a) Infrastructure planning, and b) Procurement planning and is applicable to barrage construction also. It also includes the finalization of a programme of works and, intermediate review of the programme vis-à-vis the actual work going on in the field. These points are briefly explained below.

Infrastructure planning

This aspect of planning needs to ensure approach roads, power and water supply, workshop, stores, aggregate processing plant, concrete batching and mixing plants, camps and work sheds. It also requires the establishment of other amenities, such as market, schools, medical facilities, and other social and cultural needs of the field staff and workers. The planning should be carried out to the extent possible before the work starts, so that the uncertainties and delays in execution of work, and precise time estimates for the job planning could be evaluated.

Procurement planning

This requires the storage of various construction related materials, like explosives for blasting rocks, cement for construction, steel sheet piles, structural and reinforcement steel, aggregates, etc. The construction equipment necessary to execute the work have also to be procured along with adequate spare parts and accessories.

Programme of works

This should be prepared at the start of the construction activities and consist mainly of Bar Chart Programme for the project duration showing the quantities and monthly progress required for various major items of the project. Another master network plan based on PERT/CPM planning may have to be worked out for monitoring the project work. Based on these programmes, the planning for finance, manpower, equipment required for various activities in different seasons of work have to be prepared.

Review of programme and resources

This should be carried out from time to time as the construction work progresses and should compare items such as the budgeted programme of work and the actual programme of work reviewed at intervals of three or six months. Also, the actual performance of various machines have to be compared with the estimated performance and recommend necessary corrective measures that should be taken. Availability and procurement of essential materials like cement, reinforcement steel, sheet piles, etc. have to be reviewed as well as that for accessory and spare parts of plant and machinery in use and the availability of skilled and unskilled manpower.

Sequence of construction

This important activity has to be planned perfectly, since mistakes at this stage would be difficult to be rectified later. The major items under the sequence of construction are as follows:

● Layout of the barrage axis as per the approved plan by constructing short pillars called axis pillars at suitable locations along the line of the axis across the river.

● Benchmark location has to be established the entire project area to help site the various components like floor, crest, piers, etc. at proper elevation.

● Temporary access bridge has to be constructed for transporting men, material and equipment from one bank of the river to the other.

● Layout of cofferdams have to be decided on the site conditions, nature of river course, and programme of works for the season. Coffer dams are temporary structures constructed in the riverbed to provide an enclosed area where the actual construction might be executed. Details of the design of a cofferdam may be had from Bureau of Indian Standards Code IS:10084-1982(Part1) "Criteria for design of diversion works: Coffer dams".

● Once the coffer dams are constructed, the water within the enclosure has to be dewatered. The Bureau of Indian Standards Code IS:9759-1981 "Guidelines for dewatering during construction" may be referred for details, but the main points are noted below:

1. After completion of the excavation above the water table, dewatering of the foundations have to be commenced by well points or open pumps and the water table progressively lowered. Well point systems may be suitable for sandy soils but in silty clay foundations strata open pumps and/or deep well pump may be preferred. If an impermeable compact shingles-coffle layer is sand witched between sandy layers in the depth to be excavated, then deep-well pumps with strainer throughout its depth has to be used.

2. The preliminary requirements of dewatering pumps should be bored on the inflow to the work area, calculated on the basis of permeability of the strata and closeness of the water source.

3. During dewatering operation, care should be taken to ensure that there is no removal of fines from the sub-strata that may weaken the foundation.

4. Any seepage of water from the foundation at local points or springs have to be taken care of properly so that there is no piping of the foundation material.

5. Excavation of the foundation to the barrage profile is to be made either manually or by machines in reasonably dry conditions . During excavation,

water table should be maintained at a lower level at which the excavation is being done. The excavated soil should be disposed- off either manually or by machines, to suite the site requirements . In case machinery is employed, the final excavation of the lowest layer should be done manually to the specified depth.

6. Cutoff walls may be steel sheet -piles driven from riverbed in case of non-bouldery strata of riverbed but in bouldery strata, either concrete or steel sheet pile cut-offs have to be constructed, both by excavating a trench and then back filling with sand. For a discussion of the details of steel sheet pile driving or construction of cutoff walls in trenches the code IS:11150-1993 “Construction of concrete barrages - code of practice” may be referred to.

7. Once the cut-off walls on the upstream and downstream sides of the barrage are installed and partially covered with pile caps, the foundation surface of the raft floor has to be properly leveled, dressed and consolidated. The foundation should not contain loose pockets or materials and they should be watered and compacted to the specified relative density. Clay pockets should be treated as specified by the designer. It has to be ensured that proper drainage arrangements in the foundation according to the designs including inverted filter, wherever indicated, are provided and concreting work is taken up.

8. Instruments like piezometers, pressure cells, soil stress meters, tilt meters as specified should be installed carefully such that the electric or mechanical connections to a central control panel is least disturbed during construction.

9. The batching, mixing, placing and protection of concrete has to be done in accordance with IS:456-1978 “Code of practice for plain and reinforced concrete”.

10. Where mechanical parts like gate guidining rails, gate seals are to be installed, block outs should be left out so that the parts may be embedded later.

11. Dowel bars, or if necessary, metal sealing strips should be provided for the joints between the pile caps and barrage floor.

12. The sequence of construction of barrage bays, silt excluder and piers have to be done in lifts, starting from the downstream end of the barrage and with continuous pour in suitable layers, or as specified by the designer.

13. Abutment and flared out walls may be constructed on pile foundations or on well foundations.

14. Divide walls have to be constructed on well foundations and the wells have to be sunk to the founding levels and the work of barrage bays on either side of the divide wall should be taken up after construction of well caps.

15. The cement concrete blocks in the flexible apron on the upstream and downstream of the solid aprons of the barrage floor may be cast in-situ with alternate blocks cast at a time. These may be constructed with form work that should be so designed that when it is stripped off, the required gap is formed for filling the filler material to facilitate speedy construction, pre-cast blocks may be used.

Care and diversion of river

Since a barrage would be covering almost the entire width of the river, and it would take quite a few years to construct the whole structure, it would be necessary to construct only portions of the barrage at each construction season, when the flow in the river is relatively less. There may not possibly be any construction in the flood season. During the construction season, the river has to be diverted from the area enclosed for construction by suitable flow diversion works.

The programme of construction of river diversion work should mainly be determined by the availability of working period, likely time that would be required for construction of coffer dams, associated diversion works and construction capability.

The period available for construction of cofferdams is generally limited and depends upon the post-monsoon pattern of the river course and quantum of discharge and programme of work of various items of permanent nature. Cofferdam construction for the portions nearer to the river banks where velocities may not be high, may be of earthen type cofferdams and when the work advances into the river portion, composite type cofferdams consisting of single sheet piles backed with earthen embankments may be provided. Suitable protection on the river side has to be provided to avoid dislodging of sheet piles due to scour of soil backing. For details about the choice of coffer dam to be adopted, one may refer to the Bureau of Indian Standards Code IS:10084-1982 (part1) “Guidelines for choice of type of diversion works: cofferdams”.

Module
4
Hydraulic Structures for Flow Diversion and Storage

Lesson 3

Design of Barrage Appurtenant Structures and Rules for Barrage Operation

Instructional objectives

On completion of this lesson, the student shall learn:

1. The design steps for a canal head regulator
2. The need for sediment exclusion devices
3. Operation and regulation rules for barrages

4.3.0 Introduction

A barrage constructed across a river comprises of the main diversion structure to raise the river's water level and control its flow, but there are other components which comprise of the total barrage project, that are equally important. These are as follows:

1. Canal head regulator: The structure that regulates the water inflow to a canal.
2. Sediment exclusion devices: A structure that attempts to remove suspended sediment from the river water and helps in reducing sediment entry into the canal.
3. Gates and Stoplogs: Gates are used to control flow through the main diversion structure as well as through the head regulator. Stop logs are used for emergency closure of the flow through the bay of a barrage or a head regulator.
4. River training works: These structures are necessary to guide the water towards the barrage.
5. Navigation and fish passing facilities: The obstruction of a river caused by the construction of a barrage requires these structures for free passage of ships and boats or migratory fishes up and down the river.

Though all the above appurtenant structures are important, only the first two, that is, Canal head regulator and Sediment exclusion devices have been discussed in this lesson in detail, with references to the appropriate Bureau of Indian Standard codes. A list of the important standards for these and other components is given below:

	IS code number	***Name of code***
1	IS 6996 (Part 1): 1989	Hydraulic design of barrages and weirs – Guidelines
2	IS 7720: 1991	Criteria for investigation, planning and layout for barrages and weirs
3	IS 14955: 2001	Guidelines for hydraulic model studies of barrages and weirs
4	IS 11130: 1984	Criteria for structural design of barrages and weirs

5	IS 11150: 1993	Construction of concrete barrages – Code of practice
6	IS 13495: 1992	Design of sediment excluders – Guidelines
7	IS 6531: 1994	Canal head regulators – Criteria for design
8	IS 7495: 1974	Criteria for hydraulic design of silt selective head regulator for sediment control in off-taking canals

4.3.1 Design of Canal Head Regulator

As mentioned before, the regulator provided at the head of a canal off-taking from the pool behind a barrage is termed as canal head regulator. The structure has the following functions to fulfill:

- Regulation of supply of water into the canal or water conductor system for purposes of irrigation, hydroelectric power generation, industrial or domestic water supply, etc.
- Control of the entry of silt into the canal.

Hence it is very important that the design of the head regulator is made carefully for satisfactory hydraulic and structural performances. The various aspects of a canal head regulator's design is discussed in this section. The Bureau of Indian standards code IS:6531-1994 "Canal Head Regulators - Criteria for design" recommends the details for the layout and design of these structures and important aspects from this code have been included herein.

Location and layout

The location of a canal head regulator is interlinked with the location of the diversion work. The head regulator should be located as close to the diversion structure as possible and preferably at the end of the outer curve (convex bend), if available at same portion of the river. The general location of a head regulator in relation to the barrage may be seen from Figure 1.

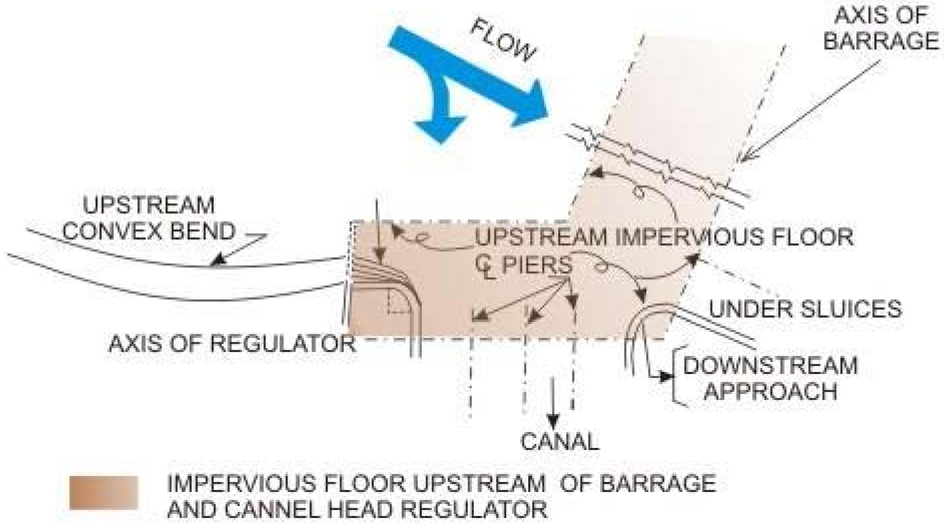


FIGURE 1. Typical location of a canal head regulator in relation to a barrage

Adapted from IS 6531: 1994 Canal Head Regulators - Criteria for Design

While the location of the head regulator adjacent to the abutment of the diversion structure is preferred, it may not sometimes be possible to locate it there due to topographical features such as hills, etc. In that case, the head regulator may have to be located upstream near the periphery of the pond, but not very far from the main structure. If the discharge requirements are small, sometimes the head regulator is provided in the form of an opening in the wing wall of the abutment.

The head regulator has to be properly aligned with respect to the barrage axis, so as to reduce the quantity of silt entering the canal and also to avoid back flow and formation of stagnant zones in the pocket. To achieve this, the axis of the regulator may be kept at an angle varying from 90^0 to 110^0, as shown in Figure 2.

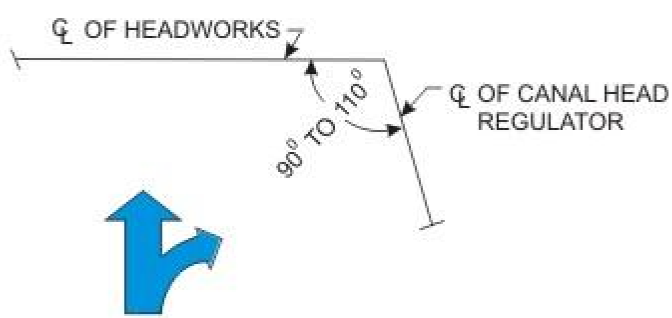


FIGURE 2. Relative inclination of head regulator & barrage axes

Though this angle recommended is preferred, the final layout is invariably tested in a model study that checks the different flow combination of the Undersluice gates and the canal regulator gates. A typical layout of a canal head regulator may be seen in Figure 3. A longitudinal section through the structure is shown in Figure 4 and one off-taking by the side of a sediment excluder is shown in Figure 5.

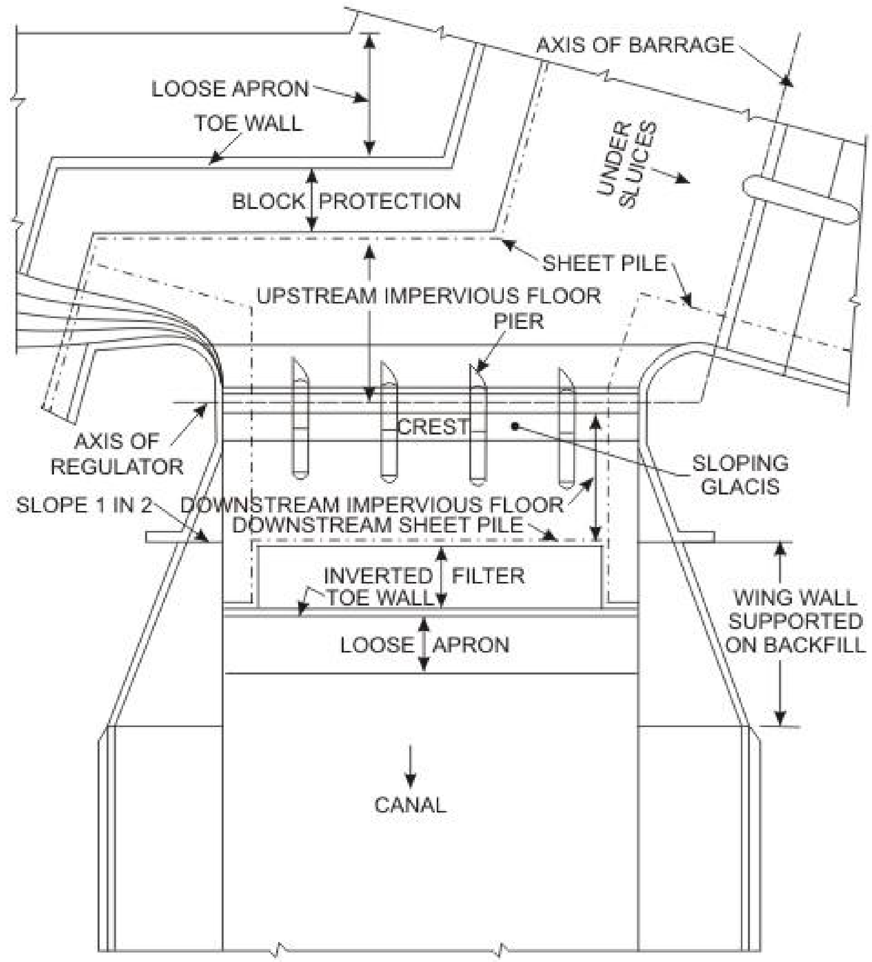


FIGURE 3. Typical layput of a canal head regulator
Adapted from IS 6531: 1994 Canal Head Regulators - Criteria for Design

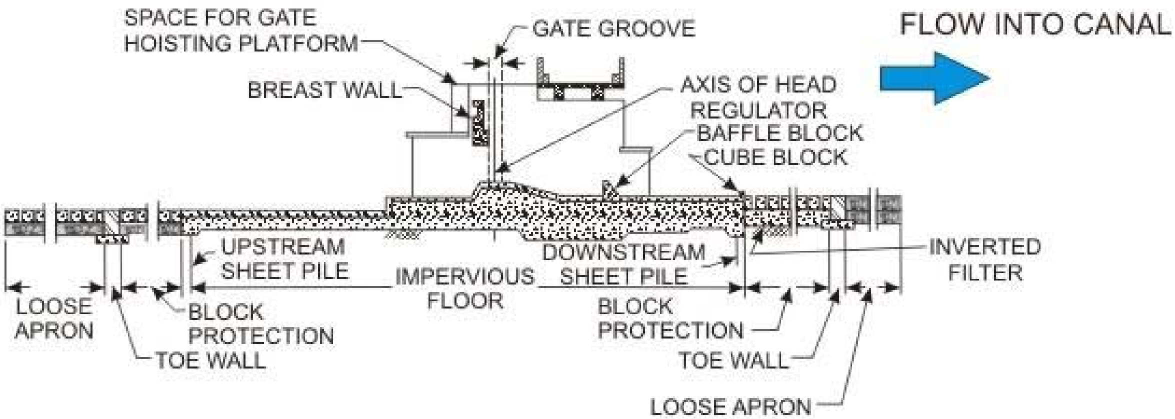


FIGURE 4. Sectional view through a canal head regulator
Adapted from IS 6531: 1994 Canal Head Regulators - Criteria for Design

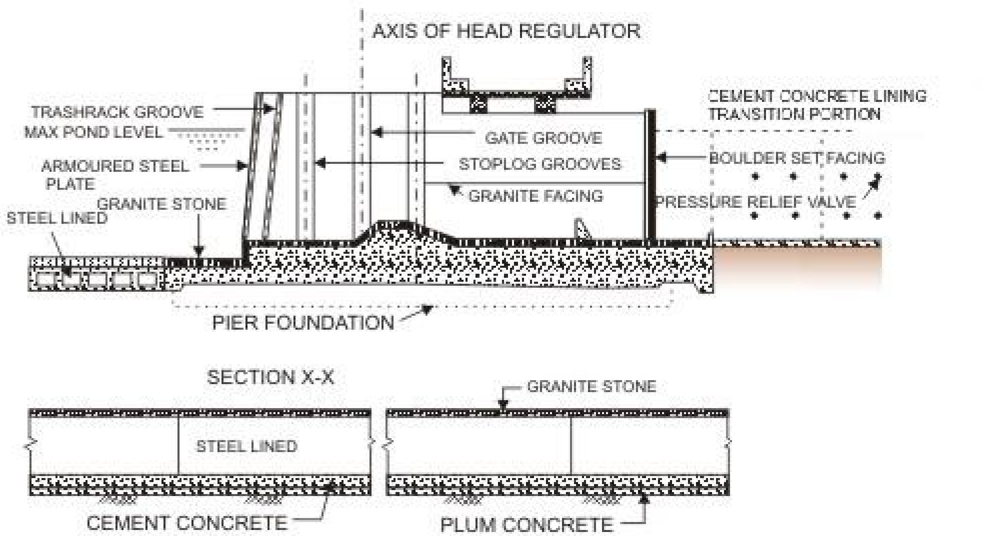


FIGURE 5. Canal head regulator section with Sediment Excluder
Adapted from IS 6531: 1994 Canal Head Regulators - Criteria for Design

The head regulator can be constructed independent of the abutment separated from it by suitable joints and seals or it can be made monolithic with it. The abutments of the head regulator themselves can be separated from its floor by longitudinal joints and seals or they can be made monolithic with the raft floor of the head regulator and the whole structure can be designed as a trough section (Figure 6).

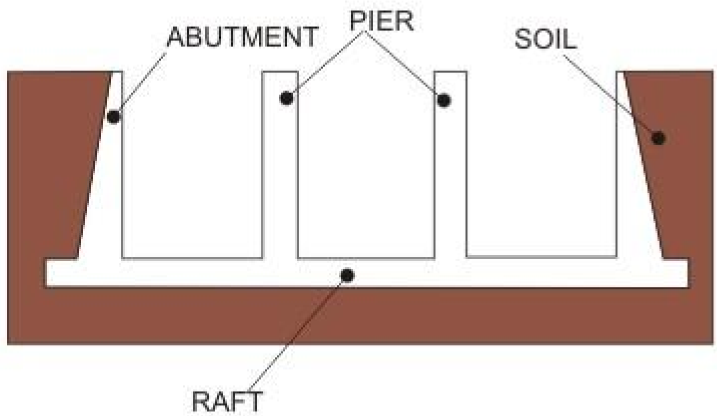


FIGURE 6. Head regulator as a monolithic trough section.

The regulation of water through a head regulator is provided usually by vertical lift gates. However, nowadays radial gates are also becoming common, though they are preferred for headworks having a relatively large difference in elevation between pond level and the canal full supply level which would ensure non-submergence of the radial gate's trunnion pin. Usually, a road bridge is also provided across a head regulator for vehicular traffic or for inspection purposes and would be suitably connected by road to the bridge across the main barrage structure. For the operation of the gates, a working platform across the head regulator has to be provided.

Hydraulic design

The hydraulic design of a canal head regulator consists of the following:

- Fixation of pond level of the pool behind the barrage
- Fixation of crest level, width and shape of sill
- Fixation of waterway, number and width of spans and height of gate openings, requirement of breast wall, etc.
- Shape of approaches and other component parts
- Safety of the structure from surface flow condition
- Safety of the structure from sub-surface flow conditions, and
- Energy dissipation arrangements

These aspects are discussed in the following paragraphs in detail

Pond level

The pond level in the undersluice pocket upstream of the canal head regulator may be obtained by adding the working head to pass the canal design discharge through the regulator with the water level in the canal at full supply level, and the head losses in the regulator. If under certain situations there is a limitation of the pond level, the full supply level should be fixed by subtracting the working head from the pond level.

Crest level, width and shape of sill

The sill crest level and waterway are interrelated. The sill level should be fixed by subtracting from the pond level the head over the sill that is required to pass the full supply discharge into the canal at a specified pond level. To obtain control on the entry of silt into the canal it is desirable that the sill of the head regulator be kept higher than the sill of the under-sluices, as much as possible, and at least by a difference of 1.2 to 1.5 meters. If silt excluders are provided, then the crest level of the sill should be kept at about 0.5 meters higher than the top surface of the silt excluders.

The required head over sill, **H**, for passing a discharge **Q**, with an effective waterway **L**, has to be worked out from the following formula, which is meant for flow that is uncontrolled (with out any gate control).

$$\mathbf{Q = C_d\, L_e H_e^{3/2}} \qquad (1)$$

Where $\mathbf{Q}$ = discharge in m3/s

$\mathbf{C_d}$ = coefficient of discharge

$\mathbf{L_e}$ = effective waterway

$\mathbf{H_e}$ = required head over crest for passing discharge Q, in meters

The coefficient $\mathbf{C_d}$ is not constant but depends on many factors (refer Figure 7) such as head above sill, shape and width of sill (**W**), upstream slope ($\mathbf{Z_u}$) and downstream slope ($\mathbf{Z_D}$) of the sill, height above the upstream floor (**P**) and roughness of the surface.

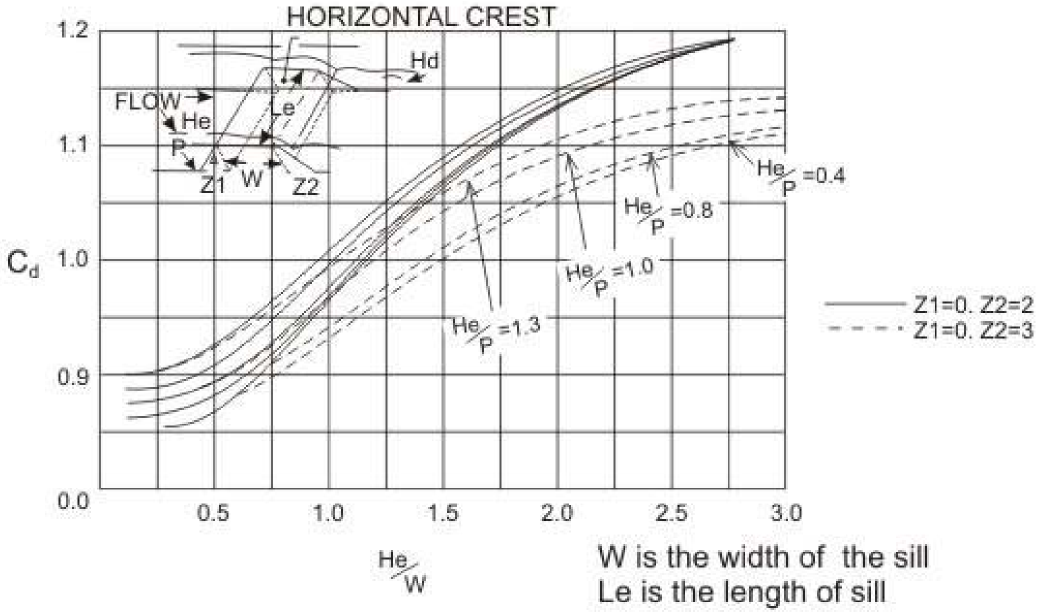


FIGURE 7. Recommended values of coefficient of discharge for varying He, P and W

Adapted from IS 6531: 1994 Canal Head Regulators - Criteria for Design

A typical set of curves for finding $\mathbf{C_d}$ at different values of $\mathbf{H_e/P}$ but for $\mathbf{Z_1}$=0 and Z2=2 is shown in Figure 7. Different sets of curves are available for $\mathbf{Z_1}$=0 and Z2=3, the details of which may be found in IS: 6531-1994. Of course, as the submergence increases, that is $\mathbf{H_d/H_e}$ tends to 1, the coefficient of discharge $\mathbf{C_d}$ also reduces. (Here, $\mathbf{H_d}$ is the downstream water depth above crest and $\mathbf{H_e}$ is the upstream total head above crest). The discharge reduction coefficients for various degrees of submergences are shown in Figure 8.

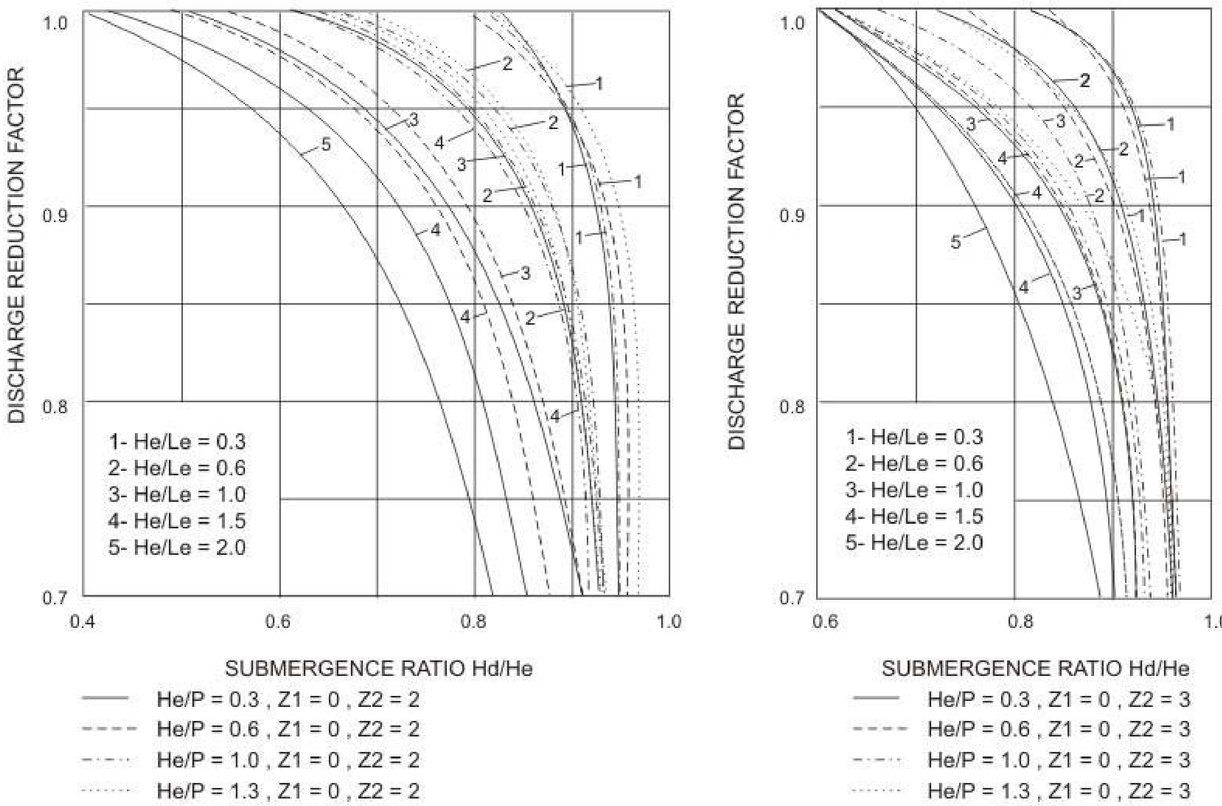


FIGURE 8. Variation of discharge reduction factor for submerged flow
Adapted from IS 6531: 1994 Canal Head Regulators - Criteria for Design

When the outflow is controlled by partial opening of the gates of the head regulator, the discharge formula for submerged sluice flow has to be used, which is as follows:

$$Q=2/3(2g)^{1/2}\, C_d\, L_e(H_1^{3/2} - H_2^{3/2}) \qquad (2)$$

Where **Q** = discharge (in m^3/s)

C_d = coefficient of discharge

L_e = effective waterway (in meters)

H_1 and **H_2** = total heads to the bottom and top of orifice

The width of the sill has to be kept according to the requirements of gates, trash rocks and stop logs subject to a minimum of **2/3 H_e**, where **H_e** is the total upstream head above crest.

The edges of the sill have to be rounded off with a radius equal to **H_e.** The upstream face should generally be kept vertical and the downstream sloped at 2H:1V or flatter.

Determination of waterway; number and width of spans

The waterway should be adequate to pass the required discharge through the head regulator without difficulty. After deciding the effective waterway the total waterway between the abutments including the piers have to be estimated from the following formula

$$\mathbf{L_t = L_e + 2(N\,K_p + K_a)\,H_e + W} \qquad (3)$$

Where

- $\mathbf{L_t}$ = total waterway
- $\mathbf{L_e}$ = effective waterway
- **N** = number of piers
- $\mathbf{K_p}$ = pier contraction coefficient
- $\mathbf{K_a}$ = abutment contraction coefficient
- $\mathbf{H_e}$ = head over crest, and
- **W** = total width of all piers

The recommended values of K_a and K_o have been shown in Figure 9.

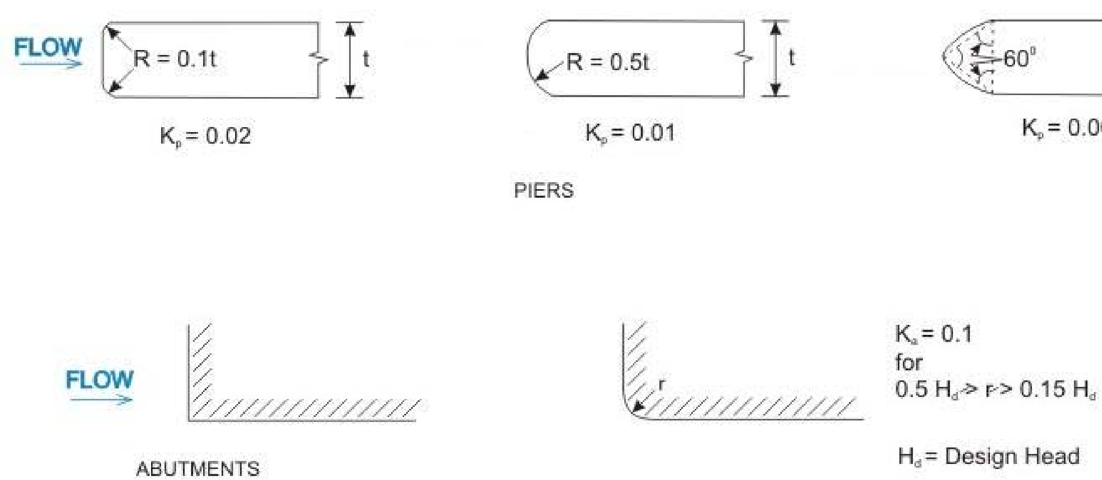


FIGURE 9. Recommended values of K_p and K_a

Shape of approaches and other component parts

The upstream inlet should be provided with circular, elliptical or hyperbolic transitions. The splay may be of the order of 1:1 to 3:1. At the downstream end, straight, parabolic or hyperbolic transitions may be provided with the splays ranging from 3:1 to 5:1 (Figure 10). All dimensions have to be tested in model studies for final estimates.

The wing walls should normally kept vertical up to the end of the impervious floor beyond which they should be flared from vertical to the actual slope of the canal section.

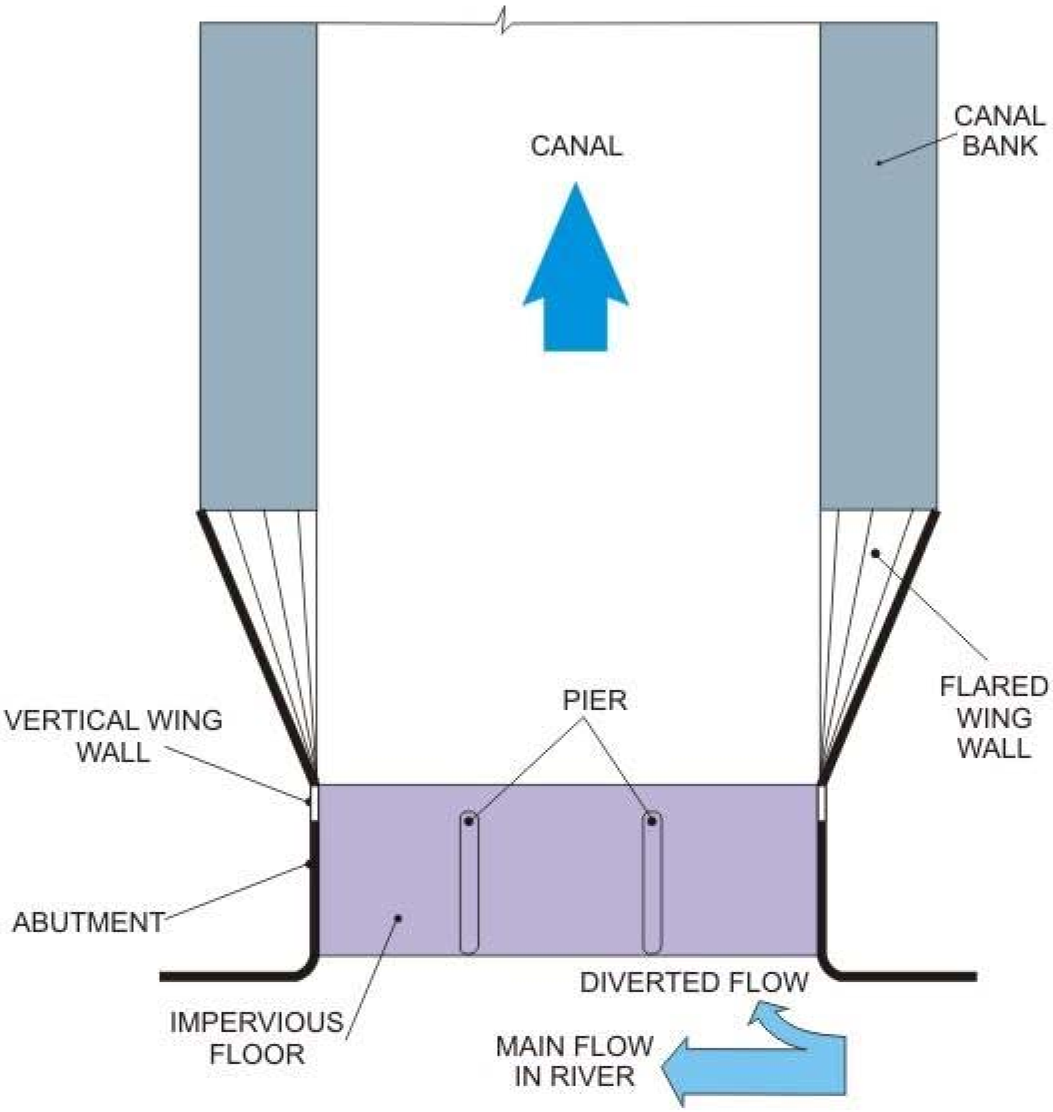


FIGURE 10. Transitions for upstream inlet

Safety of the structure from surface flow condition and energy dissipation arrangements

For head regulators located on non cohesive and erodible foundations, the unlined portion of the floor has to be protected against scour. However, if the head regulator is located on non-erodible beds, then these precautions may not be necessary.

On the upstream edge of the head regulator floor, a cutoff (or sheet pile) has to be provided and taken to the same depth as the upstream sheet pile of the main barrage structure (Figure 11).

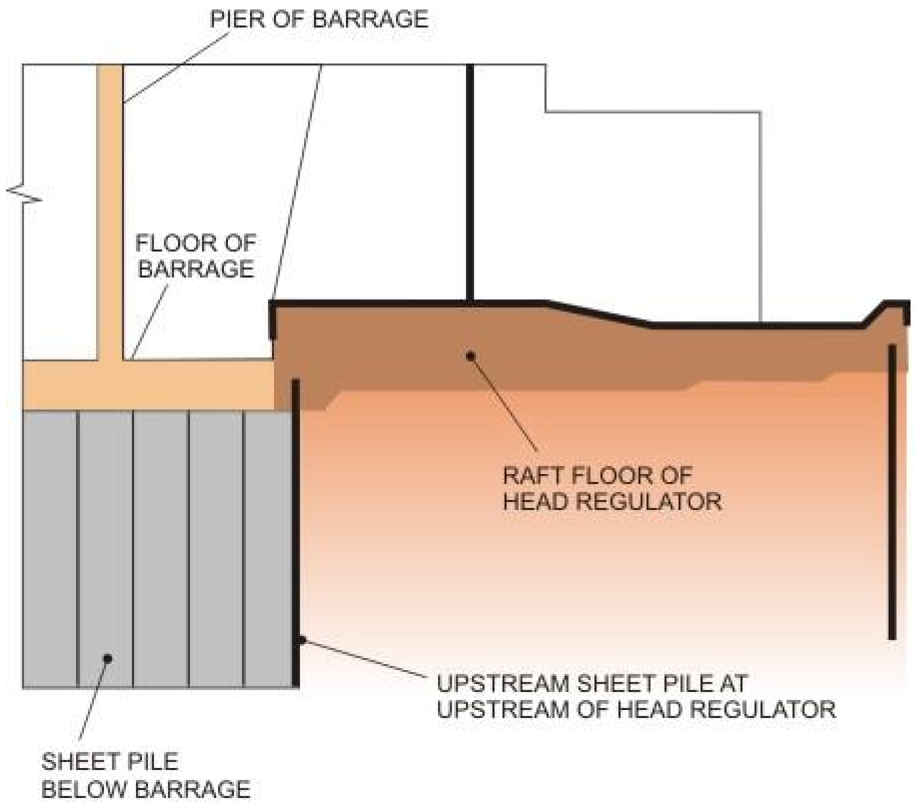


FIGURE 11. Sheet pile on the upstream of head regulator floor

On the downstream side of the sill, the head regulator shall have to be provided by a proper energy dissipating arrangement, which is usually done through the formation of a hydraulic jump for different discharge conditions. For various gate openings with pond level on the upstream (in the pool), the discharge through the head regulator have to be worked out. From these values, the cistern levels and lengths would have to be calculated and the governing values adopted for a profile. Additional energy dissipating devices such as chute blocks, friction blocks, end sill or dentated sill, etc. could also be provided wherever necessary. For head regulators with small discharging capacities, additional energy dissipating devices except an end sill may not be necessary. Details of energy dissipation devices based on hydraulic jump considerations may be had from Bureau of Indian Standards Code IS:4997-1968 "Criteria for design of hydraulic jump type stilling basins with horizontal and sloping aprons".

For evaluating the thickness of the floor of the head regulator, the hydraulic jump profiles for different flow conditions have to be plotted. Flow conditions would vary for different discharges in the canal with a corresponding gate opening of the regulator. The average height of the jump trough should then be obtained by deducting the levels of the jump profile from corresponding subsurface hydraulic gradient line. This will be taken as the unbalanced head for which safety of the floor (glacis, cistern) has to be ensured . As a rough guide, the unbalanced head may be assumed to be $\mathbf{1/2(d_2-d_1)}$

where **d_1** and **d_2** are the conjugate depths at the beginning and end of the hydraulic jump.

Safety of the structure from sub-surface flow condition

Similar to the design of the main barrage floor, the floor of a head regulator has to be checked for critical sub-surface flow conditions, if the head regulator is located on a permeable foundation (Figure 12). The factors that have to be checked are as follows:

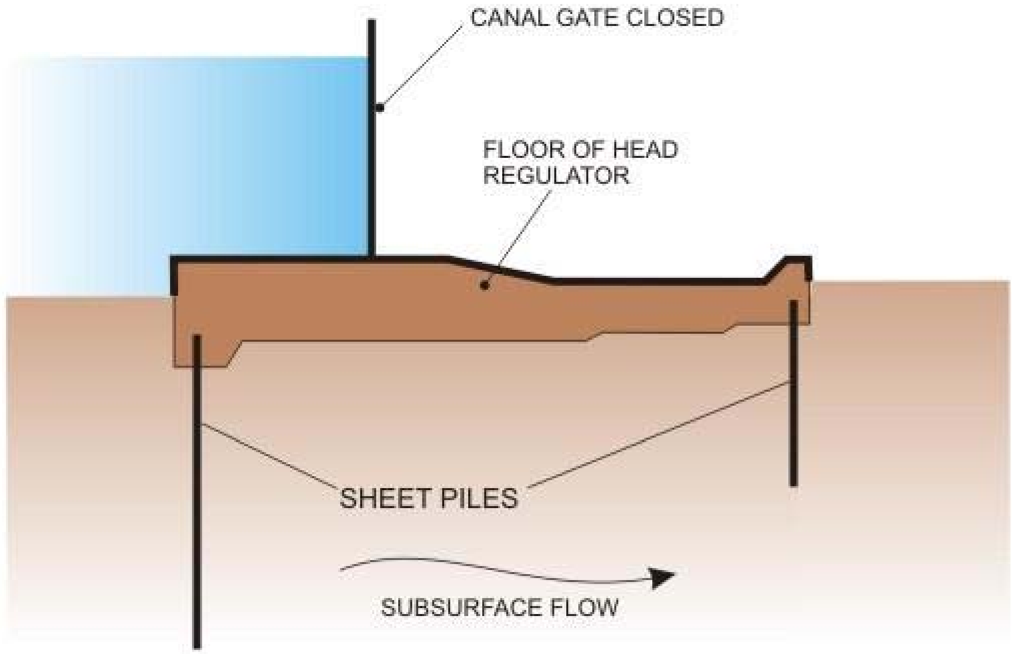


FIGURE 12. Critical subsurface flow for head regulator has to be checked for canal gate closed condition

The exit gradient for the upward rising seepage flow just downstream of the solid apron has to be determined by the formulae suggested in Lesson 4.2. The exit gradient has to be safe according to the type of bed material as per the guidelines given in that lesson.

The total length of solid floor and depth of downstream cutoff (or sheet-pile), which are inter-related, have to be determined from the conditions enumerated for sub-surface flow for main barrage floor in Lesson 4.2. However, it must be remembered that the total floor length can be decreased by increasing the depth of the downstream cut-off and vice versa, but increase in depth of downstream cut-off should result in increase in the concentration of uplift pressures, specially in the downstream half of the floor. A balance between the two should have to be arrived on the basis of economic studies.

Of course, the depth of downstream cutoff shall have to be worked out for the floor length decided to ensure safe exit gradient. If the depth of downstream cutoff so calculated is excessive, it can be reduced by increasing the upstream floor length.

Also, the uplift pressures at the key points on the floor have to be determined from the formula and graphs given in Lesson 4.2 corresponding to the condition of a high flood level in the river at the upstream of the head regulator and with the gates of the regulator all closed and no water in the canal downstream.

Just downstream of the solid apron of the head regulator, an inverted filter 1.5D to 2D long has to be provided, where D is the depth of scour below bed level determined from Lacey's formula, as for the main barrage structure given in lesson 4.2. This layer has to be overlain with 1.5/1.5/0.9m concrete blocks with open gaps in between the blocks and filled with coarse material like stone chips. The graded inverted filter may range in thickness from 0.5m to 0.8m and should conform to the following design criteria:

$$\frac{d_{15} \text{ of filter}}{d_{15} \text{ of foundation}} \geq 4 \geq \frac{d_{15} \text{ of filter}}{d_{85} \text{ of foundation}} \qquad (4)$$

In the above relation, $\boldsymbol{d_x}$ stands for the grain size than which $\boldsymbol{x}$ percent is finer.

Downstream of the inverted filter, loose apron 1.5D long consisting of either boulders weighing not less than 40 kg or gabions made of wire crated have to be provided.

On the upstream of the solid floor of the canal regulator, blocks and loose stone apron may be provided which should be similar to that provided upstream of the main barrage structure.

4.3.2 Sediment exclusion devices

A sediment exclusion device is provided as a part of the undersluice bays of the barrage floor in the river pocket adjacent to the head regulator to minimize sediment entry into the canal through the head regulator. As such, the excluders have to deal with alluvial materials such as boulders, gravel, and sand or silt depending upon the parent bed material and that which is being transported by the river. The sediment exclusion structures are necessary where excessive sediment entry into the canal is likely to cause its silting up and gradual reduction in flow conveyance. Streams carry most of sediment load of coarser grade near bed. If the bottom layers are intercepted and removed before the water enters the canal, then most of the sediment load can be withdrawn and prevented from entering the canal.

All rivers in the northern and eastern parts of India which originate from the Himalayas, which are geologically quite young mountains, flow quite fast in the upper reaches and carry with them heavy sediment load due to the comparatively soft hill formations. In the plains, their velocities are reduced due to the sudden changes of river bed slope and steep slope and as a result corresponding water surface slopes and the sediment load gets deposited. There is a low-lying plain of alluvial deposits of the Ganga -

Brahmaputra plains, the rivers which carry large sediment concentrations ranging from 2000ppm to 5000ppm by weight. The rivers of the southern peninsula such as Godavari, Krishna, Cauvery and their tributaries, which rise from the Sahyadri ranges on the south of Vindhyas flow through a plateau by hard volcanic rocks over a terrain with a hard, high and strong banks on either sides. As such, the sediment problem on these rivers is somewhat less compared to the rivers of the north.

The sediment excluding devices are not required to be provided in river pockets under following circumstances:

- If sediment trap like flood control dams are existing in the upper reaches of the river and there are no major tributaries in between the dam and the barrage downstream
- If the sediment load coming in the river reach upstream of the barrage is only wash load
- Wherever proper regulation of the undersluices are possible so that the incoming sediment load in the pocket could be flushed completely, and
- If sill of canal head regulator is 4m or higher above the floor level of the pocket, decision on the omission may be taken after the assessment on the basis of model studies.

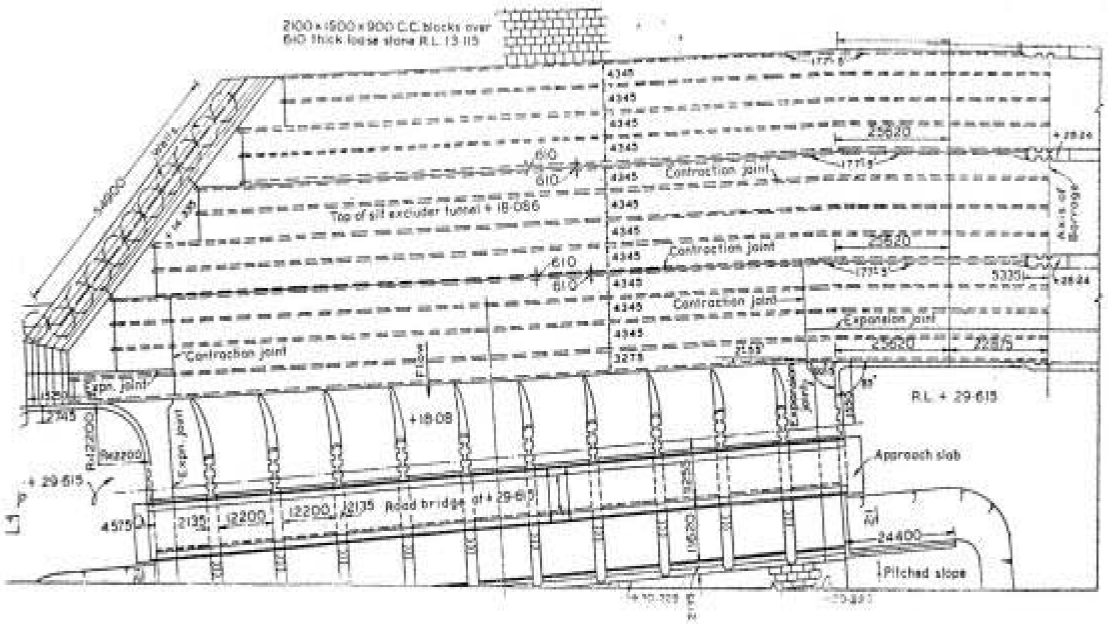
Sediment excluders are required to be provided in the river pockets of barrages when the following characteristics predominate:

- When there is high ponding upstream of the barrage to meet the canal discharge requirement
- When the river/tributary brings in sediment load of the order of 1500ppm and above, and contains significant percentage of coarse and medium sized sediment
- If the river is in the aggrading stage or wherever formation of bed bars/shoals is noticed due to unfavorable approach condition
- Bed building stage of the river may occur due to barrage obstruction to flow as well as improper regulation of barrage gates
- Due to adverse flow curvature upstream of the barrage head regulator, most of sediment load in high river stages may likely to settle in front of the head regulator and may enter its way into the canal
- In spite of suitable location of head regulator, river training measures for arriving at favourable curvature of flow, providing divide walls for separating pockets from barrage bays and suitable gate regulation of barrage undersluice bays for sand exclusion, a large quantity of coarse material may find its way into the pocket. In such cases for efficient working of canal, silt excluders are required to be provided
- Exclusion of gravels and boulders could be achieved by providing barrage crest at river bed level and ponding operation only during non-flood season. Similarly, features such as frontal intake, provision of river sluice, undersluices with low crests, provision of divide-walls their orientation and lengths, barrage gate regulation are some of the features depending upon sediment load and

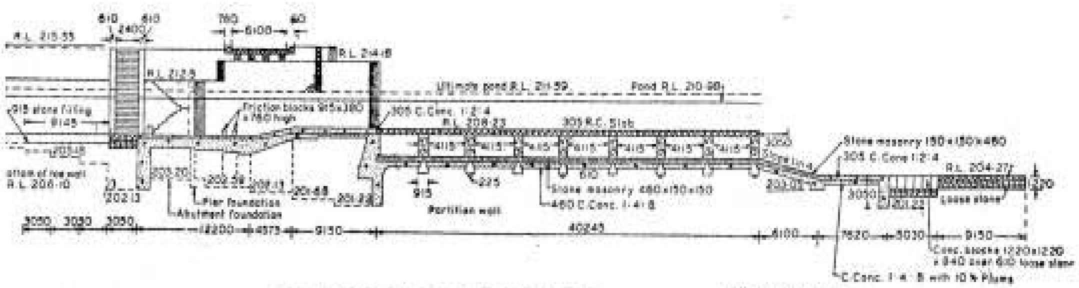
hydrograph characteristics of river as well as cut-off available and pond level to be maintained. Generally, gravels and boulders are excluded by keeping the crest of spillway bays low and suspended load and bed load is removed through special silt excluding structures.

The structure that is constructed in the undersluice portion of the barrages extending into the river pocket adjacent to the canal head regulator to minimize sediment entry into the a canal head regulator is done with the help of the divide wall which creates a relatively a quite zone (referred to as pocket) in front of the canal intake. The wall divides the stream flow as it approaches the barrages so that part of the flow is diverted to the spillways and part through the undersluice bays. The pocket acts like a ponding area of low velocity which allows much of the sediment to get deposited and flow downstream instead of getting into the canal. The sediment which gets deposited is taken downstream through the sediment excluders which are nothing but a series of tunnels made of reinforced concrete at the river bed level laid in front of the head regulator and ending at the undersluice bays.

There are, generally, two common types of layout for sediment excluders. The first type has the mouths of the tunnels that are laid staggered (Figure 13), and is sometimes called the Khanki-type, the name of which is derived from one of the projects where it has been used. The other type of excluder has all the tunnel mouths aligned at an angle in plan (Figure 14), which is called the Trimmu-type excluder.



ALL DIMENSIONS IN MM



ALL DIMENSIONS IN MM

Figure 13. Khanki type sediment excluder

(Image courtesy: Manual of Barrages, CBIP, 1985)

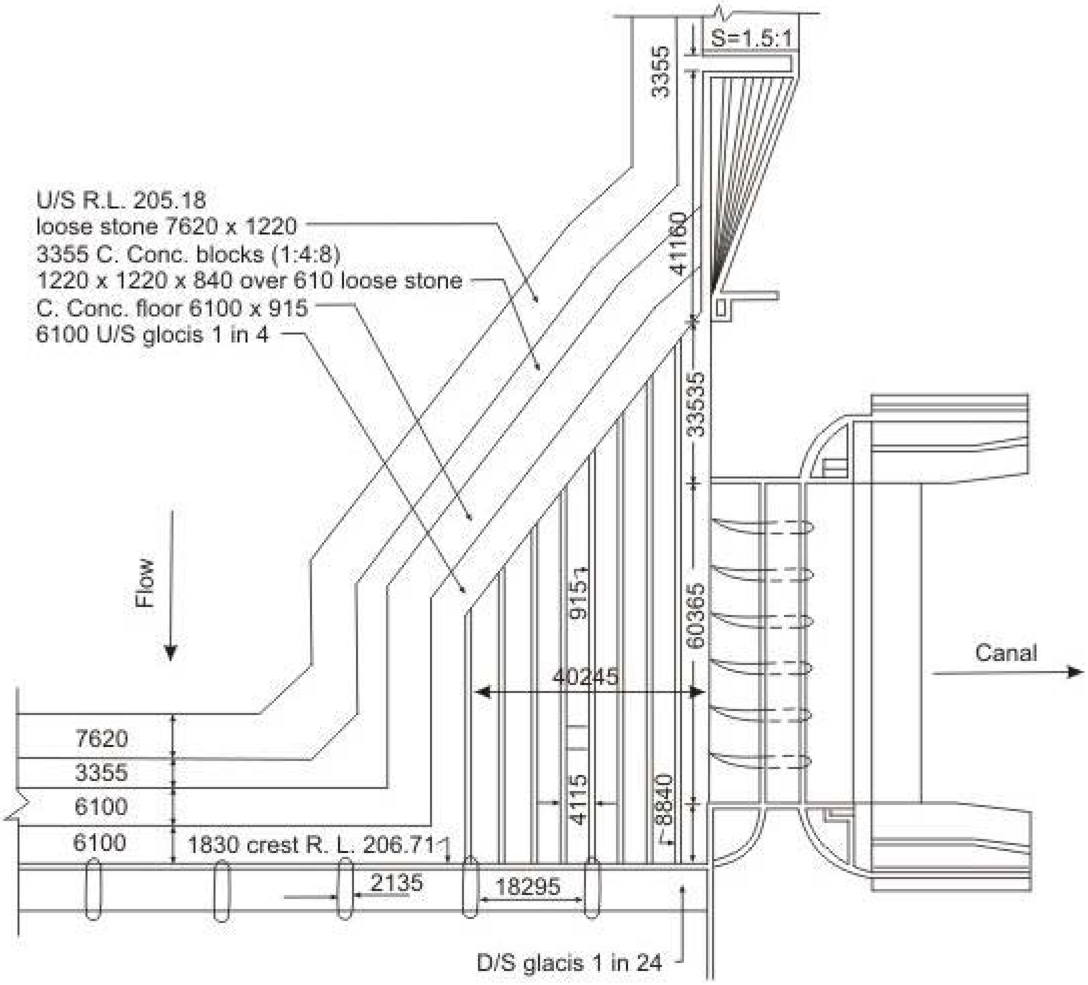


FIGURE 14. Trimmu-type sediment excluder
(Image courtesy: Manual of Barrages, CBIP, 1985)

A typical cross section of the excluder tunnels is shown in Figure 15, where the canal head regulator has been shown in longitudinal section, since it is at right angles to the excluder tunnels.

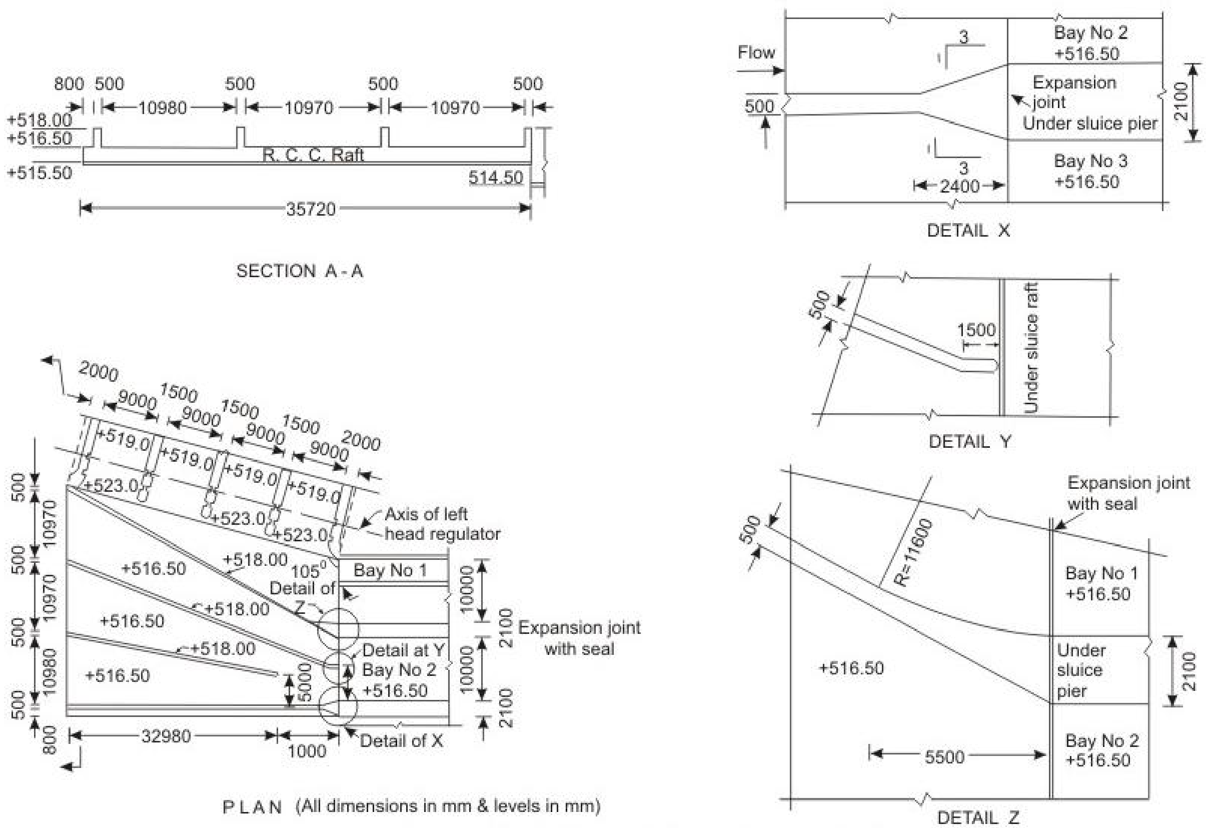


FIGURE 15. Sediment deflector for removing large boulders.
(Image courtesy: Manual of Barrages, CBIP, 1985)

Though both the staggered-entry and inclined-entry types of excluders have been used, there exist a few differences between them which may be considered while proposing a particular type. The following list enumerates the major comparative merits between the two.

- The tunnel mouths of the Khanki type being staggered may be extended to any length, each being independent of the other. The positions can be fixed to suit the approach conditions of the river. Hence, this type of excluder possesses a greater flexibility in tunnel arrangement compared to the Kalabagh type.
- The Khanki type of excluders is more sensitive to changes in river approach conditions and their performance may deteriorate if the river conditions keep changing with time. This is not the case with the Kalabagh type of excluders.
- The Khanki type of excluders is more complicated to construct as compared to the Kalabagh type.

Diversion structures constructed in mountainous regions may have large sized boulders rolling down the river and to remove them away from the head regulator, a structure in the form of a sediment excluder without the top slab has to be provided, which is called a Sediment Deflector as shown in Figure 16.

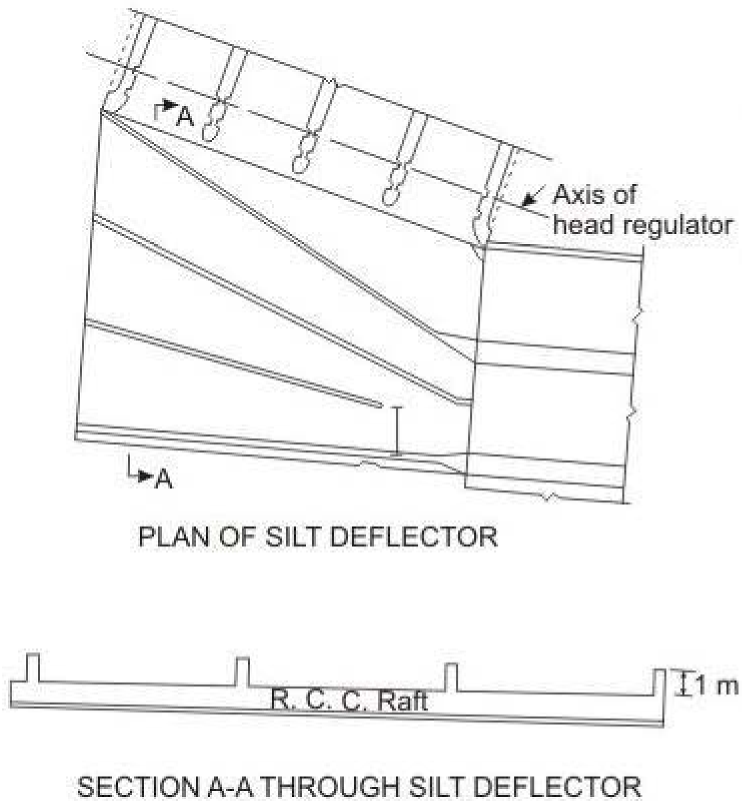


FIGURE 16. Silt / sediment deflector

The crest level of head regulators should be higher than the top of the deflector by at least 0.5m to 1m so as to prevent the entry of coarse silt into the head regulator.

For the design of the sediment excluders, the following parameters have to be decided:

• Number of undersluice bays of the barrage that have to be provided with sediment excluder tunnels
• Number of tunnels that are to be given for each bay of undersluice
•Location of the mouths of the tunnels
•The velocity and discharge that may have to be permitted through the tunnels.

Interested readers may refer Bureau of Indian Standards Code IS:13495-1992 "Design of sediment excluders - guidelines" for finalizing the tentative values of the parameters mentioned above apart from the layout, shape, longitudinal section and cross section of the tunnels. The final shape has to be decided from an appropriate model study.

4.3.3 Operation and regulation rules for barrages

A barrage is such a structure that has gates for controlling flow across almost the whole river section with their crest levels being very close to the riverbed. Hence operation of any gate or groups of gates not only affects the flow pattern in the upstream, that is, in the pool and in the downstream but also the river bed level changes associated with the changes in flow velocity. Further the undersluice bays have to be operated in such a way that there is not significant entry of silt into the off-taking canal. In order to prevent any unnatural flow behaviour and river morphological changes while satisfying the requirements of maintaining the pond level and prevention of sediment entry into the canal, a set of general guidelines have been formulated. Some of the important ones amongst these have been enumerated below:

1. The required pond level is to be maintained both during the non-monsoon flows and the falling flood periods.

2. The non-monsoon flows remain as far as possible near the undersluice bays so that feeding of the canal through a head regulator is not affected. In order to achieve this, therefore, most of the spillway bays are kept shut or opened very marginally. It is only the undersluice bays that operate and pass most of the river discharge which, in turn, creates a deep channel in the riverbed towards the bank where the canal is off-taking.

3. Though it is essential to draw water towards the canal head regulator side by operating the undersluices, it is also to be seen that a fairly uniform distribution of discharge takes place along the width of the barrage, as far as possible.

4. The gate operations should be such that the risk of deep scours or shoal formations (that is, deposition of sediments to form mounds) in the vicinity of the barrage both on the upstream and downstream is minimized, as far as possible . In order to achieve this, it is essential that the gate openings of adjacent bays should not be abruptly different.

5. A gate opening sequence has to be evolved such that deposition of silt and debris is avoided as far as possible on the upstream pool.

6. On the downstream of the barrage structure, the hydraulic jump should not be allowed to form beyond the toe of the downstream glacis.

7. A relatively high intensity of flow is to be avoided in the regions of deep scour, if any has been formed.

8. If a shoal has formed on either upstream or downstream it has to be washed out by an appropriate gate opening sequence.

9. The gate operation schedule should also consider the safe rate of lowering or rising of the pond level.

10. Constant and regular supply of water into an off-taking canal has to be ensured even though the discharge coming into the barrage pool may fluctuate as by the outflowing discharges of a power house located on the upstream side.

The operation and regulation of barrage gates can be divided into three distinct periods, as mentioned below:

1. Before monsoon (pre-monsoon)
2. During monsoon
3. After monsoon (post-monsoon)

The general gate operation strategies for these three periods are given below:

Pre-monsoon operation

This is a low flow period and wastage of water has to be avoided during this time, as far as possible. The barrage gates shall have to be regulated such that all the available supplies are conserved and pond level is maintained. Any excess flow over and above the requirements through the head regulators have to be released through the undersluice bays and silt excluder tunnels, wherever provided. The releases through the head regulator of the canal have to be based on the accepted discharge formula. For ready reckoning they are usually converted to discharge tables. These tables have to be occasionally checked for accuracy by taking actual measurement of flows in the canal. For any flashy flood, the canal may have to be closed temporarily, if the concentration of suspended sediment is in excess of the safe prescribed limits.

Monsoon operation

Gauges to indicate flood stage have to be installed sufficiently upstream (about a kilometer or more) of the barrage at suitable location so as to ensure adequate margin of time for operation of gates at the barrage site. During low floods, the gauges have to be signalled and recorded at every three hours while in medium and high floods, these shall be recorded every hour. The signaller at the head works, on receiving the flood warning shall communicate to the official of the headworks and other regulation points downstream. All these can be smoothly achieved if a wireless or telephone connection is established between the gauge reading point and the canal head works at barrage site. The water levels may also be the automatic floating type whose signal can be electrically transmitted to the barrage regulation point.

In order to create most favourable conditions for sediment exclusion from the canal, “Still-Pond” regulation have to be adopted, as explained below. However, in locations where the canals cannot be closed for silt removal, “semi-still-pond” regulation have to be adopted. These two modes of operation are explained below.

Still pond operation

In still pond operation, all the gates of the undersluice bays have to be kept closed so as to limit the discharge flowing into the pocket to be equal to the canal withdrawal. The specified or required discharge only should be drawn into the canal and the surplus river discharge should be passed through the spillway bays or river sluice bays, if provided. As the undersluice bays are kept closed, the low velocity in the pocket causes the sediment to settle down and relatively clear water enters the canal. However, the pocket gets silted up in this process after sometime.

At, that time, the canal head regulator gates should be closed and the deposited silt should be flushed out by opening the gates of the undersluice bays. The canal supply may be stopped during this scouring operation which may take about 24 hours. After the deposited silt has been flushed out sufficiently, the head regulator gates should be opened and undersluices closed. This operation is desirable where the crest of the head regulator is at a sufficiently higher level than that of the upstream floor of the undersluice bays. This still pond operation should be continued till the river stage reaches the pond level after which the undersluice gates should be opened to avoid overtopping.

Semi-still-pond operation

In the semi-still-pond operation, the gates of the canal head regulator are not closed for flushing of the silt deposited in the pocket. The gates of the undersluice bays should be kept partially open to the minimum necessary so that the bed material in the pocket could be passed downstream. The discharge in excess of the canal requirement should be passed through the undersluice bays and silt excluder tunnels, wherever provided.

During the monsoon months, it is important to keep a constant watch over the sediment entering the head regulator, a portion of which may have to be discarded through a sediment-extractor, if any, provided within the canal. Further, it may have to be ensured that sediment deposition takes place only to the extent that can be washed out early in the cold weather before the full demand develops. For these conditions to satisfy, the following actions may be necessary:

1. Sediment charge observations for both suspended sediment and bed load have to be made at least once a day in low floods immediately below the head regulator, below the silt ejector, if any, and at any other sensitive point lower down the canal. The frequency of observations may have to be increased in medium and high floods as required.

2. The cross section of the canal shall have to be taken at a few critical points to keep a watch on the extent of sediment deposition in the canal.

3. Water surface slopes at the critical points in the head reaches of the canals have to be kept under observation with the help of gauge observations of water levels.

4. The ponding upstream of power stations, for the case of power channels, shall have to be restricted to the requisite extent so as to avoid harmful sediment deposition.

5. The canal may have to be closed from the head under the following situations:

- Beyond a specified sediment charge during medium or high flood and re-opened when the sediment charge drops down below the specified limit. Since the silt carrying capacity of the canal would govern the specified limit, it would vary from project to project and should be estimated based on actual data or experience of the engineer.

- When sediment deposition at the critical points has reached the maximum permissible bed level. This limit along with the sediment charge in excess of which the canal is to be kept closed, may have to be fixed for different months during the monsoon period in order to be able to meet the irrigation or power demands.

Since cross flows and vertex formations dangerously cause deep scours both on the upstream and downstream of the barrage leading to washing away or sinking of cement concrete blocks and loose stone aprons, and damage to the nose and shanks of guide bunds, visual observations of the direction of current and vortex formation during low and medium floods should be made. After critically observing the effects of different patterns of gate operation on the formation of vortices, the engineer-in-charge would have to judiciously select the correct pattern which cause only minimum scour and minimum shoal formation.

The engineer-in-charge shall also have to monitor the shoal formations, changing network of spill channels, etc., which cause unequal distribution of flows through different bays, cross flow near the barrage floor ends, etc. The shoal formations quite close to the barrage may be washed out by judicious gate operation strategies.

The pond level has to be kept at the minimum required to feed the canal with the required discharge by suitably operating the gates. If a higher pond is maintained, then the extent of shoal formation would increase.

Post-monsoon operation

The sediment concentration observations and cross section of the critical points on the canal have to be continued but at less frequent intervals till satisfactory conditions have been established. Still or semi-still pond operation, with sediment excluders or sediment extractions, depending on the surplus water available, have to be continued till the water becomes reasonably clear.

When a canal is first opened, a low supply have to run for a few hours at least and the depth should gradually be raised according to the requirements. The rate of filling and lowering of the canal should be prescribed and these should not be transgressed.

If a study of the survey data indicates that shoal formation has occurred on the upstream and/or on the downstream of the barrage in spite of a judicial operation of gates, during normal and flushing operation of the pool, the shoal have to be removed by dredging to the extent possible so that satisfactory flow conditions are established and also the desired capacity is restored.

Satellite imageries may be studied to detect significant changes of the bank-lines for over the past years and remedial measures taken to improve the river behaviour.

Module
4
Hydraulic Structures for Flow Diversion and Storage

Lesson 4

Structures for Water Storage – Investigation, Planning and Layout

Instructional objectives

On completion of this lesson, the student shall learn:

1. The different types of storage structures, that is, dams
2. The decisive factors for choosing suitable location for a dam
3. The criteria for selecting a particular dam type
4. The important appurtenant structures for a dam
5. Typical layouts of dams

4.4.0 Introduction

In Lesson 4.1, the two primary types of valley structures, ***storage*** and ***diversion*****,** were discussed. These are again displayed in Figure 1 for recapitulation.

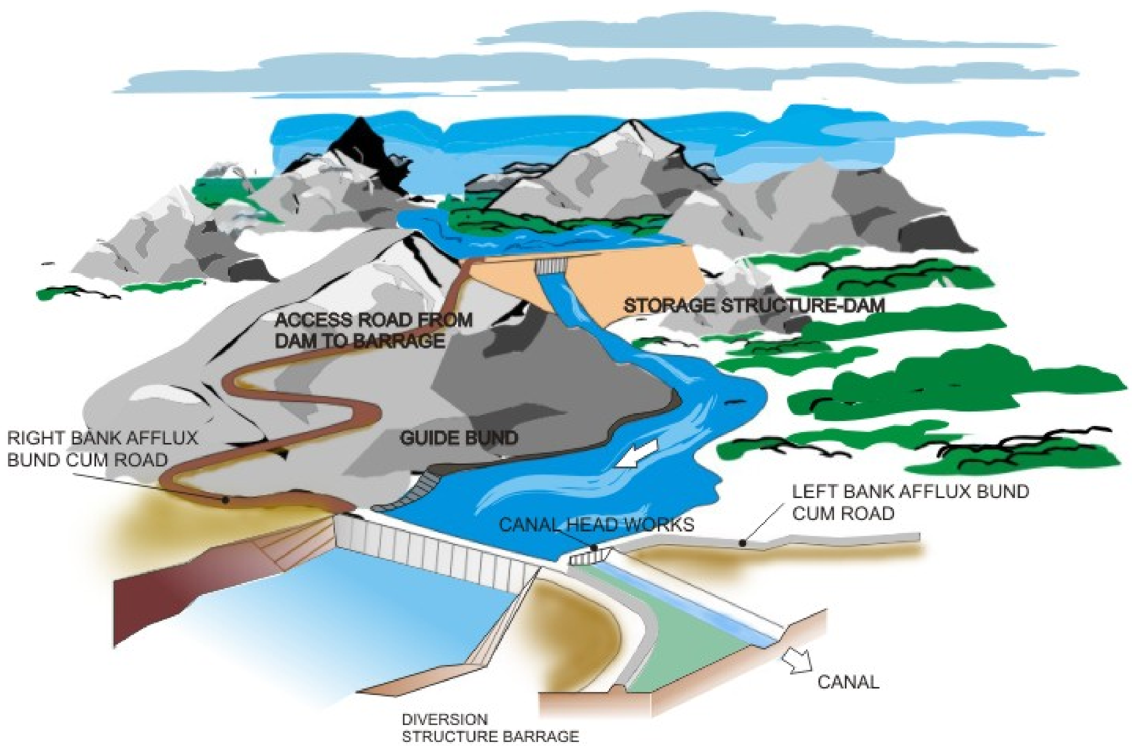


FIGURE 1. Structures for harnessing water resources potential of a river

Structures that are created as obstructions across rivers with an intention to store some of the water for future use are called storage dams. They are functionally slightly different from the structures used for flow diversion, called Barrages or weirs discussed in the Lessons 4.1 to 4.3. A diversion structure is primarily meant to create an elevation rise of the river water such that it may flow into a canal, perhaps through out the year or at least during the lean flow period of the river. During high flood in a river, the canal is kept closed in cases like irrigation canals for fear of high sediment entering and depositing on the canal bed. Even if some water is drawn into a canal during river flood season, the main bays of a diversion structures are usually kept wide open to let the flood water pass down the river with the minimum obstruction.

This is not so in case of storage dams, for which the storage of water especially a portion of the flood flow is primary concern. The storage of water is done with an intention to either reduce the impact of a flood down stream or to use the water beneficially in future. This is achieved by creating an obstruction of sufficient height which creates a reservation on the up stream of the structure. Naturally, since the reservoir would have a finite capacity which would vary with height of the dam and the shape of the river valley on the up stream, any excess flood water has to be discharged through a spillway. Hence, the principal components of a storage dam would be a storage a structure to obstruct river flow, a spillway for discharging excess flood water and outlets for allowing the storage water to be with drawn for the rest of the year for some useful purpose or even let it flow downstream at a regulated quantity. Some times sluice are provided in the body of the dam to lower the water level in the dam at the time of an emergency it is not necessary that all the principal components of a storage dam be located in the same structure. In fact, all three may be located separately. Of course, the spillway is usually made of reinforced concrete and sometimes combined with a concrete dam. But it may be economical or practically more feasible to construct an earthen or rock fill dam, in which case there has to be separate spill way made of concrete. In fact, it is also not essential for the spill way to be adjacent to the main dam, and can be located any convenient position at the periphery of the reservoir, if that helps in some way. Similarly, out let works may be located at any suitable place in the reservoir and possibly connected to a canal or a tunnel.

Sometimes dams are constructed to create a head difference for generating power but without using the storage. This may be done due to the requirements of the riparian rights, as in the case of the Salal Dam on Sutlej, which uses the head available to generate power put does not have an outlet or river sluice. Other types of dams include detention dams which are primarily created to retard flood runoff and control flood peaks.

In this lesson, we shall discuss about the types of dams in vogue, geotechnical and other site investigations that is required for planning the most suitable type of dam and the concept of general layout of an entire dam project including its various appurtenant structures.

4.4.1 Types of dams

Almost each dam that has been constructed all over the world is unique. This is so because a particular type is chosen because of the considerations of many factors, as discussed in subsequent sections. In fact, dam engineering brings together a range of disciplines, like structural, hydraulics and hydrology, geotechnical, environmental etc.

Never the less, primary purpose of a dam is to provide for the safe retention and storage of water. Structurally, a dam must be stable against overturning and sliding, either or within the foundations. The rock or soil on which it stands must be competent to withstand the superimposed loads without crushing or undue yielding. The reservoir basin is created must be watertight and seepage through the foundation of the dam should be minimal.

Though each situation demands a unique proposal for the type of dam, a broad classification based on the construction material can be made in dividing the types of dams that have been commonly constructed as:

1. Embankment dams, which are constructed of earth fill and/or rock fill, and

2. Concrete dams, which are constructed of mass concrete.

Of course, there are some dams that were constructed using rubble masonry, like the Nagarjuna Sagar dam on the river Krishna. But mostly embankment dams are more common for technical and economical reasons all over the world, they account for nearly 80 percent of all the large dams that have been built in modern times. The two main types of dams are further explained in the following paragraphs.

Embankment Dams

These can be defined as dams constructed of natural materials excavated or obtained from the vicinity of the dam site. The materials available are utilized to the best advantage in relation to their characteristics as an engineered bulk fill defined zones within the dam section. The natural fill materials are placed and compacted without the addition of any binding agent. Two main types of embankment dams that are commonly constructed include the following:

1. Earth-fill or earthen embankments –These may be classified as dams use compacted soil for constructing the bulk of the dam volume. An earth fill dam is constructed primarily of selected engineering soils compacted uniformly and intensively in the relatively thin layers and at a controlled moisture content. Some of the common sections designed for earth fill embankment dams are shown in Figure 2.

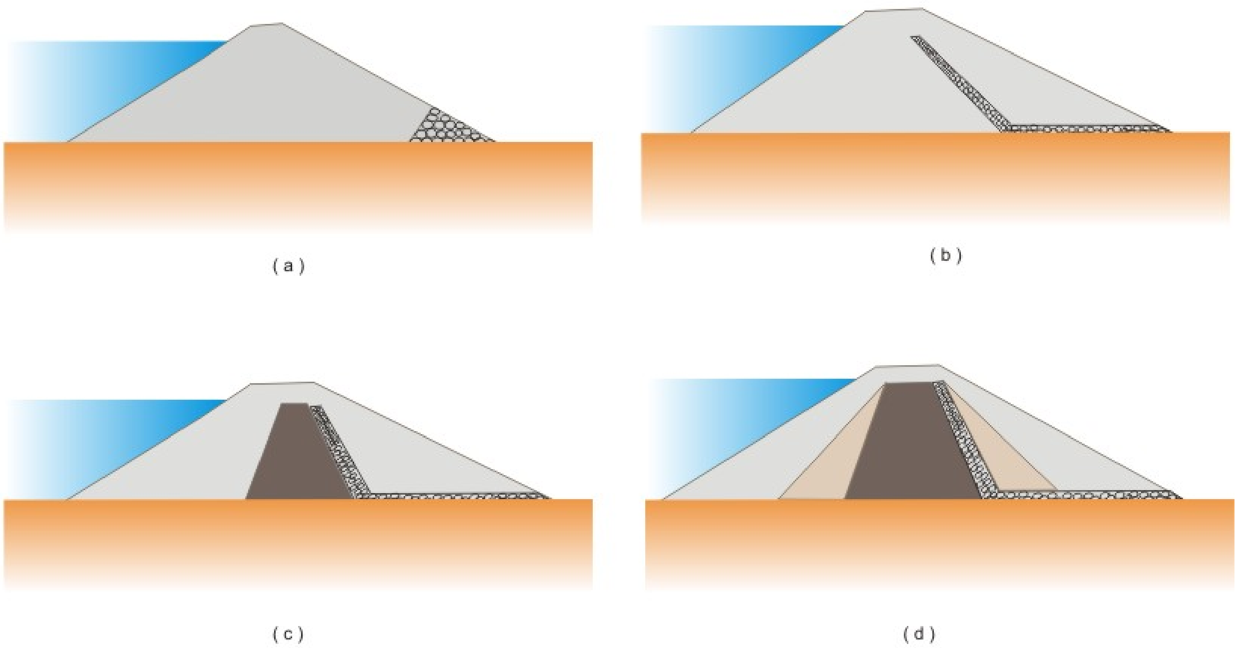


FIGURE 2. Principal types of earthen embarkment dams
(a) Homogeneous with toe drain,
(b) Homogeneous with chimney drain & horizontal blanket ,
(c) Zoned with clay core chimney drain & horizontal blanket ,
(d) Zoned with earth & rockfill

2. Rock-fill embankments – In these types of dams, there is an impervious core of compacted earth fill or a slender concrete or bituminous membrane but the bulk of the dam volume is made of coarse grained gravels, crushed rocks or boulders. Typical sections of rock fill dams are shown in Figure 3. The stability of the outer shell of a rock fill dam relies on the frictional forces acting in between each rock gravel piece which ensures its safety against sliding kind of failure during earth quakes.

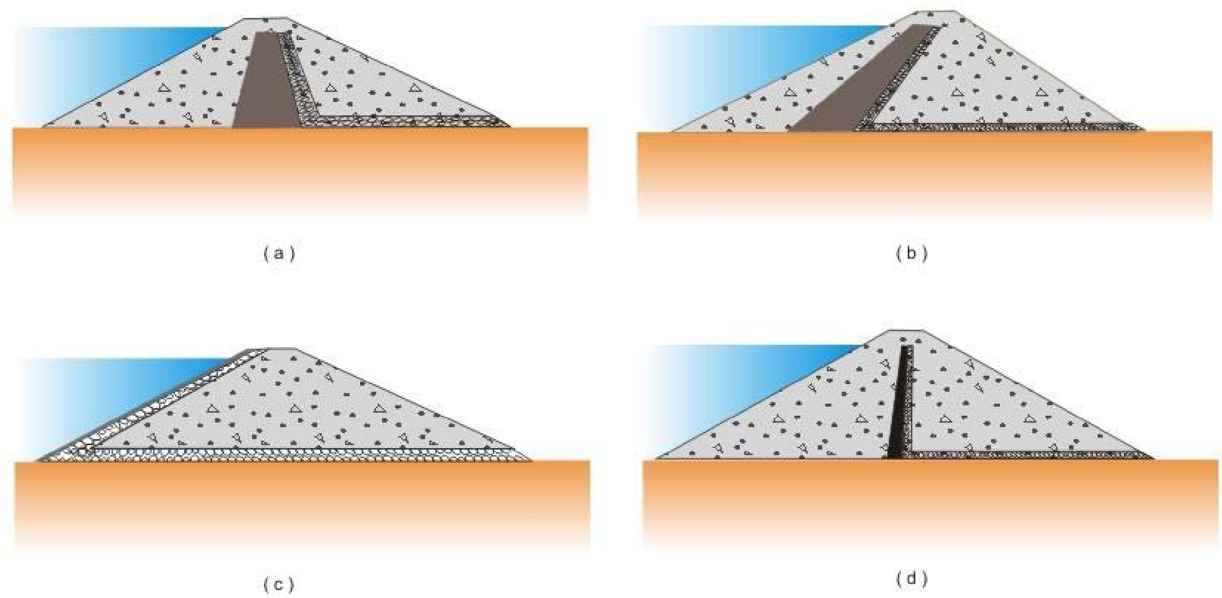


Figure 3. Principal types of rockfill embankment dams, with the following features:
(a) Vertical clay core & drains, (b) Inclined clay core & drains,
(c) Upstream decked with asphaltic or concrete membrane & drains ;
(d) Central asphaltic membrane with drains.

It may be observed that shell of the rock fill dam is more permeable than that of an earth fill dam. Mostly, the water is prevented from flowing down by the impervious core of the rock fill dam.

Embankment dams are advantageous in the following major aspects:

1. These are suitable for river valleys of any type: steep gorges or wide valleys
2. Can adapt to a broad range of foundation conditions, ranging from good rock to even permeable soil type of foundation
3. Uses naturally available materials
4. Relatively less costly

Amongst the disadvantages, it may be raid that they have greater susceptibility to damage than concrete dams due to the possibility of getting washed away during an over tapping of the spill way which may occur if there is a flash flood in the river and the gates of the spill way are not operated in time or the spill way itself is of inadequate capacity.

A further disadvantage of the embankment dam when compared to a concrete dam is that the former requires to have a separate spill way in contrast to the latter, where the spill way may be integrated within the dam body itself. Also earthen embankments are prone to concealed leakage, perhaps due to faulty construction or internal erosion in the dam body or in a pervious foundation.

Concrete Dams

The use of mass concrete in dam construction started from about 1900 for reasons of ease of construction and to suit complex designs, like having spill way within the dam body. From about 1950 onwards, mass concrete came to be strengthened by the use of additives like slag or pulverized fuel ash, in order to reduce temperature induced problems or avoid undesirable cracking or to reduce the total cost of the project.

Amongst concrete dams, too, there are many varieties, the principal types of which described below.

1. Gravity dams (Figure 4a): A gravity dam is one which depends entirely on it's own mass for stability. The basic gravity profile is triangular in shape, but for practical purposes, is modified at the top. Some gravity dams are slightly curved in plan, with the curvature being towards the river upstream. It is mostly due to aesthetic and other reasons, rather than having an arch action for providing greater stability.

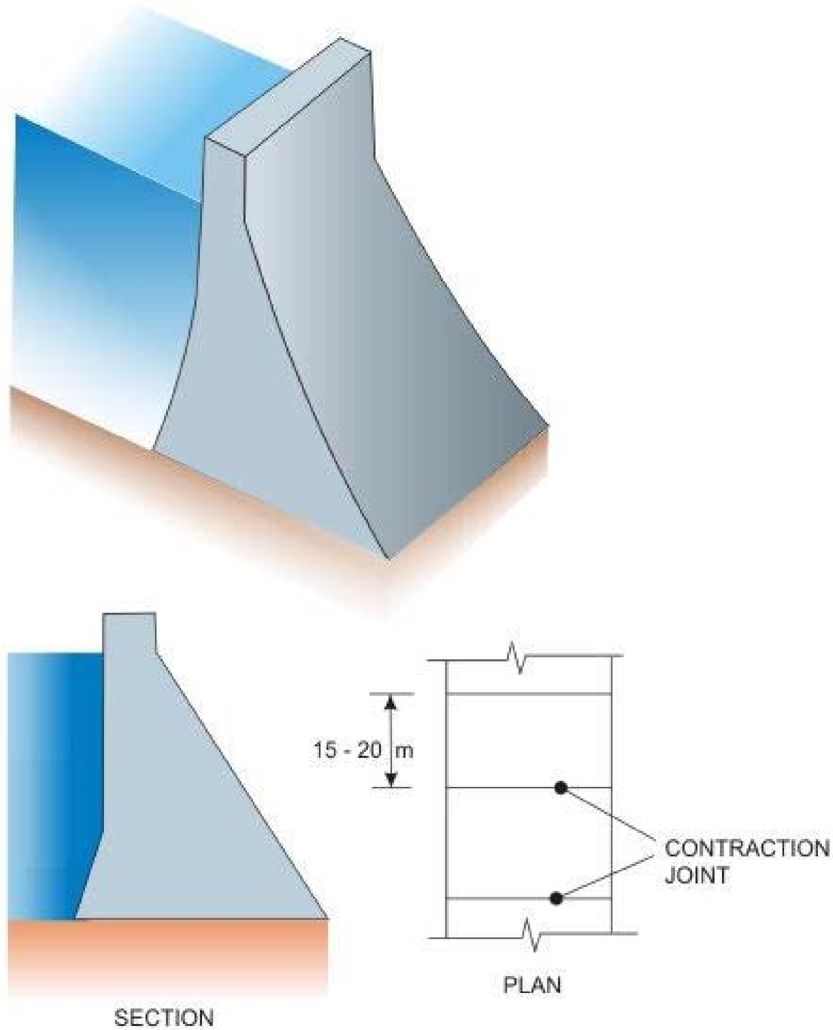


FIGURE 4(a) Concrete Gravity dam

2. Buttress dams (Figure 4b): This type of dams consist of a continuous upstream face supported at regular intervals by buttress walls on the down stream side. These dams are generally lighter than the solid type of dam but likely to induce slightly higher stresses at the foundation since most of the load now passes through the buttress walls and not spread uniformly over the foundation as in a solid gravity dam.

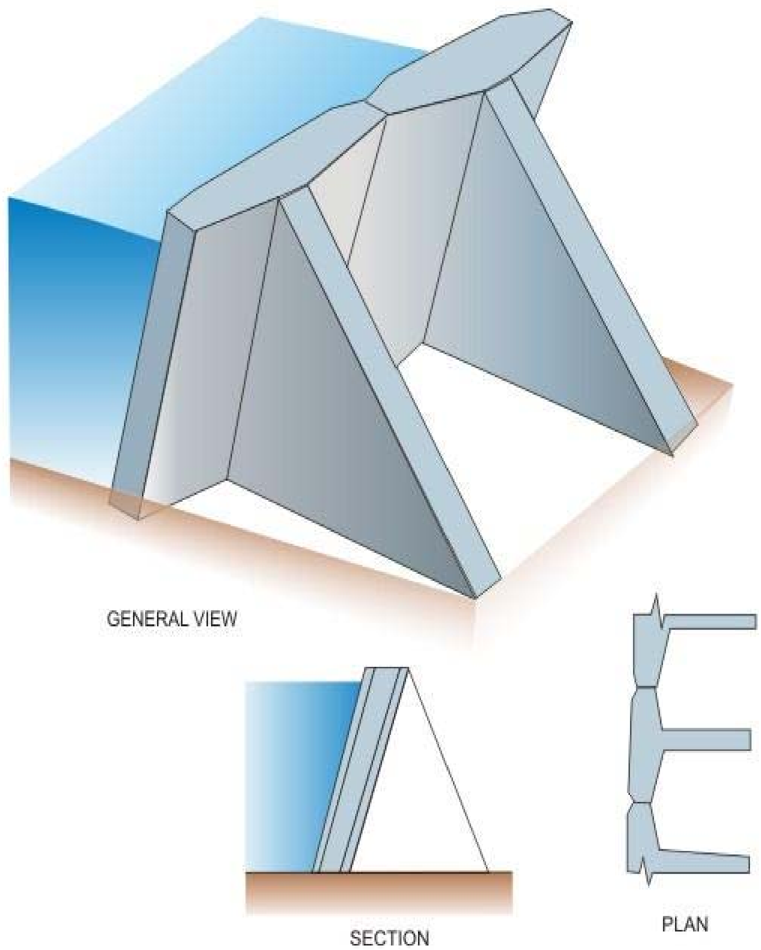


FIGURE 4b Buttress Dam

3. Arch dams (Figures 4c and d): These types of dams have considerable upstream curvature in plan and rely on an arching action on the abutments through which it passes most of the water load on to the walls of the river valley. This type of dam is structurally more efficient gravity dams and greatly reduces the volume of concrete required. Of course, a prime necessity in constructing an arch dam is to have sound foundation and abutments.

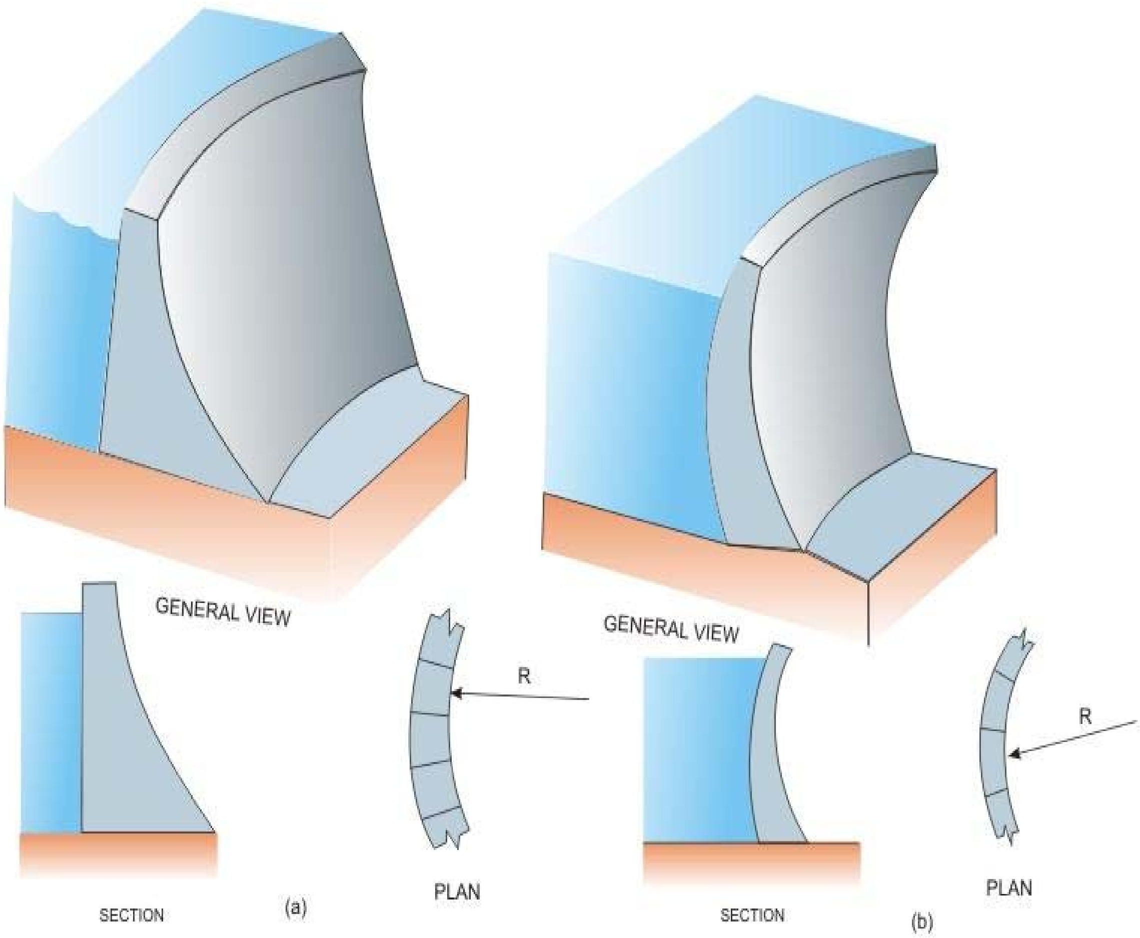


FIGURE 4. Principal types of concrete dams(continued) ; (c) Arch gravity dam ; (d) Cupola or double curvature - arch dam.

These are a few other types of concerted dams that have been constructed, a couple of which have been illustrated in Figures 5a and b.

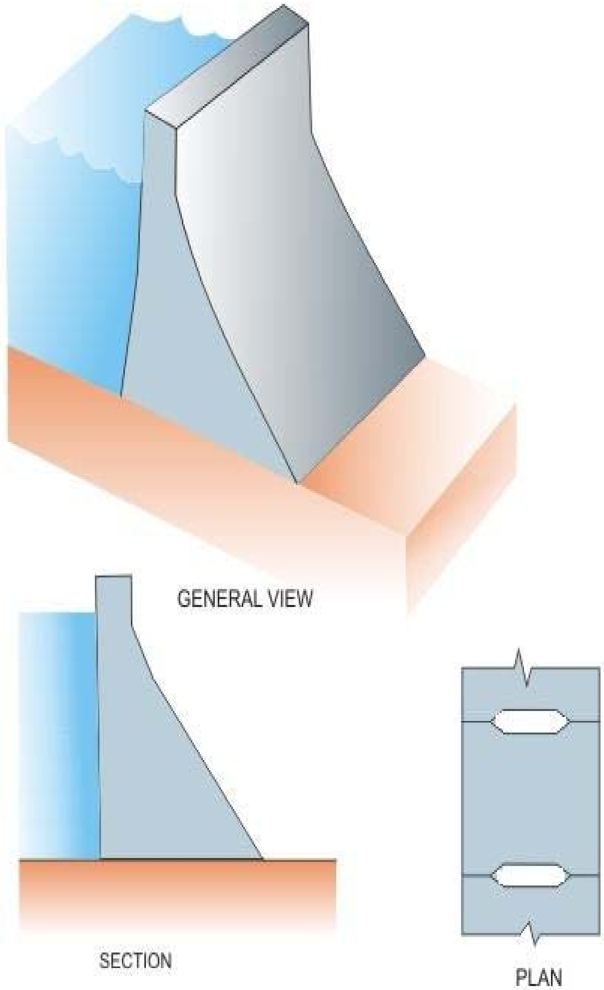


FIGURE 5a. Hollow gravity dam

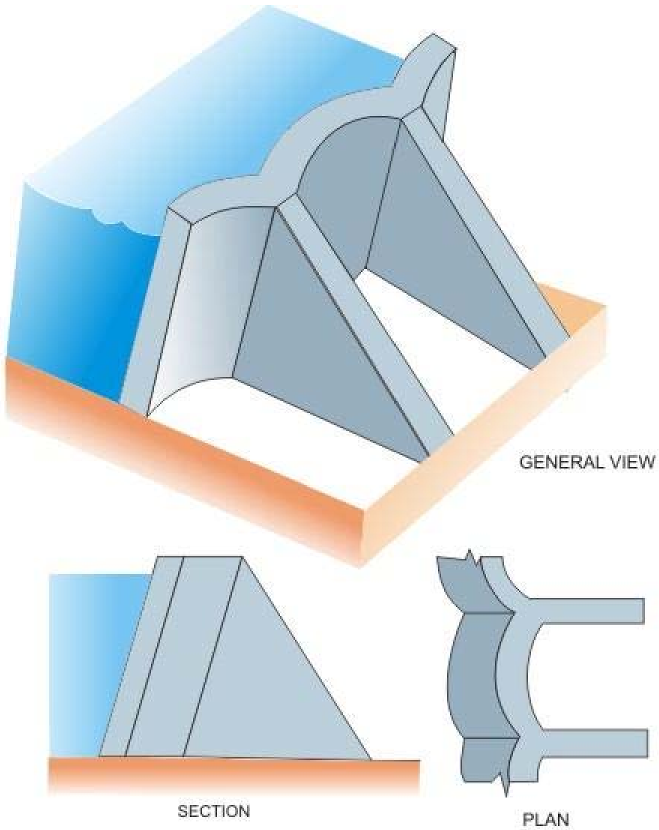


FIGURE 5b. Multiple Arch dam

Some of these, it may be noted, are made of reinforced concrete, and not mass concrete for gravity dam. Some of major advantages of having a concrete dam are listed below:

1. Concrete gravity, hollow gravity or buttress dams are suitable to all kinds of river valleys- narrow or wide, provided that good rocky foundation is available at moderate depths below the river bed.

2. Concrete dams are not sensitive to over topping, unlike the embankment dams. However, the water over topping the concrete dam may destroy the foundation down stream due to the impact of the falling water.

3. Concrete dams may accommodate a crest spill way, or sluice ways through the body of the dam, which is not possible for embankment dams. The cost of having a separate spill way channel is thus avoided.

4. Concrete dams are more resistant to withstand seismic disturbance.

5. Cupola or Double curvature arch dam is an extremely strong and efficient structure, provided a narrow valley exists with good rock in abutments and foundation.

4.4.2 Choosing a suitable location of dam

With a decision made about the necessity of having the dam across a river near about a broad area, attention is focused on narrowing down into one, or preferably two or three alternate sites that may be apparently suitable from visual inspection. Detailed investigations are carried next to examine the option that satisfies as being most economical, technically more suitable, convenient for construction etc. The various investigations that are carried out for finalizing a particular dam, specifying its type, height, method of construction, etc. are mentioned below.

Topographic Requirement

From a preliminary observation of the elevation contour maps of a region, one has to decide on an option that seeks a gorge which is most narrow, which would require minimum quantities of dam construction material. At the same time, an ideal location may also be decided from the volume of the water that may be able to store in the reservoir behind the dam. This may be observed from Figure 6 which shows two possible alternate sites that may be considered suitable from the available narrow gorge sites, A-A and B-B. Though the heights of the dams are nearly the same as may be observed from the corresponding elevation contours, the length of the dam for the latter is clearly more with an expected corresponding higher volume of construction material that would be required. Nevertheless, a further inspection of the elevation contours on the upstream indicates that a dam A-A would have a much smaller reservoir as compared to that at B-B.

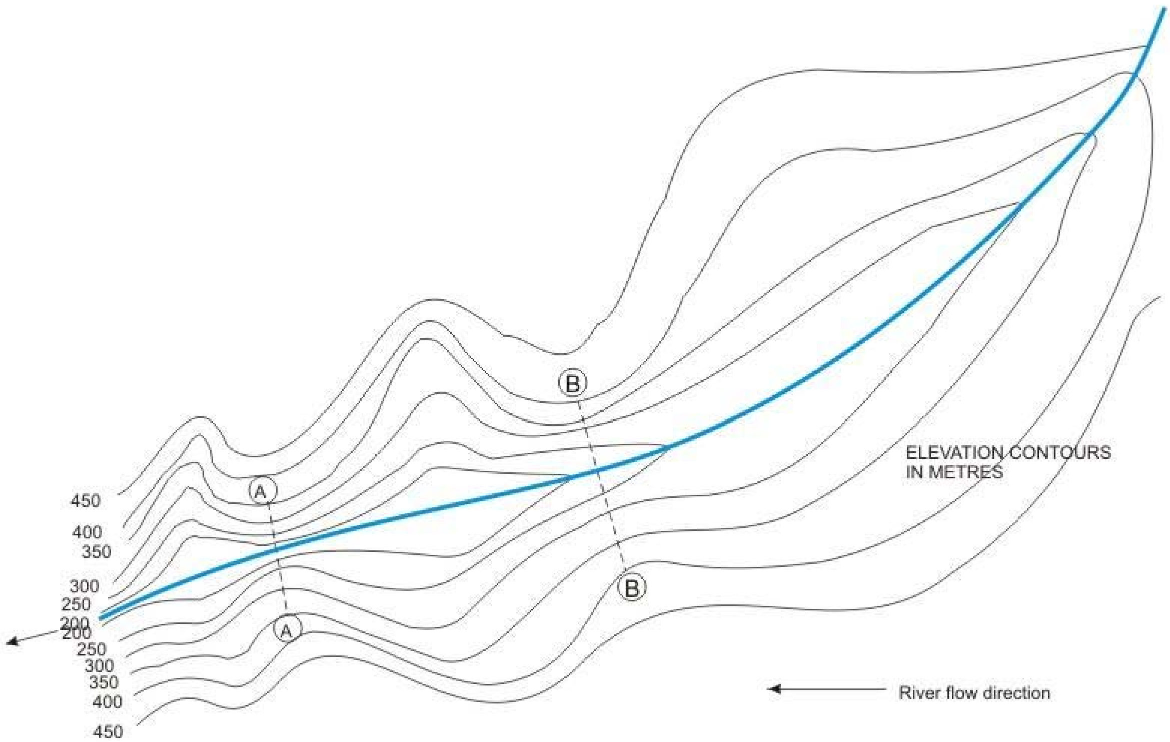


FIGURE 6. Topography of a typical damsite area showing possible alternate location

Submergence possibilities

Though a preliminary investigation indicates one location to be more preferred than others, it may have to be seen what losses have to be sustained if each of these alternatives are selected. There could be a valuable forest area which might get submerged. Of course, it is possible to replenish the loss by planting more saplings at other places not subjected to submergence. However, there could be some industry located upstream, or some mines, whose relocation may not be economically possible.

Geotechnical suitability

Since a dam is a massive structure, the foundation should be geo technically sound to sustain the high stresses that is expected to be developed due to the self weight of the structure, water pressure of the reservoir and earth quake vibration induced forces at the dam body and the water in the reservoir.

The geological and geotechnical investigation of a dam site for detailed evaluation is to be directed towards determining the geological structure, stratigraphy, faulting, foliation and jointing, and to establish ground water conditions adjacent to the dam site, including the abutments. The objective of these investigations is to:

- Determine the engineering parameters which can be reliably used to evaluate the stability the dam foundation.

- To determine the seepage patterns and the parameters for enabling the assessment of the probable seepage pressures.
- To make sure that there are no bad patches within the would–be reservoir area, like lime stone caves, through which reservoir water may leak out.

It needs also to be established by geotechnical investigations about available construction materials in the economical vicinity of the dam site. The tasks that need to be performed for geotechnical testing include the following:

- Logging of all natural and excavated rock exposures and borehole records.
- Correlation between the exposures and borehole data and inferring out a spatial pattern of subsurface rock jointing patterns, layers or seams of weak materials, etc.
- Excavation of additional trial pits, boreholes, shafts and exploratory adits wherever considered necessary.

For carrying out testing in rock, rotary drilling and coring techniques have to be used. The Bureau of Indian Standard code IS: 6926-1996 "Diamond core drilling – site investigation for river valley projects- code of practice" may be referred for subsurface exploration for valley projects using core drilling with diamond drill bit. The following two codes may also be referred for recommendations about the ways of storing the geo logical data and preparation of drilling information which might be useful to the designer of the dam:

- IS: 4078-1967 "Code of practice for indexing and storage of drill cores"
- IS: 4464-1967 "Code of practice presentation of drilling information and core description in foundation investigation"

For exploring the continuity or character of subsurface formation and depth, explorations have to be carried out through drifts or tunnels. They are most frequently used for investigation of fault shear zone, buried channels and suspected places of weakness in dam foundation, abutments and beneath steep slopes or back of cliff like faces.

For investigating open fissures or the explore zones of weak rock which could break up in the core barrel and are incapable of being recovered intact, deep trial pits may have to be used. For continuous exposure of the ground along a line or section, trenches may be used, which are best suited for shallow explorations on moderately steep slopes, as for abutment of dams. Exploration may also be done through borings, which are perhaps the simplest methods for subsurface investigation and sampling.

The details of various methods followed for subsurface exploration for dams may be found in the following Bureau of Indian Standards codes:

- IS: 6955-1973 "Code of practice for subsurface exploration for earth and rock fill dams"

- IS: 4453-1967 “Code of practice for exploration by pits, trenches, drifts and shafts”
- IS: 1892-1962 “Code of practice for site investigation for foundations”

Sometimes, geophysical methods can be appropriate to obtain in relatively very short time; information regarding the nature of the various strata and their position and depth of change. These are not direct measurements and methods include a) Seismic refraction technique and b) Electrical resistivity technique. In the former, earth vibration are set up artificially by explorations which causes waves to travel to the subsurface layers which get refracted due to the changes in waves speeds and are finally picked up by geophones placed on the surface at certain intervals. In the latter, four electrodes at equal distances are laid along a line and an electric current is passed between outer electrodes. The potential difference between the inner electrodes is measured to obtain the resistivity.

All the above investigation techniques are used in one way or other to evaluate the following engineering parameters that are necessary inputs to dam designs:

1. Depth of overburden.
2. Permeability and porosity parameters with reference to seepage control.
3. Compressibility characteristics of sandy strata and their relative density.
4. Shear strength and consolidation properties of cohesive strata.

Depending on the above parameters, the most suitable type of dams can be selected based on designer’s judgement. If one site is geologically not suitable (for example, having very weak foundation) then a different location may have to be considered.

Hydrologic adequacy

At any site, the construction of a dam for creating a reservoir for water storage would require the following points to be adequately satisfied:

1. The average quantity of water available in the river through out year.
2. The minimum flow of the river, both as the absolute minimum and the minimum average over a period of a month or a year.
3. The maximum flow that has been recorded and estimates of what might occur in future.

When records of sufficient time are not available or are of doubtful accuracy, correlation is attempted with rainfalls and flows that have been recorded in the surrounding catchments. Hydrometeorology studies can give estimates of future predications. Some times visual inspections may also lead to signs of high flood marks.

Apart from the flow parameters mentioned above, which are required for designing the size of the reservoir (and consequently the dam height) and that of the spill way for passing the largest flood flow, local rain fall records are also essential to prepare reasonable construction schedule of dam. Continuous heavy rains during most of the year could deter the possibility of constructing an earth dam.

Sedimentation possibilities

The average quantity of sediment carried by the river has to be known, as precisely as possible, which would give an idea of the rate at which a proposed reservoir way get filled up. If the rate of sedimentation is too excessive on a certain river, then the project may have to be dropped or suitable engineering design, like provision of large sluices using the a concrete dam at lower levels of the dam, have to be incorporated by the design engineer.

Availability of a suitable spillway site

All dam should have an adequate spillway for passing flood flows. If a river gorge is narrow, then there may not be sufficient spillway width available and a suitable location on the periphery of the reservoir has to be found to locate a spillway. If that is not possible, then the proposed dam site has to be abandoned and other alternatives are selected. A good judgment is also required to decide the suitable type of dam based on spillway consideration.

Possibility of river diversion during construction

The way, river can be diverted at a particular site for making way for construction of the dam may affect the design of the dam and also the construction schedule.

4.4.3 Selection of the type of dam

The following major factors may have to be considered while selecting a suitable type of dam.

1. **Environment and public opinion-** a dam must be constructed without disturbing the surrounding environment, at least to the minimum extent as is possible. If constructing a dam at a certain location entails quarrying up large areas of beautiful greenery, for example, then another alternative may have to be thought of or a suitable remedial measure chalked up.

2. **Availability of construction material-** If the cost of transportation of construction material is excessively high, then an alternate design with locally available materials, have to be considered. For any type of dam, the source of construction material has to be critically evaluated. For earth fill dams, borrow source are prone to over estimation of the available yield of suitable material owing to undetected variations in soil type or quality. It is therefore essential to prone quantities of individual fill materials substantially before proposing a dam section at a site using those materials. The proving of sources for rock fill is superficially more, straight forward than earth fill. The essential requirement is a source of sound durable rock, the location of which sis generally apparent from the initial geological appraisal. Investigation of the suitability of the rock fill will normally require trial fill and in the case of excavated or quarried rock, it will also

be necessary to conduct blasting or ripping trials to determine rock fragment size, grading, shape, etc. Aggregate sources for concrete dams include borrow areas and the use of crust aggregate derived from quarries or excavations.

3. **Unavailability of skilled workers-** In case of sophisticated dam section, skilled workers are an absolute necessity. Unavailability of such workers at proposed dam construction site may have to force the designer to adopt a more easy to construct a type of dam.

4. **Seismicity-** With available dynamic structural analysis computer program for dams using techniques like the Finite Element Method, it is possible to analyse the behavior of the dam under earth quake vibrations thereby making it possible for the designer to check if a particular section is suitable or not. It has been observed that rock fill dams provided with filters, materials from which could move into and seal crakes in the core material appears to be one of the subject type of section that may be provided in earth quake prone regions. One of the foremost dynamic structural analysis computer program developed was that by G L Fenves and A K Chopra of Earthquake Engineering Research Center of University of California at Berkeley. A description of the program is available at the following web-site: http://nisee.berkeley.edu/elibrary/getdoc?id=141382.

5. **Geology and foundation strength-** The existence of joint patterns in the abutments (their orientation, inclination and infilling) may indicate the possibility of instability under loading from an arch dam and reservoir water. Such a site would be more satisfactory for an embankment dam or an adequately dimensioned gravity dam. Where the possibility exists of differential deformation of the foundation along the axis of a dam, a gravity or arch dam would not be a suitable choice because of their inherent rigidity due to their construction in concrete. Instead, an embankment dam may be proposed, which is more flexible. Further, it may be noted that the stresses expected at the base of a dam may have to be checked with the bearing capacity of the foundation material to propose the suitability of a particular section. Embankment dams produce the least formation stress, Followed by gravity, buttress and arch, in that order.

6. **Hydrology-** If, during the construction season, there are possibilities of the partially constructed dam being overtopped by the floods of the river water, then a concrete dam section would be preferred then an embankment dam section. If an embankment dam section is still proposed to be built, then adequate diversion works have to be provided for diverting the river flood water. It may be noted that if a concrete dam is inuded and overtopped, not much loss would occur, where as if such a thing happens over an embankment dam, then the downstream face of the dam would be eroded away, gradually reducing the section and finally causing it to collapse.

7. **Valley shape and overburden-** the shape of the river valley and the overburden, that is, the loose bouldery, gravelly, or sandy material overlying the river floor also influences the type of dam that may be proposed to be constructed. In case of a wide valley with deep deposits of fine-grained soil overburden, say more than 5 meters, favours earth fill embankment dams (Figure 7a) A river valley that has much less over-burden (Figure 7b), would be suitable for embankment, gravity or buttress dams. A narrow valley with steep sides (Figure 7c) and with sound rock in the valley floor and sides may be suited to an arch or cupola dams. If not, then the economics of proposing an alternative in the form of, say, rock fill embankment dam has to be studied. In case of a wide valley separated in two parts (Figure 6d) may suggest a combination of two types of dams. An earth fill embankment may be constructed where the overburden depth is considerable and a concrete gravity dam on the site where the overburden is less. The spillway portion can then be located on the concrete gravity section. Of course, in all the four cases mentioned, the availability of alternative construction materials and their relative costs would have to be considered as well, to choose the final section.

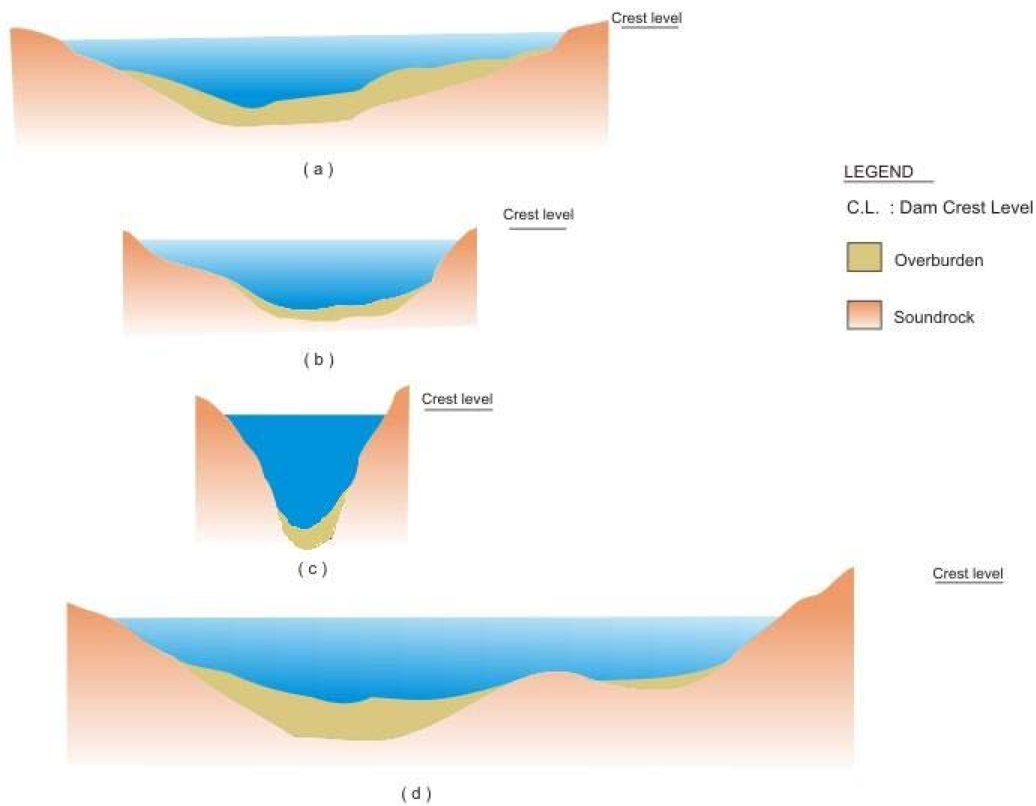


FIGURE 7. Typical valley shapes (a) Wide valley with deep overburden ; (b) Valley with little overburden ; (c) Narrow valley with little overburden ;(d) Valley with irregular depth of overburden

It may be concluded that all the possible alternative designs of dams for a particular dam site must be weighed very carefully. However, some times sufficient time is not available for comparison of acceptable alternatives. At certain sites, it may be expedient to select a simple type of dam – either simple to design or simple to construct. Of course, this may not be safest or most economical dam for a particular site.

Further, the time and money spent on investigation also has an impact on the decision of having a particular dam section. An illustration of a particular case would prove that investigation, especially for geotechnical and foundation suitability cannot be under estimated. The case in point is that of the Salal dam on river Sutlej where an embankment dam was proposed to block river and a chute spillway was found attractive over a mountain ***saddle***. A typical location of such a proposal, but with hypothetical elevation contours is shown in Figure 8a. The designs were made considering a suitable foundation of the spillway and construction commenced. However, during the course of foundation and excavation for the spillway it was that the rock quality of the saddle is quite poor due to the presence of phyllites. In order to get to the sound rock strata, further excavation was carried out, till the spillway foundation level was lowered up to the riverbed. A comparison between the original section proposed (Figure 8b) and the final section constructed (Figure 8c) shows the increase in cost that resulted due to an economization at the time of original investigation. Had a more careful (and consequently slightly more costly) investigation been carried out, then perhaps a better alternative might had been proposed with a concrete gravity dam at axis A-A with an integrated over flow spillway.

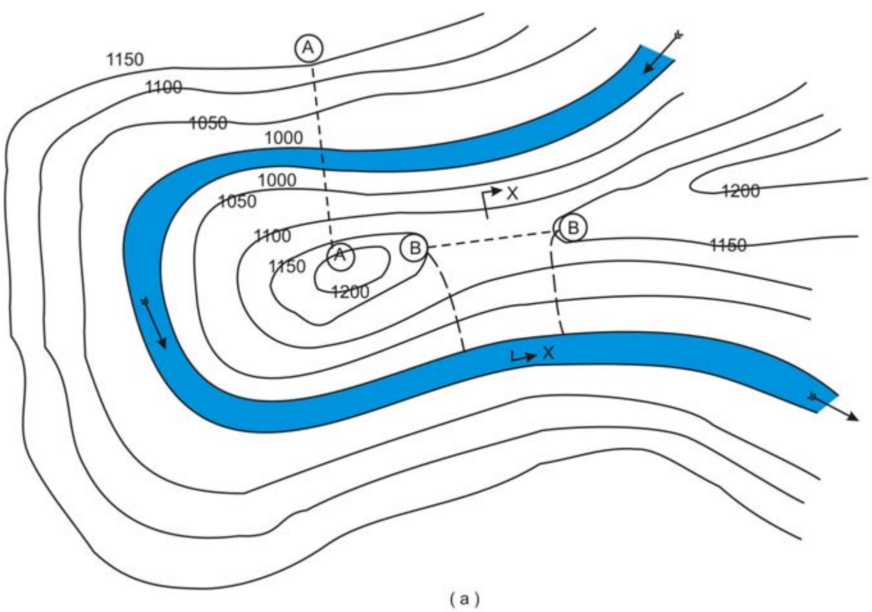


FIGURE 8(a). Schematic layout of a dam and spillway proosed at a river bend
Axis A - A proposed for embankment dam & B - B for chute spillway.

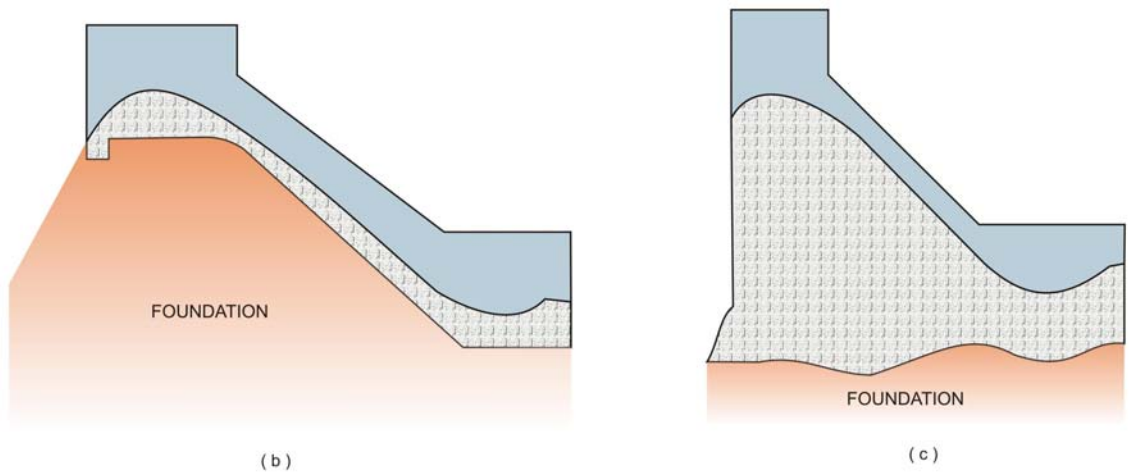


FIGURE 8 (b) Section X-X through originally proposed chute spillway for damsite shown in Figure 7(a)
(c) Final section of spillway adopted due to poor foundation geology

4.4.4 Appurtenant structure and ancillary works

It is not just sufficient to have dam constructed across a river to store water in a reservoir. Each dam has to have certain appurtenant structures to enable them to discharge their operational functions safely and effectively. One important aspect is the passage that has to be constructed to allow the flood waters to flow down the river without affecting the safety of the dam. Then there are the outlet works which provide a means for controlled and regulated discharges for meeting certain demands. These and other structures discussed briefly here, but would be dealt in detail in subsequent lessons.

Spillway

Spillways or passages for letting out flood waters when the reservoir, is over flowing has three major components:

- Entry to the spillway, which may or may not be controlled using gates.
- A channel for conveying the water from the reservoir side to the down stream of the dam.
- And energy dissipating arrangement for the water flowing down the spillway channel as it reaches a lower elevation near the outlet of the channel.

The capacity of the water conveyance of the spillway should be such that it must safely pass the maximum design flood. More than one type of spillway may be provided in a particular dam. For example in the Indira Sagar Dam on river Narmada, two sets of spillways have been provided, one called the main spillway and the other auxiliary spillway. All the spillways are gated, but the crest level of the auxiliary spillways, are slightly higher than that of the main spillways. Under normal flood situations, the main spillways may be operated but if the flood is excessive, auxiliary spillways can be brought into operation. Spillways can be integrated into any type of concrete dam, if so desired but for embankment dams, separated passage has to be designed.

Outlets

These include outlets for irrigation canals, power channels or tunnels, water conditions for domestic and industrial use etc. The provision for such works can be readily accommodated within a concrete dam. However, for embankment dams, it is normal practice to provide an external control structure or value tower, which may be quite separate from the dam. A bottom discharge facility operating under high pressure of the water in the reservoir, through provided in many concrete dams, may be quite expensive, since it is necessary to ensure long life and reliable operation. The following types of sluices are used for following purposes:

- For used during river diversion, at the time of construction of the concrete dam. If these sluices are left out in the body of the dam at a lower level, the construction of the dam can safely go on at higher levels.
- To control the rate of filling of the reservoir. This would be necessary during the first time reservoir filling.
- As part or the whole of the permanent spillway discharge, as it has been done for some arch dams.
- To release the bottom water from a stratified lake. This action may be desirable to remove foul water from the bottom of the reservoir after initial filling.

Cut off

The seepage under and round the flank of a dam must be controlled, or else the foundation of the dam may be weakened. This is achieved by the construction of a cut off or a barrier below the dam penetrating the foundation. This should be continued further towards the abutments on either flank also in order to ensure the complete safety. Different types of cutoffs are in vogue for dams resting on pervious foundation (like river overburden) or drilled and grouted holes in fissured rocks.

Internal drainage arrangement

Any dam, embankment or concrete, is bound to have seepage of water from the reservoir side to the downstream. Of course, the rate of seepage through a concrete dam would be much less than that through an embankment dam. Hence, internal drainage system is almost invariably provided in most of the dams. In embankment dam, a thin vertical wall of very pervious material acts like a drain, collecting the seepage water and passes it downstream with the help of a connected horizontal layer of pervious material that extends up to the down stream face of the embankment dam. In concrete dams, vertical drains are formed near the upstream face of the dam which collects the seeping water and pass it on to a gallery near the base of the dam that extends from one abutment to the other. From this gallery, the collected water is drained off to the downstream by drains located at suitable points.

4.4.5 Layout of dams

As pointed out earlier, each dam project is designed uniquely based on the local conditions and other factors. Hence, there may not be any generalized solution for the layout but has to be decided individually for each case. In this section, some particular projects have been illustrated demonstrating typical layouts for embankments and concrete dams. A brief description of the projects accompanies the figures.

Bhakra Dam

This is the so far tallest dam in India constructed under the Bhakra-Nangal project on the river Sutlej meant to serve irrigation and power generation. The dam is 225.6m high concrete gravity type with two power houses having an aggregate installed capacity of 1050 MW. Some kilometers downstream of the Bhakra Dam is the 27.4m high Nangal Dam for regulation and flow diversion (as done in a barrage) through a 64km long power channel supplying to two power houses of aggregate of installed capacity of 154MW and a network of irrigation canals which command an area of 2.63 million hectors in the State of Punjab, Haryana and Rajasthan.

The general layout of the Bhakra Dam and its appurtenant structures is shown in Figure 9. A section through the spillway of the dam is shown in Figure 10. Two views of the dam, one from upstream and another from downstream are shown in Figure 11.

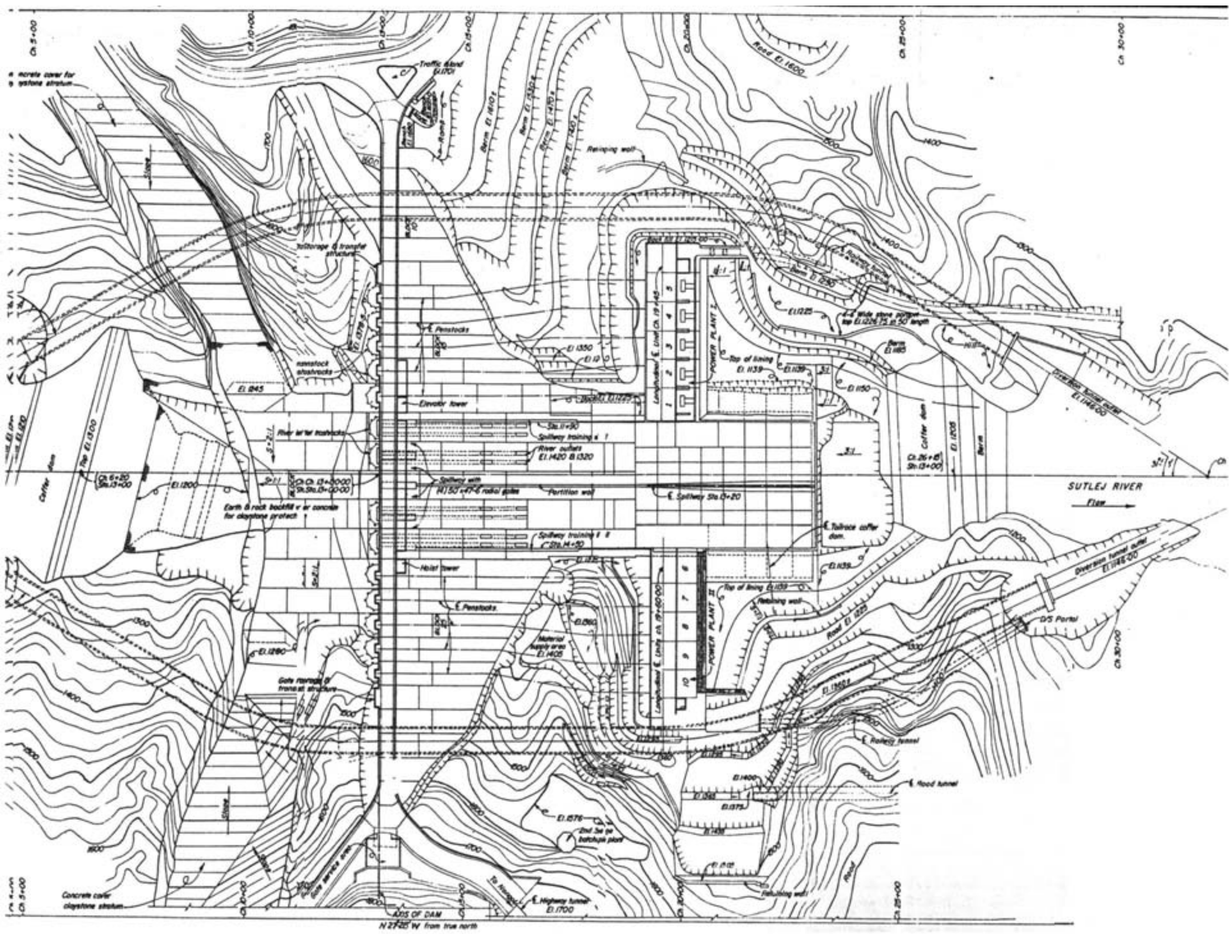


FIGURE 9. GENERAL LAYOUT OF BHAKRA DAM ON RIVER SUTLEJ

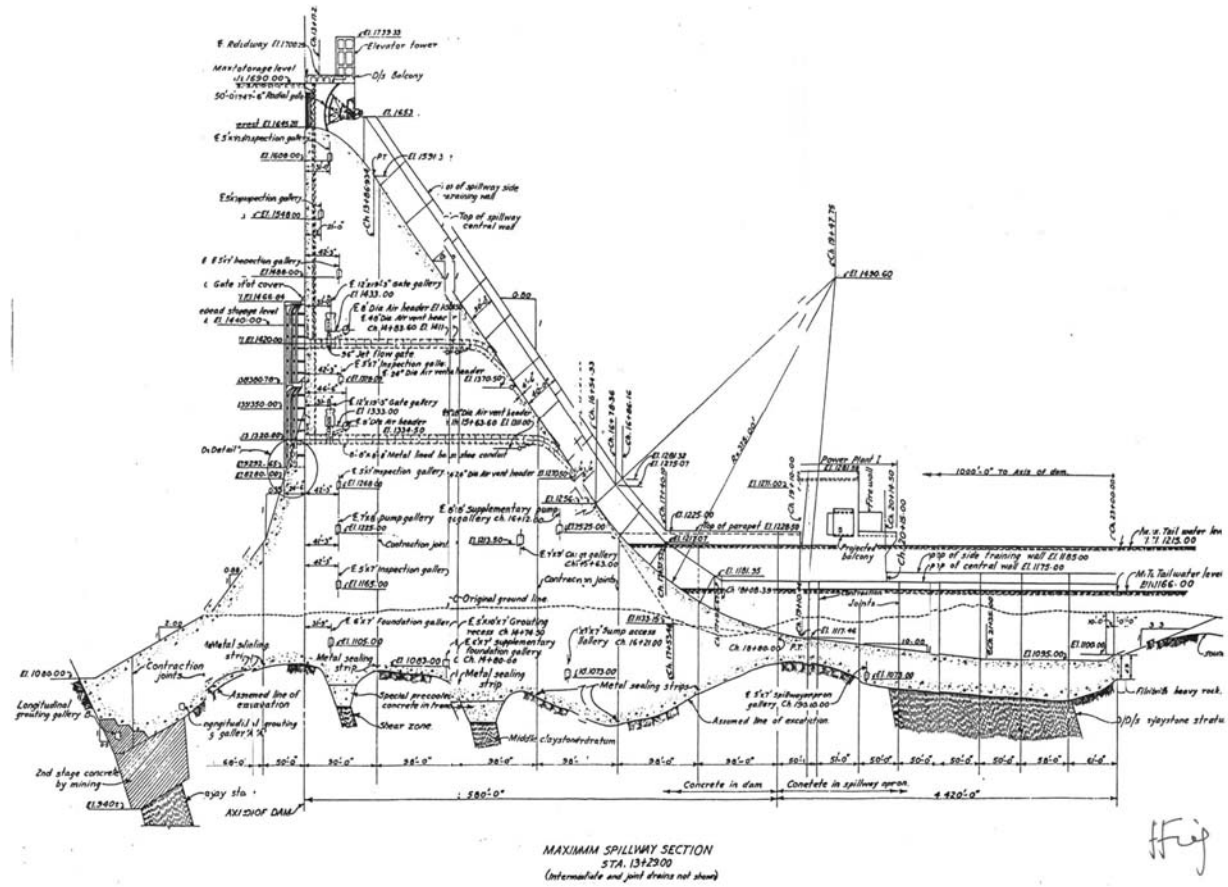


FIGURE 10. SECTION THROUGH BHAKRA DAM

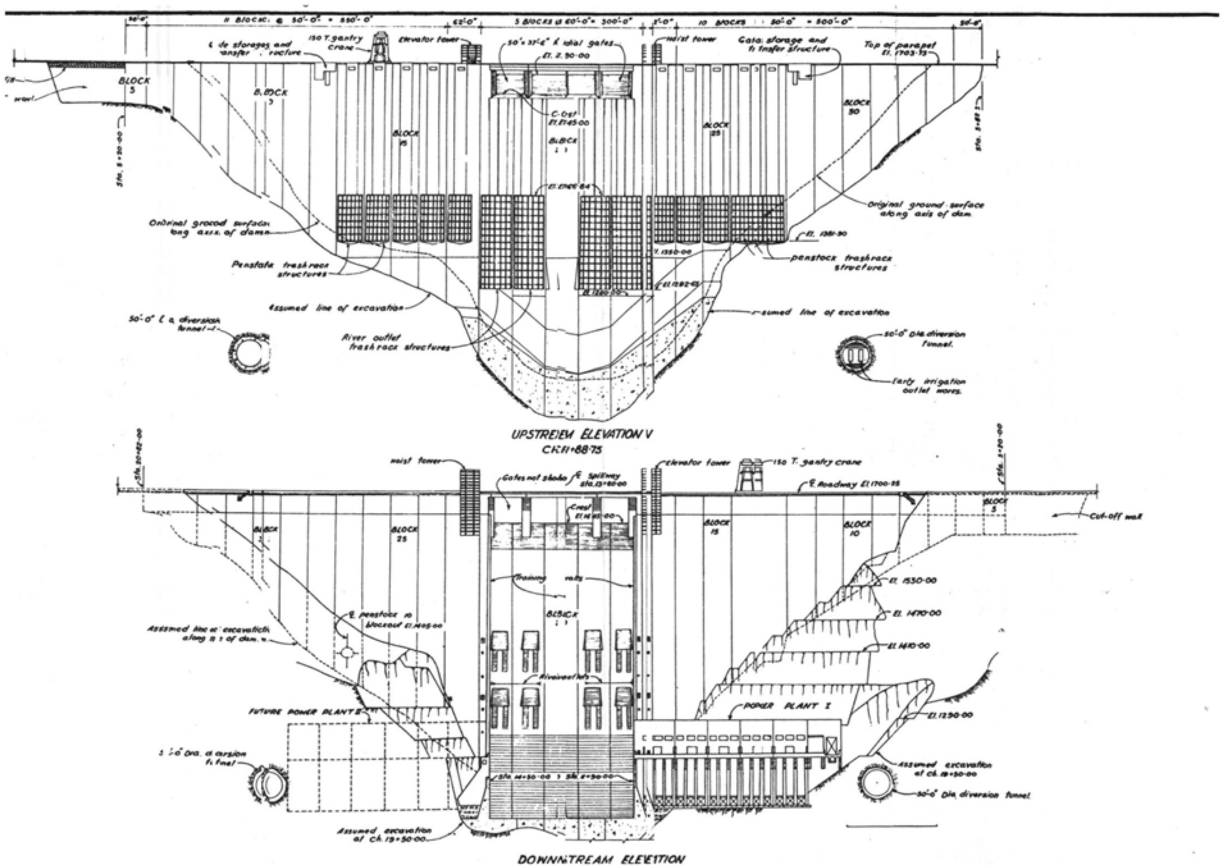


FIGURE 17. Upstream and downstream views of Bhakra dam

Nagarjuna Sagar Dam

This dam, on the river Krishna, is the tallest masonry dam not only in India but in entire world, with a maximum height (at the deepest portion of the river valley) of 124.7 m. Another unique aspect of the dam is that it was built entirely of human labour, employing about 60,000 workers almost throughout its period of construction. The dam is part of an irrigation project and has a reservoir with a gross storage capacity of $11.56 \bullet 10^6$ m^3. With two canals taking off from the reservoir at either flank to irrigate lands to the extent of $7.60 \bullet 10^5$ hectare and $4.45 \bullet 10^5$ hectare respectively, catering to about eight districts in the state of Andhra Pradesh.

A general layout of Nagarjuna Sagar Dam is shown in Figure 12 and section through the spillway and non-overflow blocks are shown in Figure 13. Though the main dam is built in masonry, it is connected to the higher banks of the river valley on either side by earthen embankment dams of length 2.56km on the left flank and 0.85km on the right flank. A section through the embankments is shown in Figure 14.

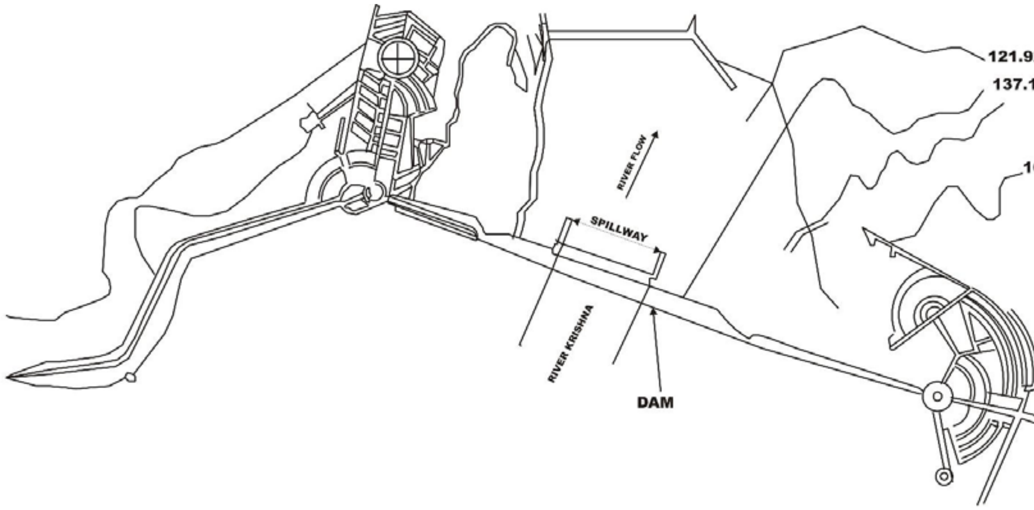


FIGURE 12. GENERAL LAYOUT OF NAGARJUNA SAGAR DAM ON RIVER KRISHNA

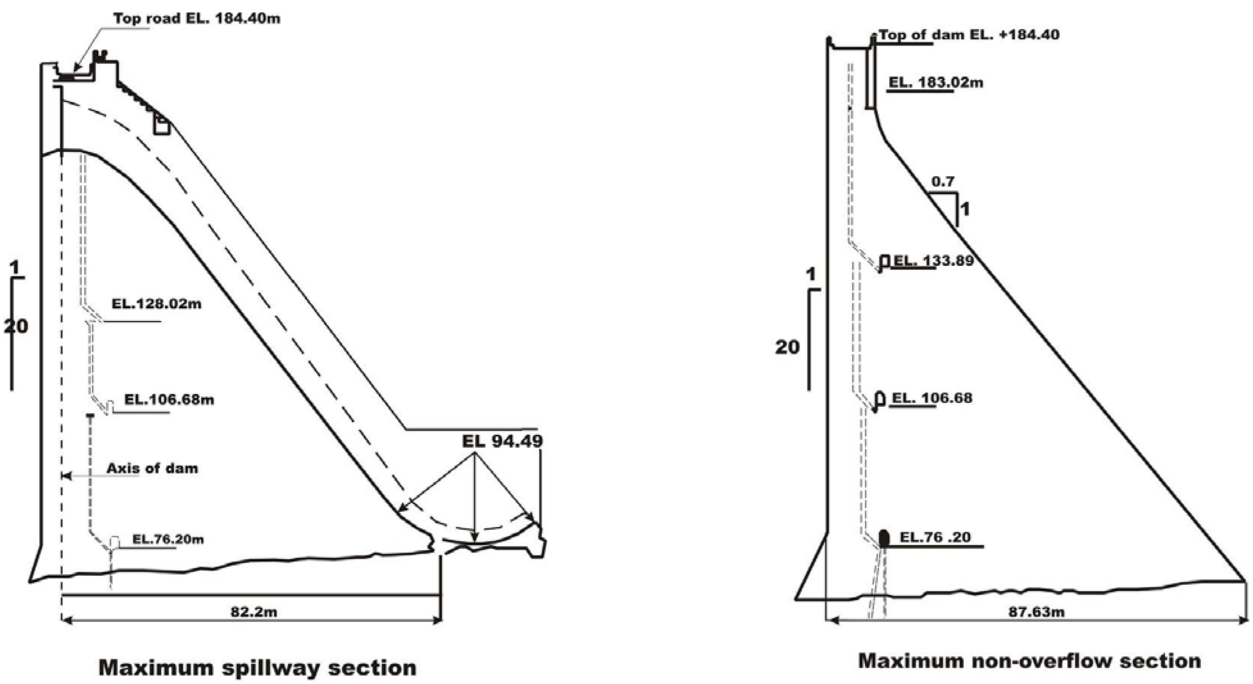


FIGURE 13 MAXIMUM SECTIONS THROUGH SPILLWAY AND NON-OVERFLOW BLOCKS OF NAGARJUNA SAGAR DAM

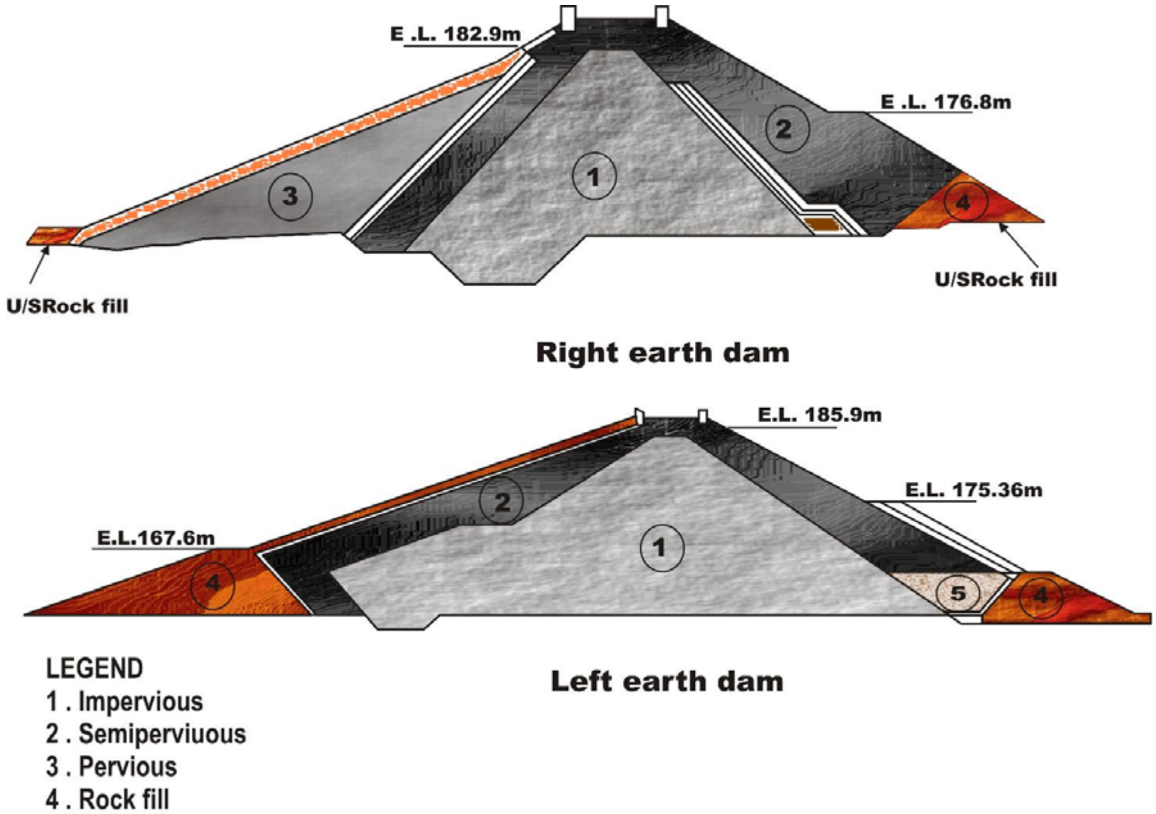


FIGURE 14. Maximum section of embankment dams of Nagarjuna Sagar project

Pong Dam

The Pong Dam is located in the Himalayan foot-hills in the district Kangra in Himachal Pradesh, on the river Beas. It is of the rock fill type with a central earthen core and sand and gravel shell zones on the upstream and downstream sides. The dam stands 132.6m from the deepest foundation level in the river valley. The length at the crest level is 1950.7m and width at the top is 13.7m except at special section where it is more. The overall base width at the deepest bed level is 610m. The dam serves the purposes of storing water for irrigation, power generation and flood control. A general layout of the dam is shown in Figure 15 and a view of the maximum section in Figure 16. The dam has an overflow type of gated chute spillway, whose plan and elevation are shown in Figure 17.

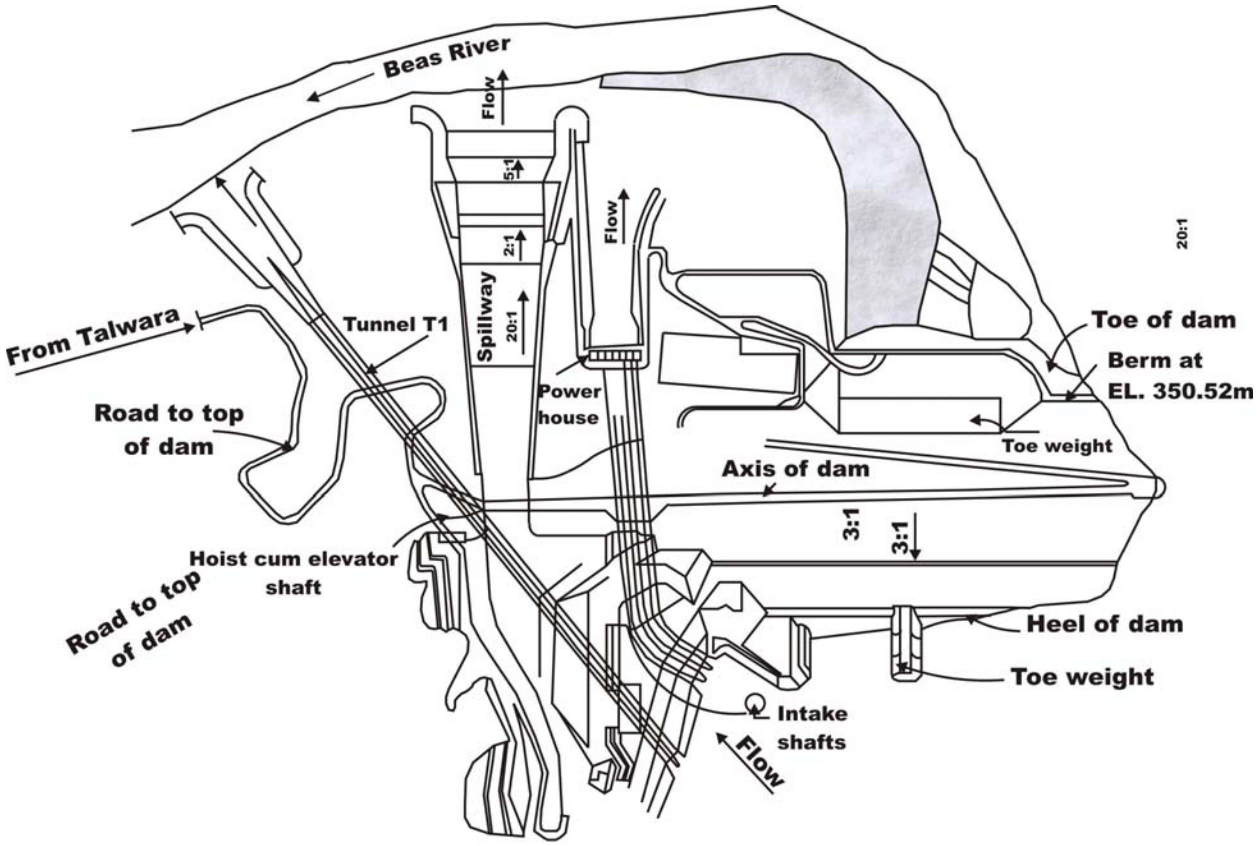


FIGURE 15. GENERAL LAYOUT OF PONG DAM ON RIVER BEAS

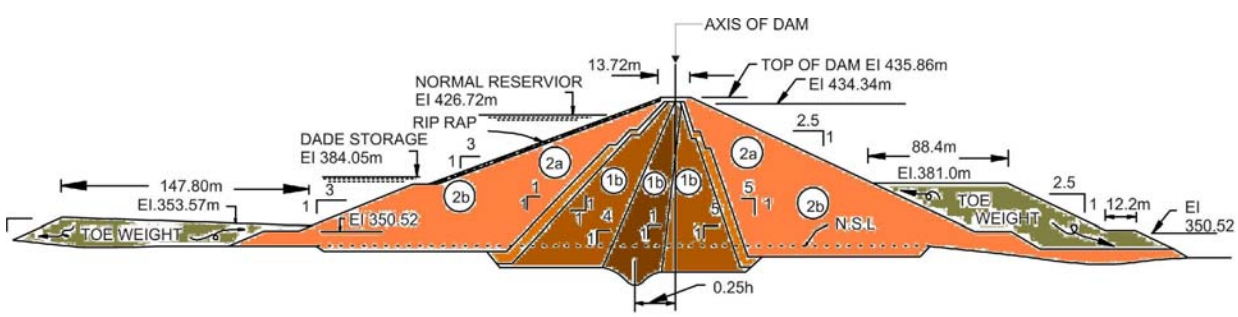
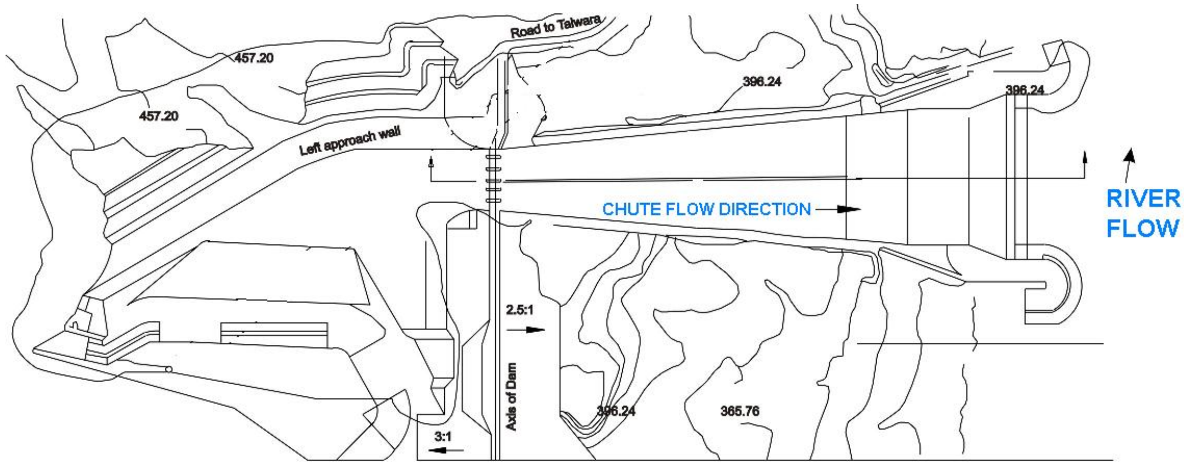
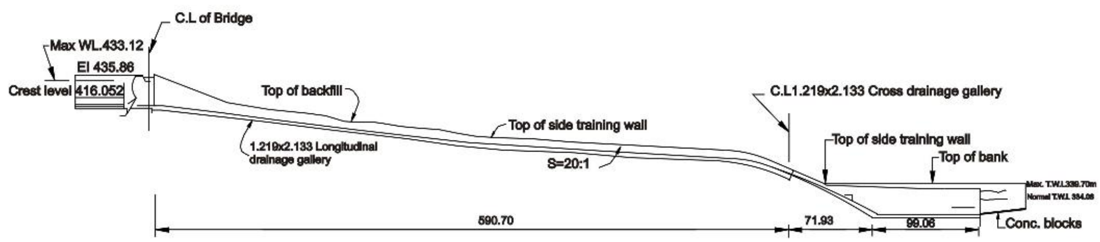


FIGURE 16.MAXIMUM NON-OVERFLOW EMBANKMENT DAM SECTION OF PONG PROJECT



PLAN



section A-A

FIGURE 17. PLAN AND SECTIONAL ELEVATION OF CHUTE SPILLWAY OF PONG PROJECT

Ukai Dam

This dam on the river Tapi is meant for irrigation to a total commended area of about 427110 ha and power generation of 193 MW. As may be seen from the general layout of the dam (Figure 18), there are two main parts of the dam- the masonry dam, as in the spillway portion (Figure 19) and power dam portion (Figure 20) and the earthen embankment dam (Figure 21).

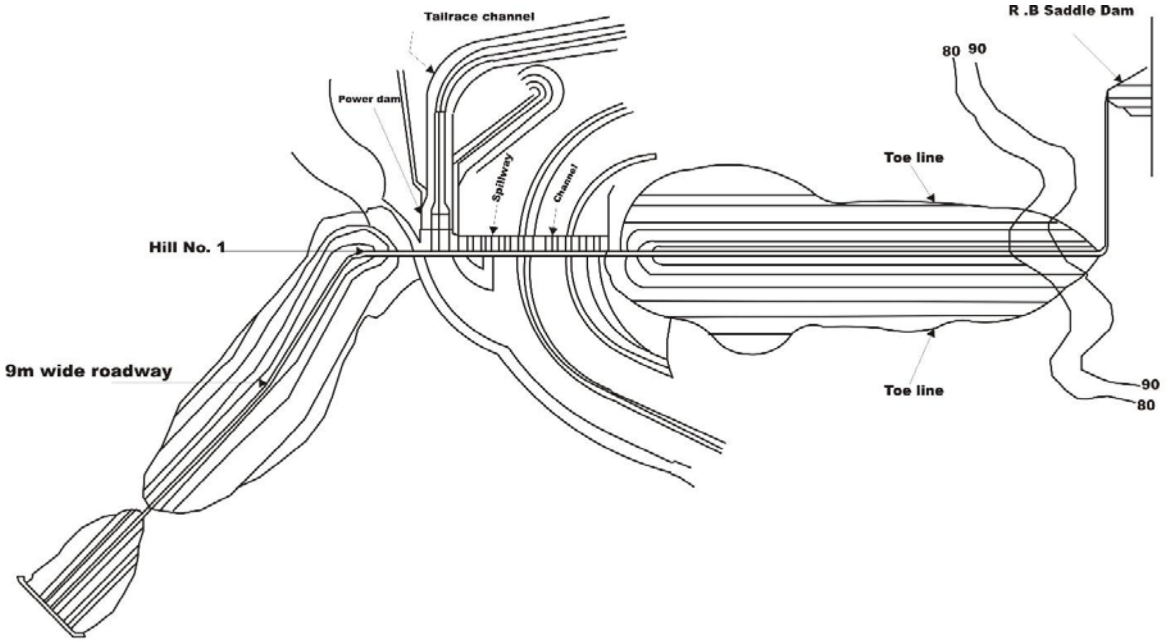


Figure 18 General layout of Ukai Dam on river Tapi

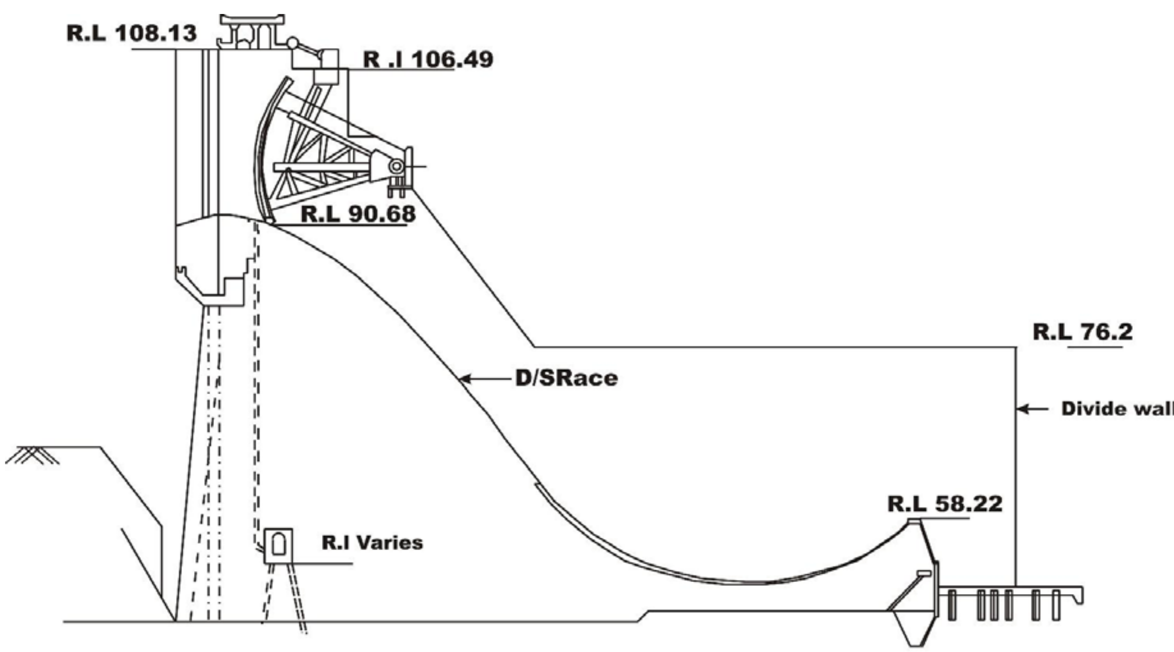


FIGURE 19. Spillway section of Ukai Dam

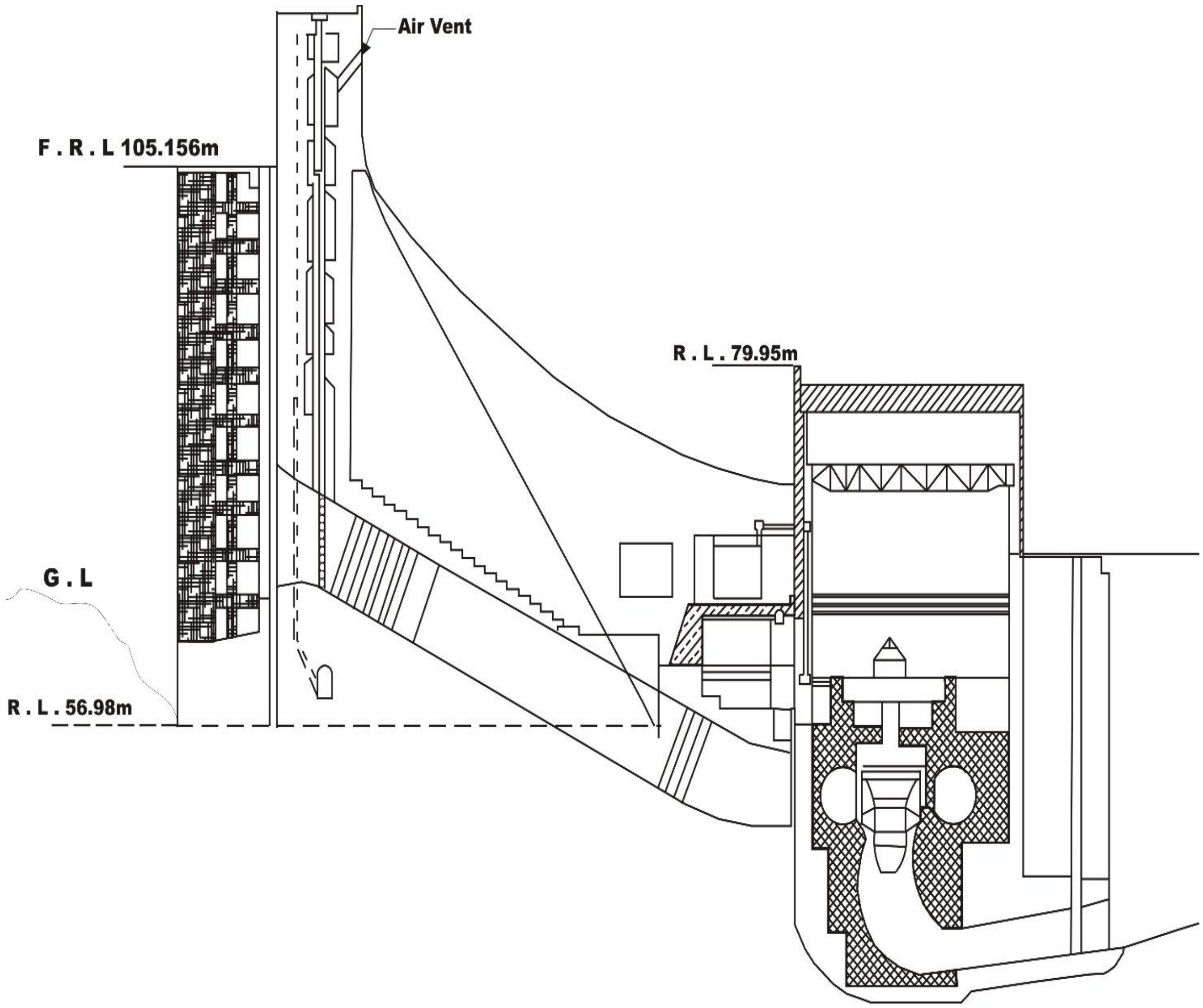
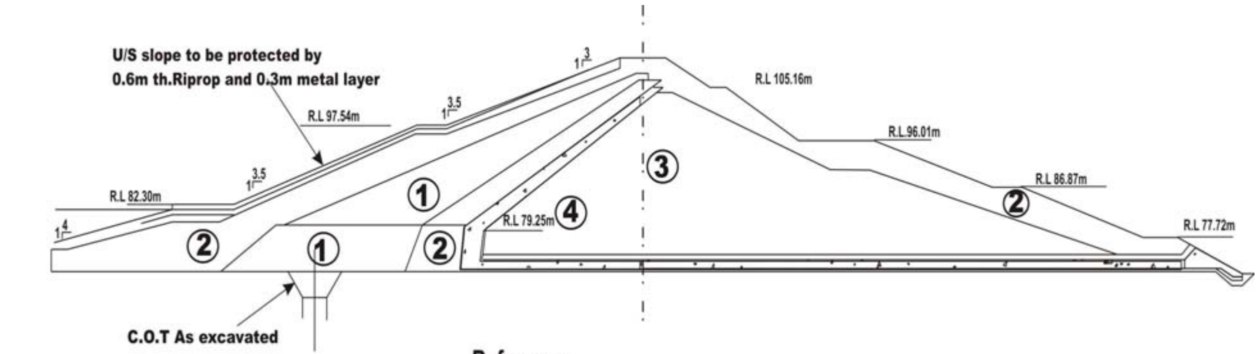
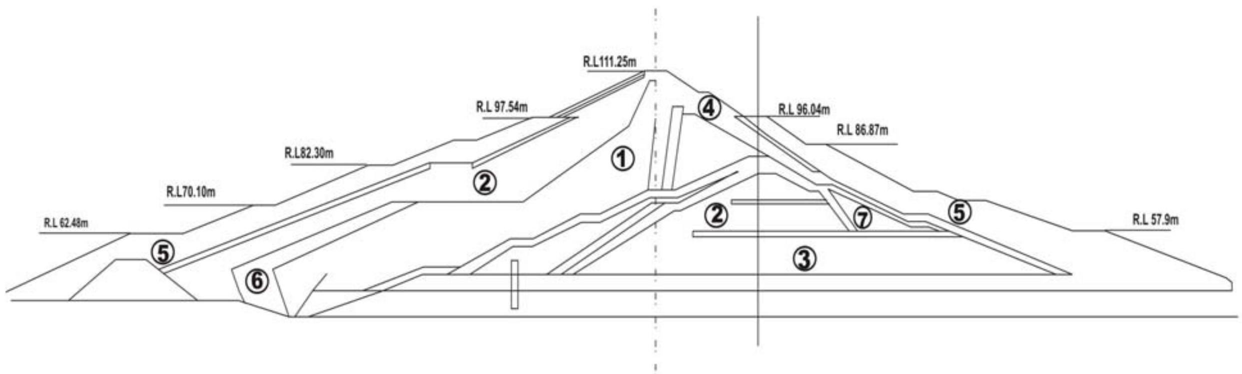


FIGURE 20. Power dam section of Ukai project



References
1. Impervious material
2. Semi-pervious material
3. Random fill (Material available from excavation of DC)
4. Filter.

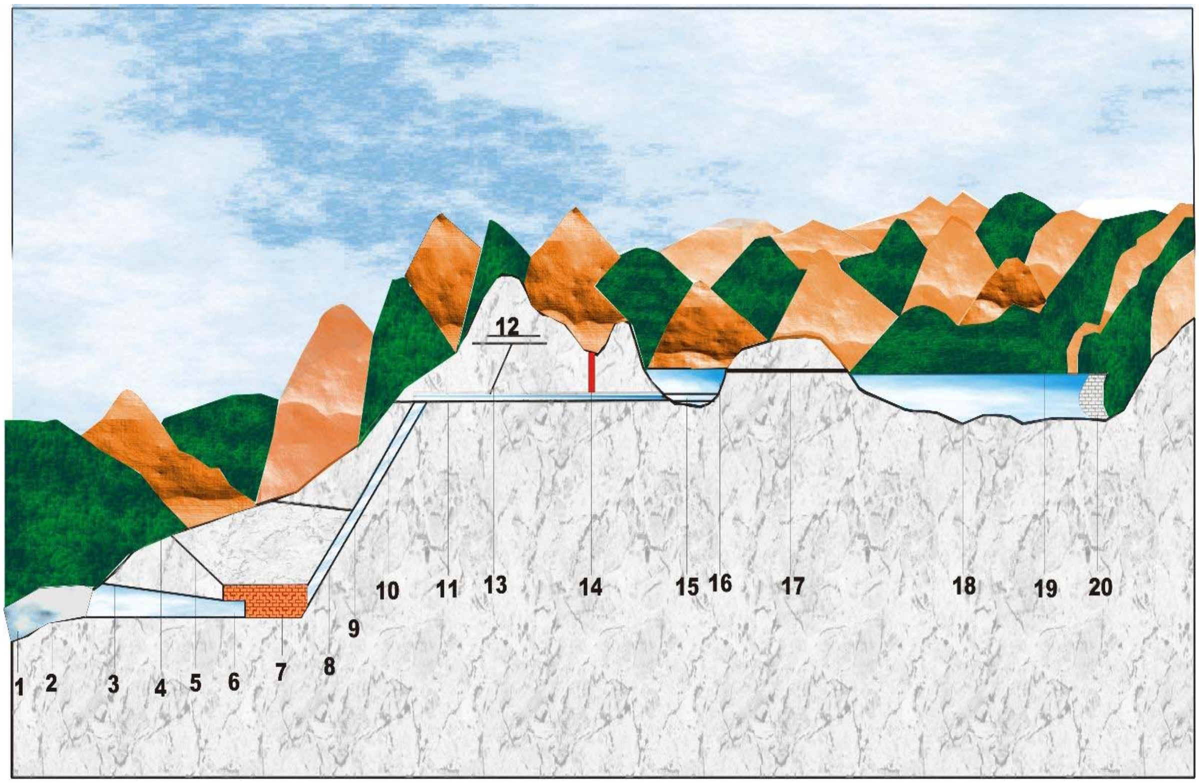


Reference
1. Impervious zone
2. Semi -pervious zone
3. Random zone
4. Sand filter
5. Rock zone
6. Silt zone
7. Sand and S.P Layers zone

FIGURE 21. TYPICAL EMBANKMENT DAM SECTIONS OF UKAI PROJECT

Idukki Dam

This dam is a part of a hydro electric project and is the only arch dam so far built in this country. The total project consists of this dam (182.9m high) across the river Periyar and another on the river Cheruthoni (152.4m high), which is a tributary of Periyar, to create a common reservoir of about $2 \bullet 10^9$ m^3 of storage. A tunnel diverts the water from this reservoir to an underground power house as shown in schematic profile (Figure 22). The Idukki Dam is defined as a thin double curvature, parabolic, asymetrical high arch dam, the plan and section which being shown in Figure 23.



LEGEND

1. Nachar	6. Tail -race tunnel	10. Penstock	14. Control gate -head race tunnel	18. Indukki Reservoir
2. R.C.C. Conduit	7. Underground power house	11. Butterfly valve chamber	15. Power tunnel Intake- tower	19. Cheruthoni Dam
3. Access tunnel	8. Pressure shafts	12. Expansion chamber	16. Kulamvu Dam	20. Indukki Arch Dam across Periyar
5. Switchyard	9. Intermediate adit	13. Surge shaft	17. Open cut channel	

FIGURE 22. Schematic profile through Intake project water conductor system, reservoir and dam

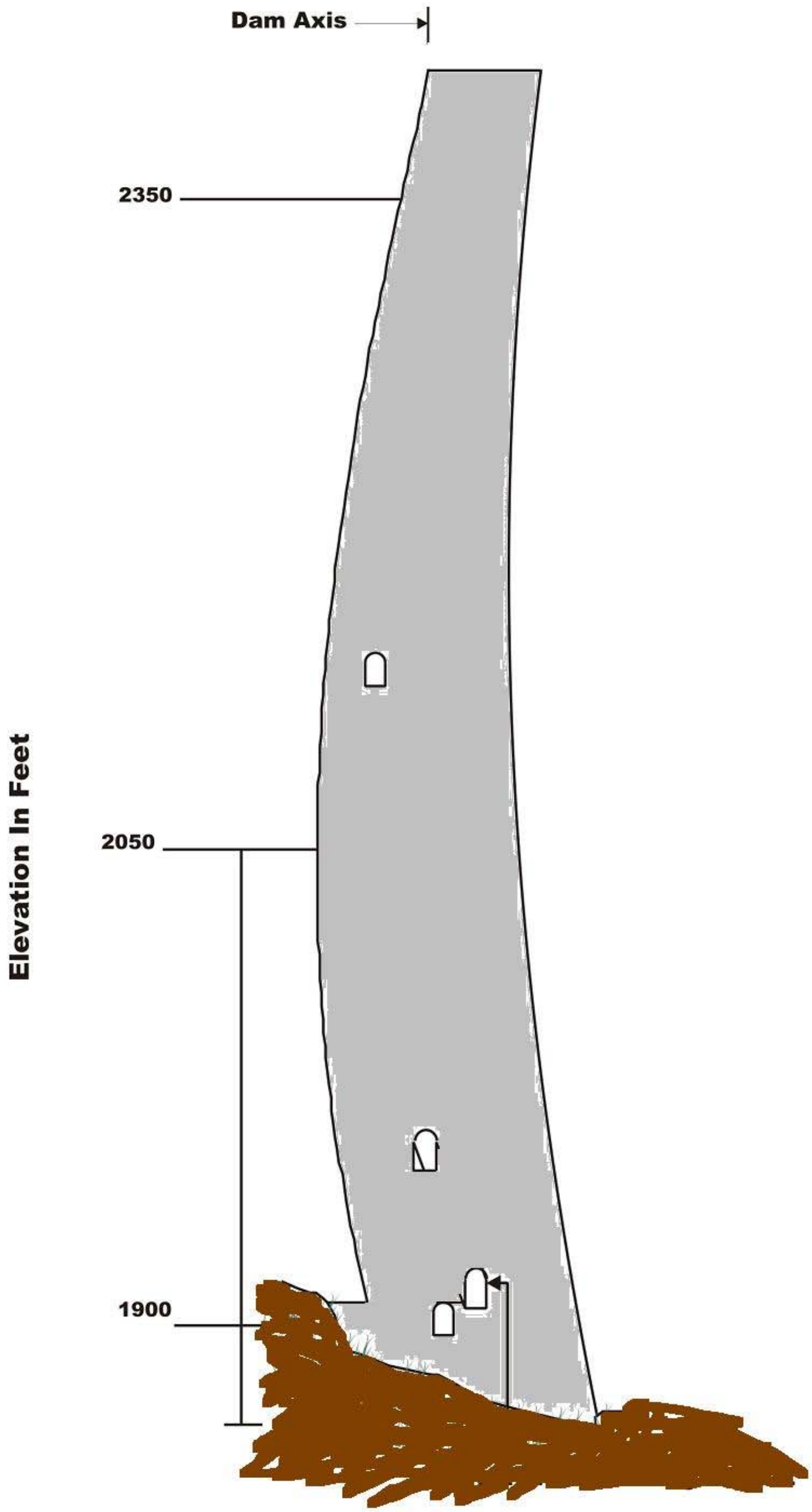
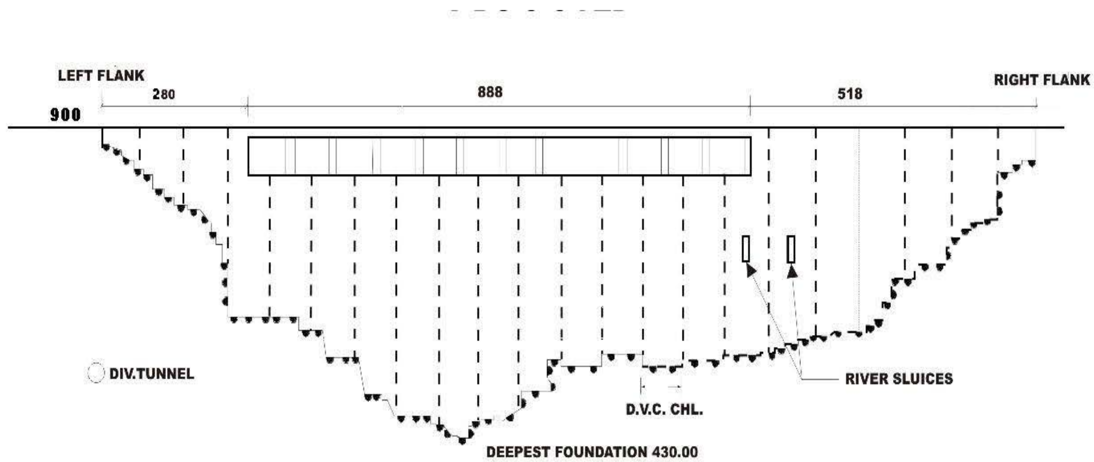


FIGURE 23. SECTION OF IDUKKI ARCH DAM SHOWING GALLERIES,SHAFTS AND ADITS

Srisailam Dam

This dam, built on the river Krishna in Andhra Pradesh, is primarily meant for hydro power generation. Apart from a total generating capacity of 770 MW, the dam also caters to the storage of water for irrigation to about $1.6 \cdot 10^5$ hectares. Though the narrowness of the gorge at the dam suggests the adoption of an arch dam, this idea was ruled out as the abutment rocks were not considered competent enough to take the arch thrusts. The dam was finalized to be of concrete- gravity type. A general layout of the project is shown in Figure 24. An upstream view of the dam along with typical sections, are shown in Figure 25.

FIGURE 24. GENERAL LAYOUT OF SRISAILAM DAM ON RIVER KRISHNA



Logitudinal section of the Srisailam Dam along reference line

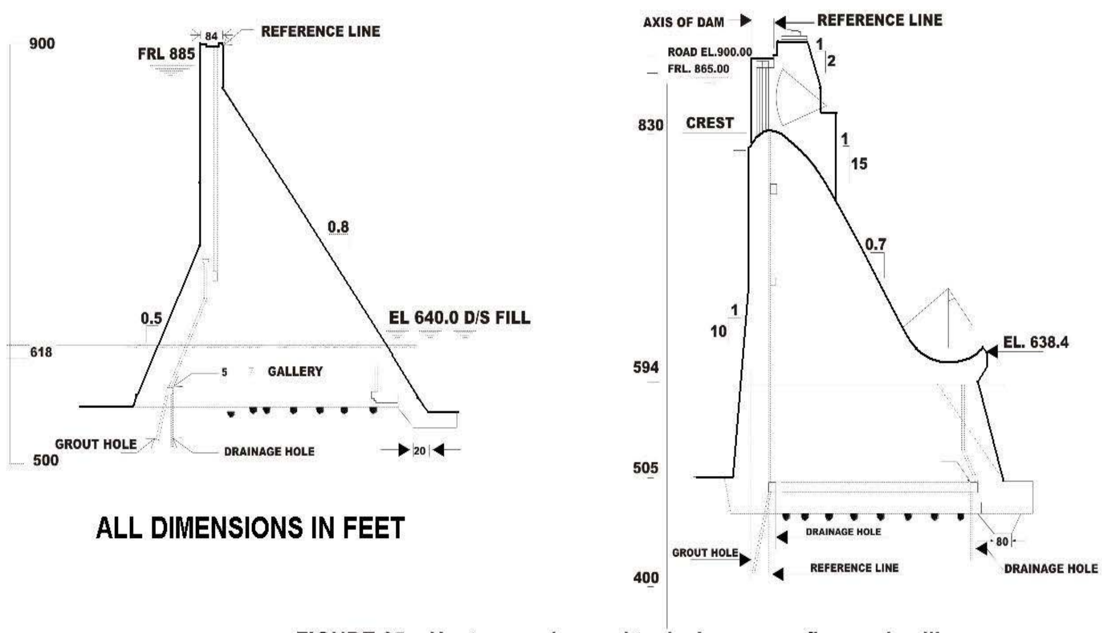


FIGURE 25. Upstream view and typical non- overflow and spillway sections of Srisailam Dam

Kangsabati and Kumari Dams

These are the part of one common project which utilizes the waters of the river Kangsabati and its tributary Kumari by providing an 11km long earthen embankment dam across the two rivers, as shown in Figure 26. The height above the deepest foundation of the Kumari Dam is 41m and the Kangsabati Dam is 88m, a typical section of which is shown in Figure 27. The spillway of the Kangsabati Dam is located on a high level saddle between two hills, the section through is shown in Figure 28. The project releases water through two canals, one each on either bank, and supplies water to about 340890 hectares.

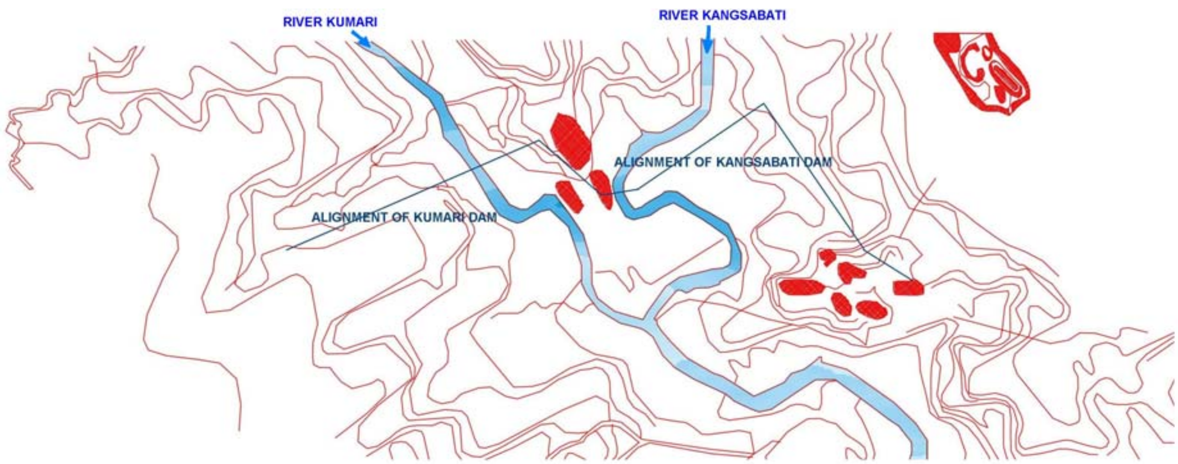


FIGURE 26. General layout of Kumari and Kangsabati dams

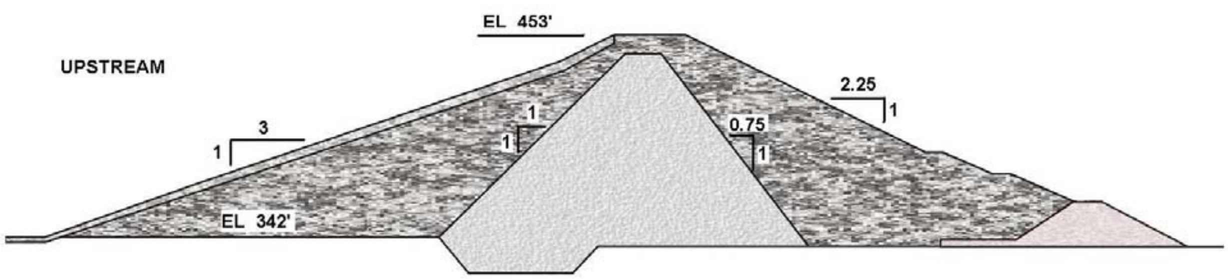


FIGURE 27. Section through Kangsabati embankment dam

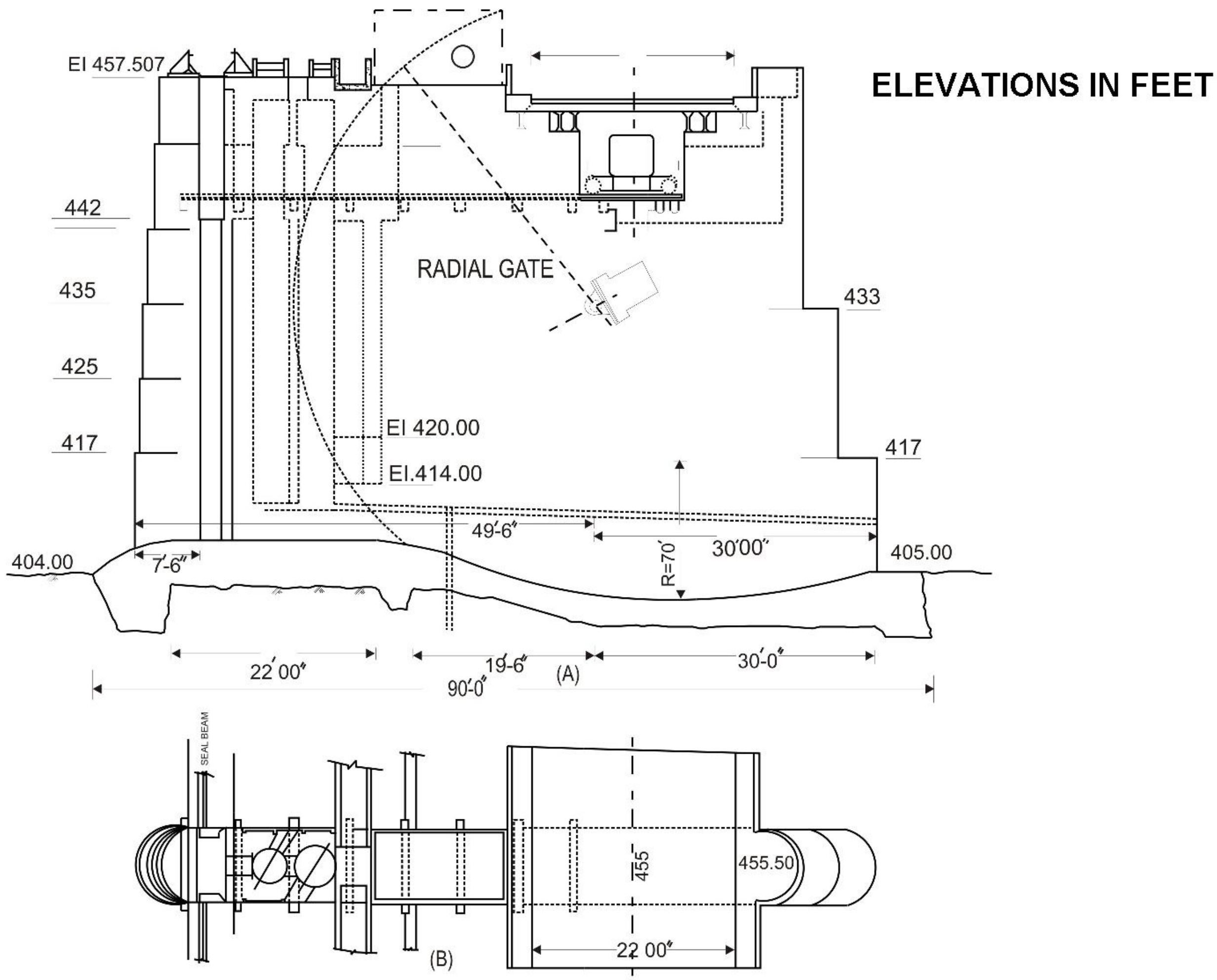


FIGURE 28. (A) SECTION THROUGH THE SPILLWAY OF KANGSABATI PROJECT
(B) PLAN OF PIER

Indira Sagar Dam

This dam on the river Narmada is primarily being built for irrigating around 1230 sq. km. through a canal about 250 km long in the state of Madhya Pradesh. The dam is of concrete gravity type of length 653 m and maximum height above the deepest foundation level is 92m. It is meant to generate power to the tune of 1000 MW. The general layout (Figure 29) shows the dam to be slightly curved towards the upstream in plan, though it is not an arch-type dam. The layout of the dam is shown enlarged in Figure 30. A typical spillway section is shown in Figure 31. It may be interesting to note that the design flood discharge being quite high (83500 cubic meters per sec.) almost all the blocks of the dam are spillway blocks and span continuously from the left to the right flanks of the dam.

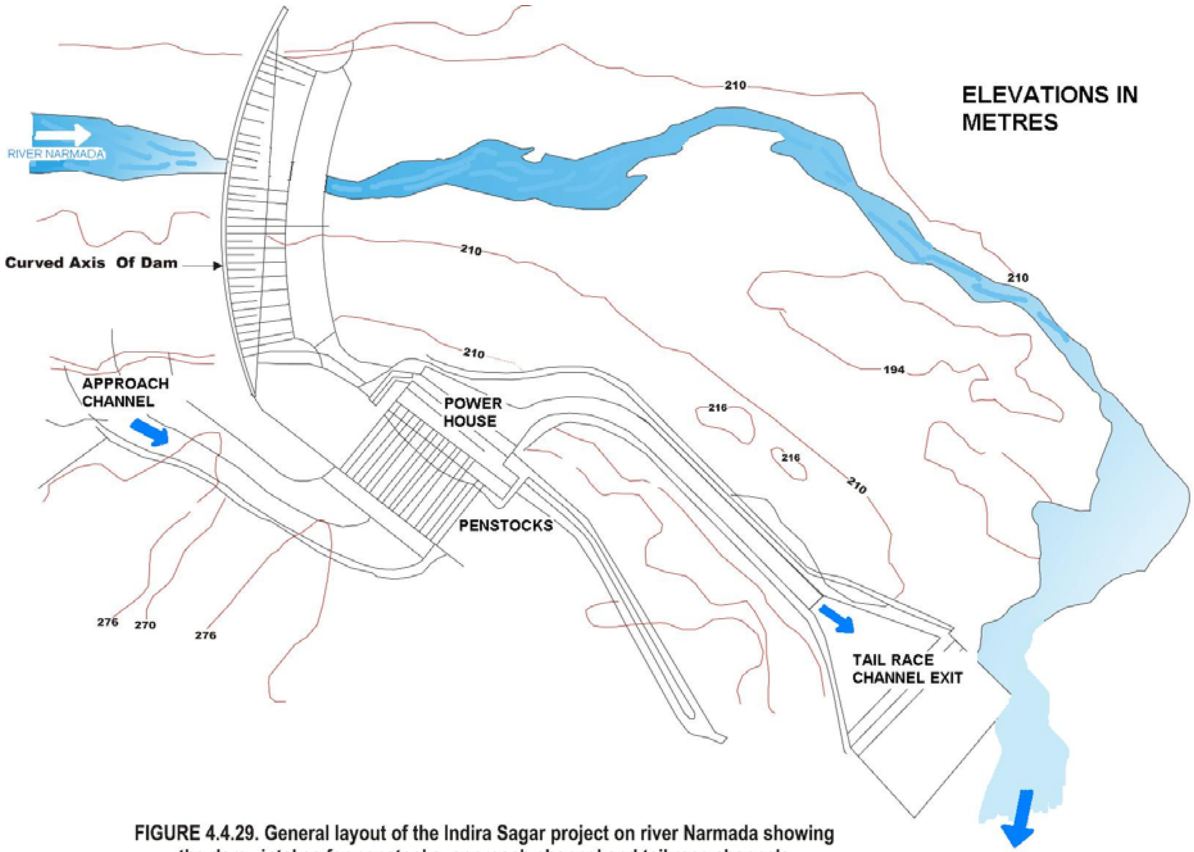


FIGURE 4.4.29. General layout of the Indira Sagar project on river Narmada showing the dam , intakes for penstocks, approach channel and tail race channels

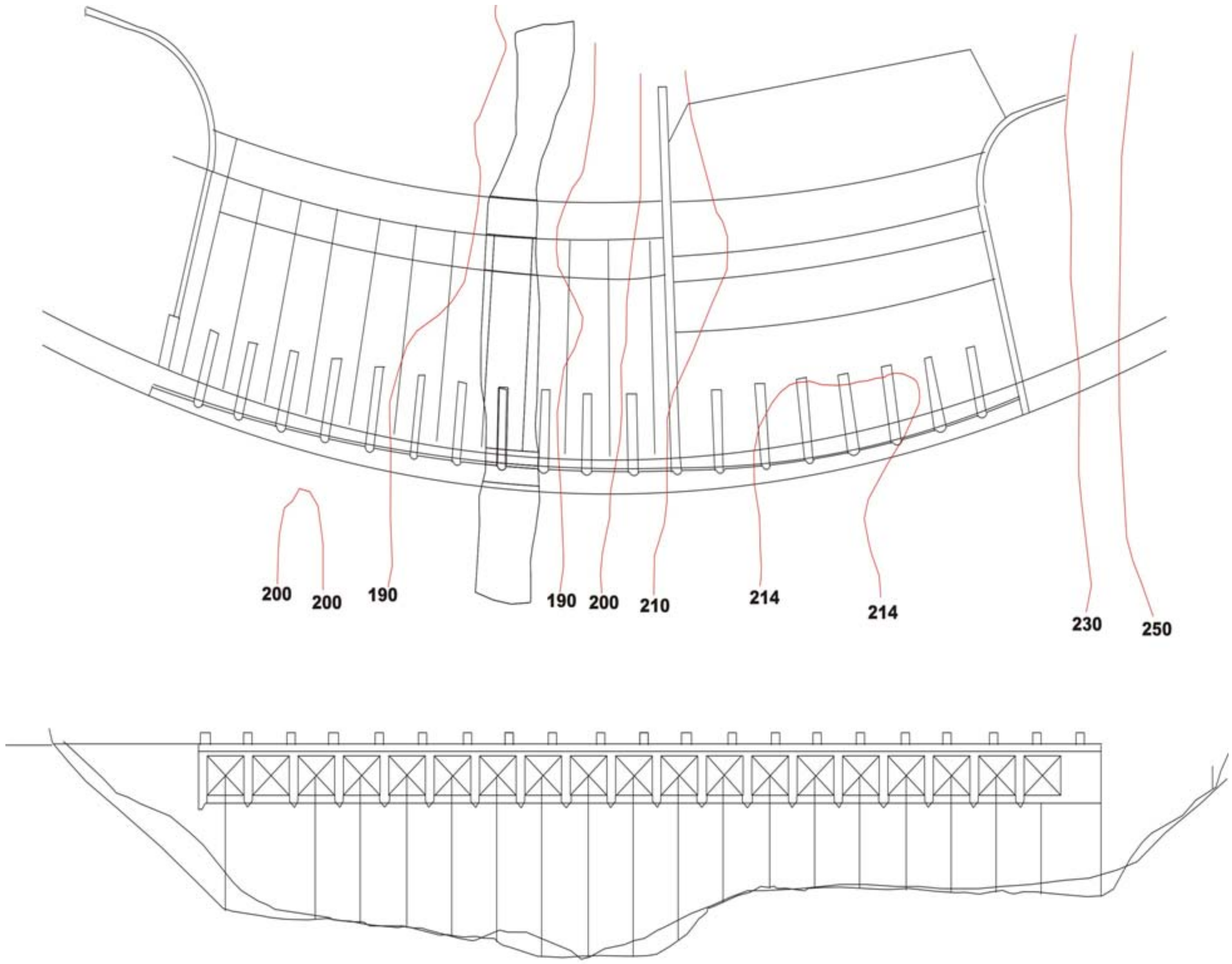


Figure 30. Detailed plan of the Indira Sagar dam and upstream ealevation

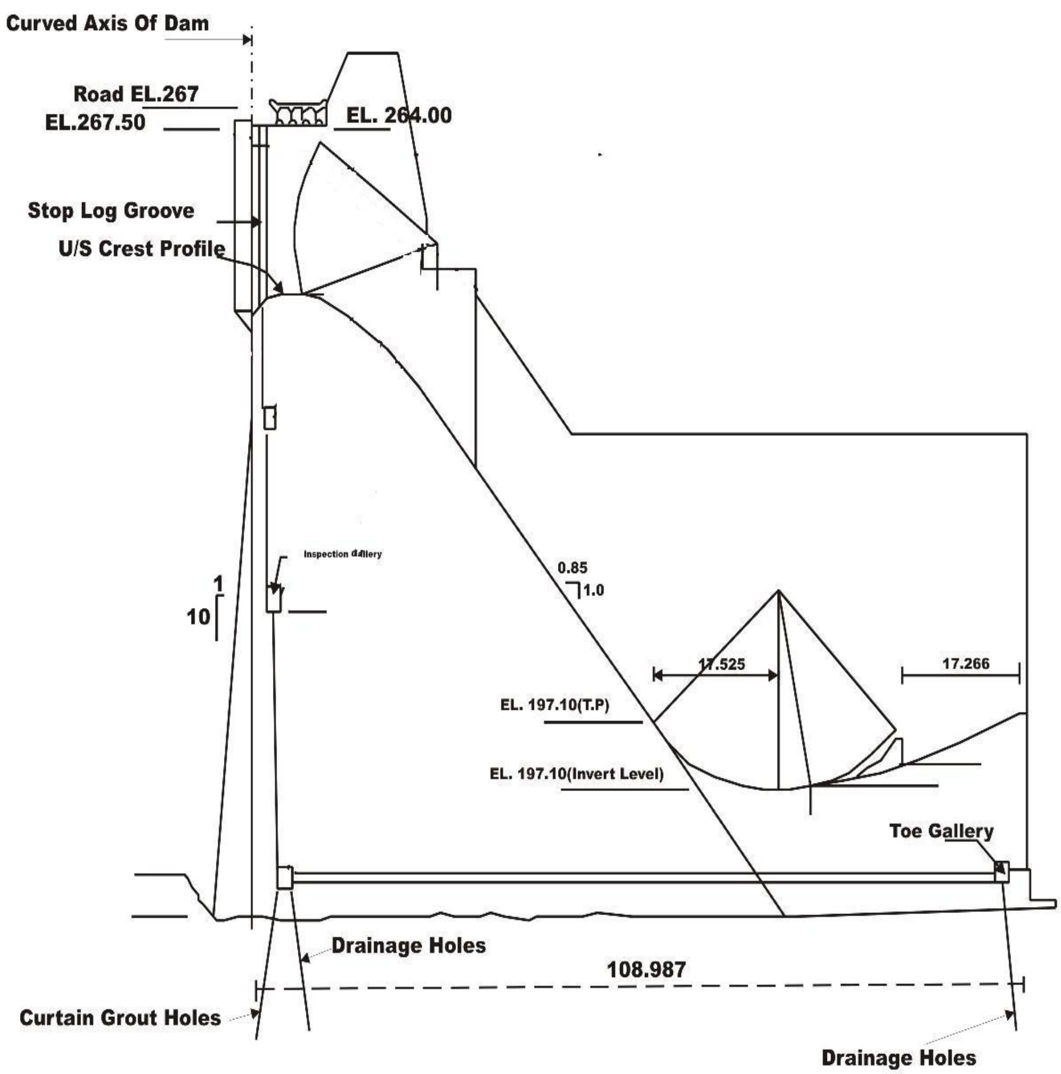


FIGURE 31. **Section view through Indira Sagar dam spilway**

Sardar Sarovar Dam

This dam, also on river Narmada, is meant to convey water through 458 km long canal in Gujarat and 74 km long canal in Rajasthan for drinking and irrigation. The area proposed to be irrigated is around 17920 sq. km. in Gujarat and 730 sq. km. in Rajasthan. The dam is of concrete gravity type, and its general layout and a typical cross section are shown in Figure 32 and Figure 33 respectively. The dam also proposes to develop 1200MW of power using direct flow in the river, and 250 MW through a power house located at the head of the canal. The turbines of the main power house are actually, reversible type that is they generate power when water flows through them but may also be used as if power is fed to them. These reversible pump turbines have been installed to lift water during night by utilizing the excess power in the national grid from a lower reservoir proposed to be made later by constructing a weir downstream.

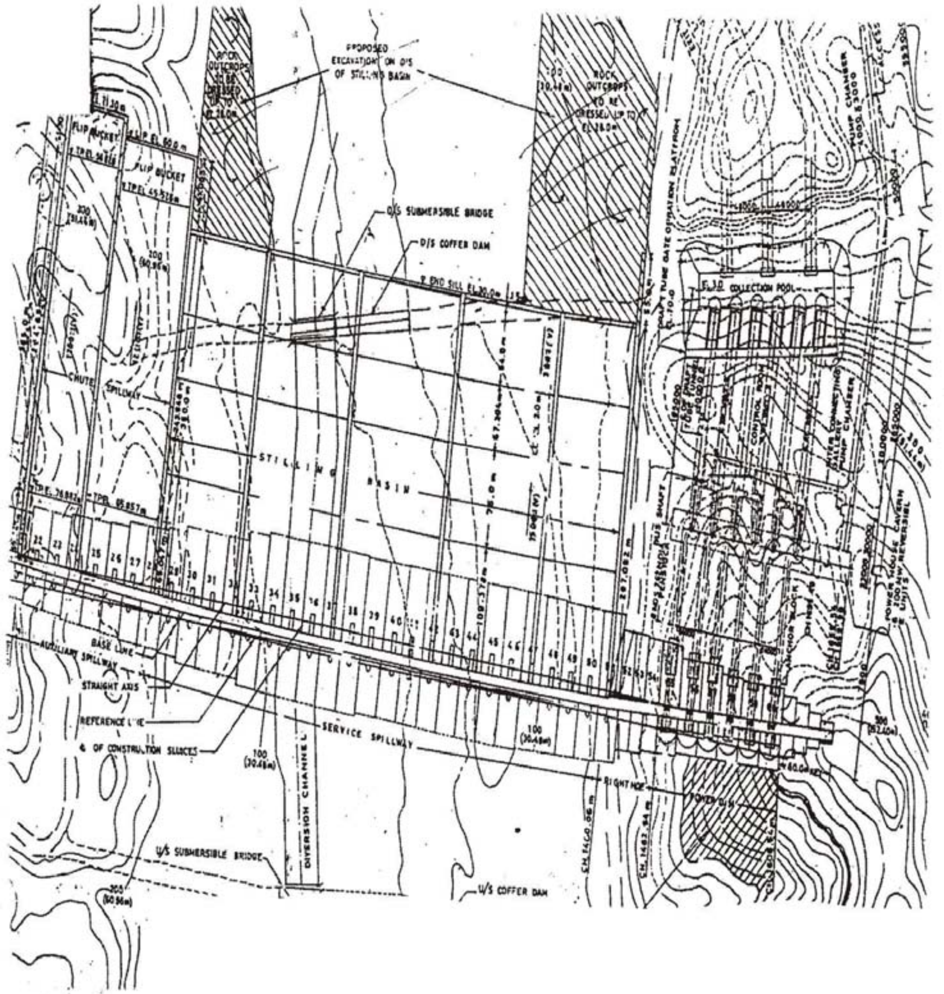


FIGURE 32. LAYOUT OF SARDAR SAROVAR DAM ON RIVER NARMADA AND ITS UNDERGROUND POWER HOUSES WITH CORRESPONDING INTAKES AND TAIL RACE TUNNELS / CHANNELS

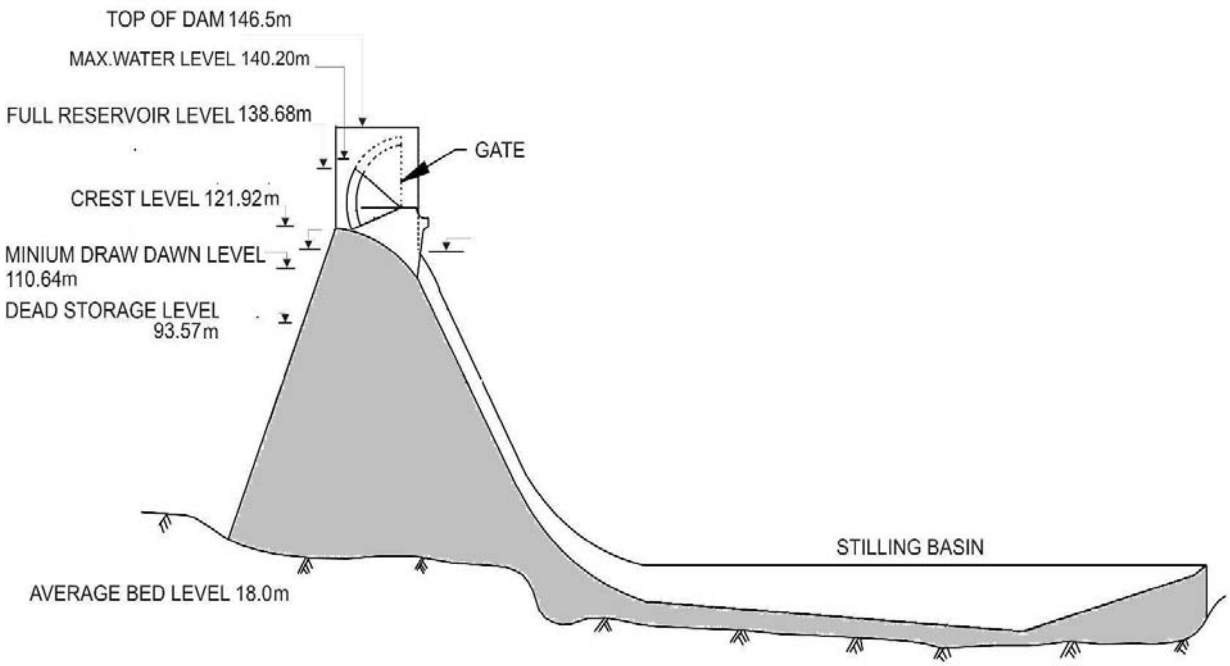


FIGURE 33.SECTIONAL VIEW THROUGH SARDAR SAROVAR DAM SPILLWAY

References

- IS: 6926-1996 "Diamond core drilling – site investigation for river valley projects- code of practice"
- IS: 4078-1967 "Code of practice for indexing and storage of drill cores"
- IS: 4464-1967 "Code of practice presentation of drilling information and core description in foundation investigation"
- IS: 6955-1973 "Code of practice for subsurface exploration for earth and rock fill dams"
- IS: 4453-1967 "Code of practice for exploration by pits, trenches, drifts and shafts"
- IS: 1892-1962 "Code of practice for site investigation for foundations"

Module 4

Hydraulic Structures for Flow Diversion and Storage

Lesson 5

Planning Of Water Storage Reservoirs

Instructional objectives

On completion of this lesson, the student shall learn:

1. The usual classification of the zones of a reservoir
2. The primary types of reservoirs and their functions
3. The steps for planning reservoirs
4. Effect of sedimentation in reservoirs
5. What are the geological explorations required to be carried out for reservoirs
6. How to determine the capacities of reservoirs
7. How to determine the dead, live and flood storages of reservoirs
8. How to reduce the loss of water from reservoirs
9. How to control sedimentation of reservoirs
10. The principles to be followed for reservoir operations

4.5.0 Introduction

As seen from Lesson 4.4, water storage reservoirs may be created by constructing a dam across a river, along with suitable appurtenant structures. How ever, in that lesson not much was discussed about fixing the size of reservoir based on the demand for which it is being constructed. Further, reservoirs are also meant to absorb a part of flood water and the excess is discharged through a spillway. It is also essential to study the relation between flood discharge, reservoirs capacity and spillway size in order to propose an economic solution to the whole project. These and topics on reservoir sedimentation have been discussed in this lesson which shall give an idea as to how a reservoir should be built and optimally operated.

Fundamentally, a reservoir serves to store water and the size of the reservoir is governed by the volume of the water that must be stored, which in turn is affected by the variability of the inflow available for the reservoir. Reservoirs are of two main categories: (a) Impounding reservoirs into which a river flows naturally, and (b) Service or balancing reservoirs receiving supplies that are pumped or channeled into them artificially. In general, service or balancing reservoirs are required to balance supply with demand. Reservoirs of the second type are relatively small in volume because the storage required by them is to balance flows for a few hours or a few days at the most. Impounding or storage reservoirs are intended to accumulate a part of the flood flow of the river for use during the non-flood months. In this lesson, our discussions would be centered on these types of reservoirs.

4.5.1 Reservoir storage zone and uses of reservoir

The storage capacity in a reservoir is nationally divided into three or four parts (Figure 1) distinguished by corresponding levels.

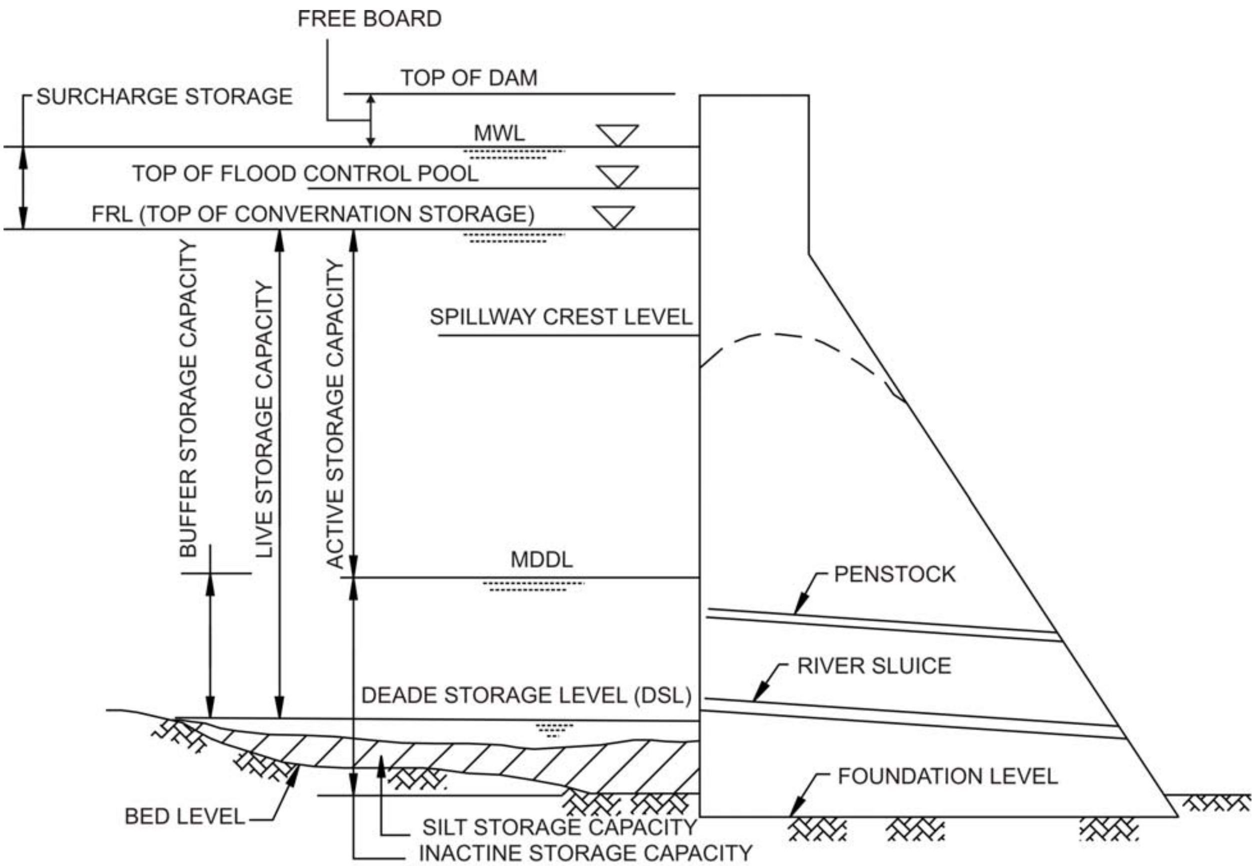


FIGURE 1. SCHEMATIC DIAGRAM SHOWING STORAGE ZONES (OF CAPACITY) NOMENCLATURE

These specific levels and parts are generally defined as follows:

***Full Reservoir Level (FRL)*:** It is the level corresponding to the storage which includes both inactive and active storages and also the flood storage, if provided for. In fact, this is the highest reservoir level that can be maintained without spillway discharge or without passing water downstream through sluice ways.

***Minimum Drawdown Level (MDDL)*:** It is the level below which the reservoir will not be drawn down so as to maintain a minimum head required in power projects.

***Dead Storage Level (DSL)*:** Below the level, there are no outlets to drain the water in the reservoir by gravity.

Maximum Water Level (MWL)*:** This id the water level that is ever likely to be attained during the passage of the design flood. It depends upon the specified initial reservoir level and the spillway gate operation rule. This level is also called sometimes as the ***Highest Reservoir Level or the ***Highest Flood Level***.

***Live storage*:** This is the storage available for the intended purpose between Full Supply Level and the Invert Level of the lowest discharge outlet. The Full Supply Level is normally that level above which over spill to waste would take place. The minimum operating level must be sufficiently above the lowest discharge outlet to avoid vortex formation and air entrainment. This may also be termed as the volume of water actually available at any time between the Dead Storage Level and the lower of the actual water level and Full Reservoir Level.

***Dead storage*:** It is the total storage below the invert level of the lowest discharge outlet from the reservoir. It may be available to contain sedimentation, provided the sediment does not adversely affect the lowest discharge.

***Outlet Surcharge or Flood storage*:** This is required as a reserve between Full Reservoir Level and the Maximum Water level to contain the peaks of floods that might occur when there is insufficient storage capacity for them below Full Reservoir Level.

Some other terms related to reservoirs are defined as follows:

***Buffer Storage*:** This is the space located just above the Dead Storage Level up to Minimum Drawdown Level. As the name implies, this zone is a buffer between the active and dead storage zones and releases from this zone are made in dry situations to cater for essential requirements only. Dead Storage and Buffer Storage together is called Interactive Storage.

***Within-the-Year Storage*:** This term is used to denote the storage of a reservoir meant for meeting the demands of a specific hydrologic year used for planning the project.

***Carry-Over Storage*:** When the entire water stored in a reservoir is not used up in a year, the unused water is stored as carry-over storage for use in subsequent years.

***Silt / Sedimentation zones*:** The space occupied by the sediment in the reservoir can be divided into separate zones. A schematic diagram showing these zones is illustrated in Figure 2 (as defined in IS: 5477).

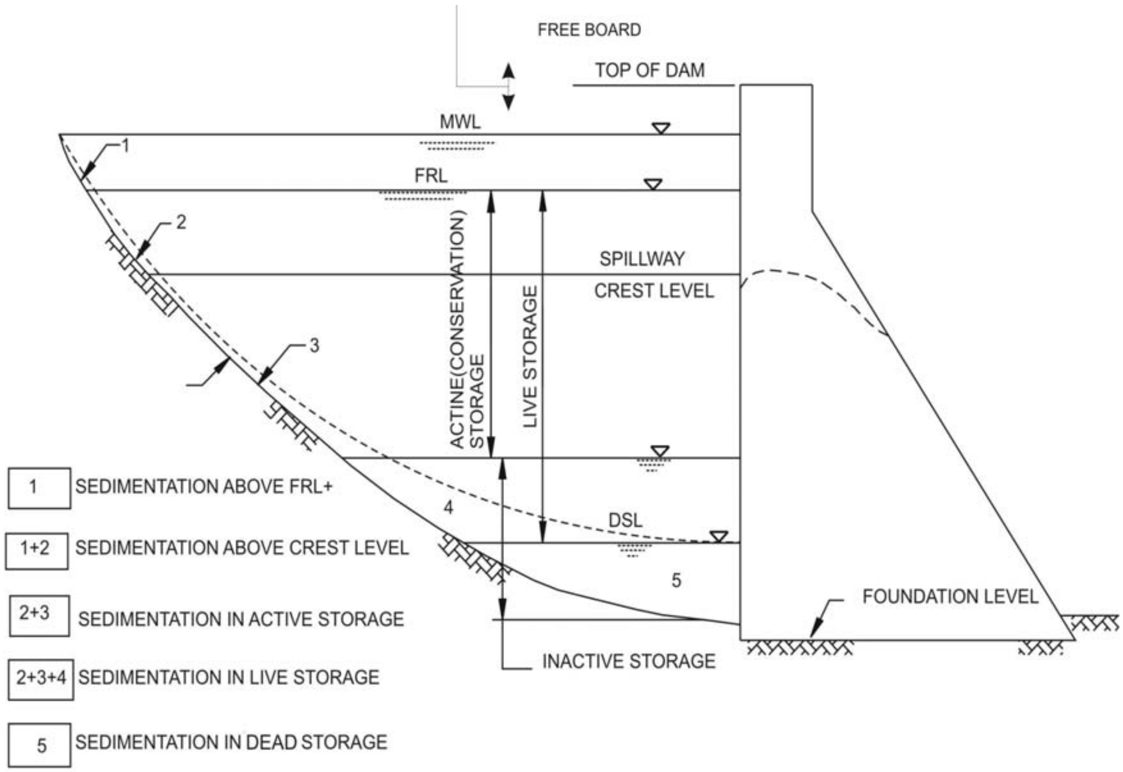


FIGURE 2. SCHEMATIC DIAGRAM SHOWING ZONES OF RESEERVOIR SEDIMENTION

***Freeboard*:** It is the margin kept for safety between the level at which the dam would be overtopped and the maximum still water level. This is required to allow for settlement of the dam, for wave run up above still water level and for unforeseen rises in water level, because of surges resulting from landslides into the reservoir from the peripheral hills, earthquakes or unforeseen floods or operational deficiencies.

The functions of reservoirs are to provide water for one or more of the following purposes. Reservoirs that provide water for a combination of these purpose, are termed as 'Multi Purpose' reservoirs.

- ***Human consumption*** and/or ***industrial use:***
- ***Irrigation:*** usually to supplement insufficient rainfall.
- ***Hydropower:*** to generate power and energy whenever water is available or to provide reliable supplies of power and energy at all times when needed to meet demand.
- ***Pumped storage hydropower schemes*:** in which the water flows from an upper to a lower reservoir, generating power and energy at times of high demand through turbines, which may be reversible, and the water is pumped back to the upper

reservoir when surplus energy is available. The cycle is usually daily or twice daily to meet peak demands. Inflow to such a reservoir is not essential, provided it is required to replace water losses through leakage and evaporation or to generate additional electricity. In such facilities, the power stations, conduits and either or both of the reservoirs could be constructed underground if it was found to do so.

- ***Flood control:*** storage capacity is required to be maintained to absorb foreseeable flood inflows to the reservoirs, so far as they would cause excess of acceptable discharge spillway opening. Storage allows future use of the flood water retained.

- ***Amenity use:*** this may include provision for boating, water sports, fishing, sight seeing.

Formally, the Bureau of Indian Standards code IS: 4410 (part 6)1983 "Glossary of terms relating to river valley projects -Reservoirs" defines the following types of reservoirs:

- ***Auxiliary or Compensatory Reservoir:*** A reservoir which supplements and absorbed the spill of a main reservoir.

- ***Balancing Reservoirs:*** A reservoir downstream of the main reservoir for holding water let down from the main reservoir in excess of that required for irrigation, power generation or other purposes.

- ***Conservation Reservoir:*** A reservoir impounding water for useful purposes, such as irrigation, power generation, recreation, domestic, industrial and municipal supply etc.

- ***Detention Reservoir:*** A reservoir where in water is stored for a relatively brief period of time, past of it being retained until the stream can safely carry the ordinary flow plus the released water. Such reservoirs usually have outlets without control gates and are used for flood regulation. These reservoirs are also called as the ***Flood Control Reservoir*** or ***Retarding Reservoir***.

- ***Distribution Reservoir:*** A reservoir connected with distribution system a water supply project, used primarily to care for fluctuations in demand which occur over short periods and as local storage in case of emergency such as a break in a main supply line failure of a pumping plant.

- ***Impounding or Storage Reservoir:*** A reservoir with gate-controlled outlets wherein surface water may be retained for a considerable period of time and released for use at a time when the normal flow of the stream is in sufficient to satisfy requirements.

- ***Multipurpose Reservoir:*** A reservoir constructed and equipped to provide storage and release of water for two or more purposes such as irrigation, flood control, power generation, navigation, pollution abatement, domestic and industrial water supply, fish culture, recreation, etc.

It may be observed that some of these objectives may be incompatible in combination. For example, water may has to be released for irrigation to suit crop growing seasons, while water releases for hydropower are required to suit the time of public and industrial demands. The latter will be affected not only by variations in economic conditions but also by variations over a day and night cycle.

Compatibility between irrigation demand and flood control strategy in operating a reservoir is even more difficult for a reservoir which intends to serve both, like the Hirakud Dam reservoir on the river Mahanadi. Flood wave moderation requires that the reservoir be emptied as much as possible so that it may absorb any incoming flood peak. However, this decision means reducing the water stored for irrigation. Usually, such a reservoir would be gradually emptied just before the arrival of monsoon rains, anticipating a certain flood and hoping that the reservoir would be filled to the brim at the end of the flood season. However, this anticipation may not hold good for all years and the reservoir does not get filled up to the optimal height. On the other hand, if the reservoir is not depleted sufficiently well, and actually a flood of high magnitude arrives, then the situation may lead to the flood inundations on the downstream.

4.5.2 Planning of reservoirs

The first step in planning the construction of a reservoir with the help of a dam is for the decision makers to be sure of the needs and purposes for which the reservoir is going to be built together with the known constraints (including financial), desired benefits. There may be social constraints, for examples people's activism may not allow a reservoir to be built up to the desired level or even the submergence of good agricultural level may be a constraint. Some times, the construction of a dam may be done that is labour intensive and using local materials, which helps the community for whom the dam is being built. This sort of work is quite common in the minor irrigation departments of various steps, especially in the drought prone areas. The Food-for-Work schemes can be utilised in creating small reservoirs that helps to serve the community. In a larger scale, similar strategy was adopted for the construction of the Nagarjuna Sagar Dam on the River Krishna, which was built entirely of coursed rubble masonry and using manual labour in thousands.

The second step is the assembly of all relevant existing information, which includes the following:

- Reports of any previous investigations and studies, if any.
- Reports on projects similar to that proposed which have already been constructed in the region.
- A geographical information system (GIS) for the area of interest may be created

using a base survey map of the region.

- Topographical data in the form of maps and satellite pictures, which may be integrated within the GIS.
- Geological data in the form of maps and borehole logs, along with the values of relevant parameters.
- Seismic activity data of the region that includes recorded peak accelerations or ground motion record.
- Meteorological and hydrological data - of available parameters like rainfall, atmospheric and water temperatures, evaporation, humidity, wind speed, hours of sunshine, river flows, river levels, sediment concentration in rivers, etc.
- For water supply projects, data on population and future population growth based on some acceptable forecast method, industrial water requirement and probable future industrial development.
- For irrigation projects, data on soils in the project area and on the crops already grown, including water requirement for the crops.
- For hydropower projects, data on past demand and forecasts of future public and industrial demand for power and energy; data on existing transmission systems, including transmission voltage and capacity.
- Data on flora and fauna in the project and on the fish in the rivers and lakes, including data on their migratory and breeding habits.
- Data on tourism and recreational use of rivers and lakes and how this may be encouraged on completion of the proposed reservoir.

As may be noted, some of the data mentioned above would be needed to design and construct the dam and its appurtenant structures which would help to store water behind the reservoir. However, there are other data that decides the following:

- How large the reservoir should be and, consequently, what should be the dam height?
- What should be the size of the spillway and at what elevation the crest level of the spillway be located?
- How many and at what levels sluices be provided and they should be of what sizes?

Two important aspects of reservoirs planning: Sedimentation Studies and Geological Explorations are described in detail in the following section.

4.5.3 Effect of sedimentation in planning of reservoirs

It is important to note that storage reservoirs built across rivers and streams loose their capacity on account of deposition of sediment. This deposition which takes place progressively in time reduces the active capacity of the reservoir to provide the outputs of water through passage of time. Accumulation of sediment at or near the dam may interfere with the future functioning of water intakes and hence affects decisions regarding location and height of various outlets. It may also result in greater inflow of into canals / water conveyance systems drawing water from the reservoir. Problems of rise in flood levels in the head reaches and unsightly deposition of sediment from recreation point of may also crop up in course of time.

In this regard, the Bureau of Indian Standard code IS: 12182 - 1987 “Guidelines for determination of effects of sedimentation in planning and performance of reservoir” is an important document which discusses some of the aspects of sedimentation that have to be considered while planning reservoirs. Some of the important points from the code are as follows:

While planning a reservoir, the degree of seriousness and the effect of sedimentation at the proposed location has to be judged from studies, which normally combination consists of:

1. Performance Assessment (Simulation) Studies with varying rate of sedimentation.
2. Likely effects of sedimentation at dam face.

In special cases, where the effects of sedimentation on backwater levels are likely to be significant, backwater studies would be useful to understand the size of river water levels. Similarly, special studies to bring out delta formation region changes may be of interest. The steps to be followed for performance assessment studies with varying rates of sedimentation are as follows:

a. Estimation of annual sediment yields into the reservoir or the average annual sediment yield and of trap efficiency expected.
b. Distribution of sediment within reservoir to obtain a sediment elevation and capacity curve at any appropriate time.
c. Simulation studies with varying rates of sedimentation.
d. Assessment of effect of sedimentation.

In general, the performance assessment of reservoir projects has to be done for varying hydrologic inputs to meet varying demands. Although analytical probability based methods are available to some extent, simulation of the reservoir system is the standard method. The method is also known as the working tables or sequential routing. In this method, the water balance of the reservoir s and of other specific locations of water use and constraints in the systems are considered. All inflows to and outflows from the reservoirs are worked out to decide the changed storage during the period. In simulation studies, the inflows to be used may be either historical inflow series, adjusted for future up stream water use changes or an adjusted synthetically generated series.

4.5.4 Procedure for planning a new reservoir

The standard procedure that needs to be carried out for planned storages requires an assessment of the importance of the problem to classify the reservoir sedimentation problem as insignificant, significant, or serious. Assessment of reservoir sedimentation problem, in a particular case may be made by comparing the expected average annual volume of sediment deposition with the gross capacity of the reservoir planned. If the ratio is more than 0.5 percent per year, the problem is usually said to be serious and special care is required in estimating the sediment yields from the catchment. If it is less than 0.1 percent per year, the problem of siltation may be insignificant and changes in reservoir performance. For cases falling between these two limits, the sedimentation problem is considered significant and requires further studies.

The following studies are required if the problem is insignificant:

1. No simulation studies with sediment correlation is necessary**.**
2. The feasible service time for the project may be decided. Sediment distribution studies to ensure that the new zero-elevation does not exceed the dead storage level may be made.

In the above, the following terms have been used, which are explained below:

- **Feasible Service Time**: For a special purpose, the period or notional period for which a reservoir is expected to provide a part of the planned benefit in respect of storage in the reservoirs being impaired by sedimentation. Customarily, it is estimated as the time after which the new zero elevation of the reservoir would equal the sill of the outlet relevant for the purpose.

- **New Zero Elevation**: The level up to which all the available capacity of the reservoir is expected to be lost due to progressive sedimentation of the reservoir up to the specified time. The specified time should be any length of time such as Full Service Time, Feasible Service time, etc.

- **Full Service Time**: For a specified purpose, the period or notional period for which the reservoir provided is expected to provide, a part of the full planned benefit inspite of sedimentation.

The following studies are required if the problem of sedimentation in the reservoir is assessed to be significant, but not serious.

1. Both the full service time and feasible service time for the reservoir may be decided.
2. Simulation studies for conditions expected at the end of full service time may be made to ensure that firm outputs with required depend ability are obtained. The studies used also assess non-dependable secondary outputs, if relevant, available at the end of this period. Studies without sedimentation, with the same firm outputs should bring out the additional potential secondary outputs which may be used, if required in economic analysis, using a linear decrease of these additional benefits over the full service time.
3. No simulation studies beyond full service time, is required.
4. Sediment distribution studies required for feasible service time are essential.

The following studies are required if the problem of sedimentation is serious.

1. All studies described for the 'Significant' case have to be made.
2. The secondary benefits available in the initial years should be more in such cases. If they are being utilized, for a proper assessment of the change of these, a simulation at half of full service time should be required.
3. In these cases, the drop of benefits after the full service time may be sharper. To bring out these effects, a simulation of the project at the end of the feasible service time is required to be done.

4.5.5 Life of reservoir and design criteria

A reservoir exists for a long time and the period of its operation should normally check large technological and socio-economic changes. The planning assumptions about the exact socio-economic outputs are, therefore, likely to be changed during operation, and similarly, the implication of socio-economic differences in the output due to sedimentation are difficult to access**.** The ever increasing demands due to both increase of population and increases in per capita needs are of a larger magnitude than the reductions in outputs, if any, of existing reservoirs. Thus effects of sedimentation, obsolescence, structural deterioration, etc. of reservoirs may require adjustments in future developmental plans and not simply replacement projects to bring back the lost potential. On a regional or national scale, it is the sufficiency of the total economic outputs, and not outputs of a particular project which is relevant. However, from local considerations, the reduction of outputs of reservoir like irrigation and flood control may cause a much greater degree of distress to the population which has got used to better socio-economic conditions because of the reservoir.

'Life' strictly is a term which may be used for system having two functional states 'ON' and 'OFF'. Systems showing gradual degradation of performance and not showing any sudden non-functional stage have no specific life period. Reservoirs fall in the later category.

The term 'life of reservoir' as loosely used denotes the period during which whole or a specified fraction of its total or active capacity is lost. In calculating this life, the progressive changes in trap efficiency towards the end of the period are commonly not considered. In some of the earlier projects, it has been assumed that all the sedimentation would occur only in the dead storage pocket and the number of years in which the pocket should be filled under this assumption was also sometimes termed as the life of reservoir. This concept was in fact used to decide the minimum size of the pocket. Under this concept, no effect of sedimentation should be felt within the live storage of the reservoir. It has subsequently been established that the silt occupies the space in the live storage of reservoir as well as the dead storage.

If the operation of the reservoir becomes impossible due to any structural defects, foundation defects, accidental damages, etc., this situation should also signify the end of the feasible service time. Before the expiry of this feasible service time, it may be possible to make large changes in the reservoir (for example, new higher level outlets, structural strengthening, etc.) or other measures, if it is economically feasible to do so. If these studies are done, the feasible service time may be extended.

4.5.6 Geological explorations for reservoir sites

In Lesson 4.4, geological exploration procedures for constructions of dams were discussed in detail. Though a dam is constructed to build a reservoir, a reservoir has a large area of spread and contained in a big chunk of the river valley upstream of the dam. Hence, while identifying a suitable site for a proposed dam, it is of paramount importance that the proposed reservoir site is also thoroughly investigated and explored. The basis of planning for such explorations is to have a rapid economical and dependable pre-investment evaluation of subsurface conditions. It is also necessary that a degree of uniformity be followed while carrying out subsurface explorations so that the frame of reference of the investigation covers all requisite aspects. In view of above, the Bureau of Indian Standards has brought out a code IS: 13216 - 1991 "Code of practice for geological exploration for reservoir sites", that discusses the relevant aspects. According to the code since reservoir projects in river valleys are meant to hold water; therefore, the following aspects of the reservoirs have to be properly investigated

(a) Water tightness of the basins
(b) Stability of the reservoir rim
(c) Availability of construction material in the reservoir area
(d) Silting
(e) Direct and indirect submergence of economic mineral wealth
(f) Seismo-techtonics

These aspects are determined through investigations carried out by surface and sub-surface exploration of proposed basin during the reconnaissance, preliminary investigation, detailed investigation, construction and post-construction stages of the project. The two basic stages of investigation: reconnaissance and preliminary investigations are explained below:

Reconnaissance

In the reconnaissance stage, the objective of investigation is to bring out the overall geological features of the reservoir and the adjacent area to enable the designers, construction engineers and geologists to pinpoint the geotechnical and ecological problems which have to be tackled. The scale of geological mapping for this stage of work need not be very large and the available geological maps on 1:50,000 or 1: 250,000 scales may be made use of. It is advantageous to carry out photo geological interpretation of aerial photographs of the area, if available. If a geological map of the area is not available, a traverse geological map should be prepared at this stage preferably using the aerial photos as base maps on which the engineering evaluation of the various geotechnical features exposed in the area should be depicted.

A topographical index map on 1: 50 000 scales should be used at this stage to delineate the areas which would require detailed study, subsequently.

To prevent an undesirable amount of leakage from the reservoir, the likely zones of such leakage, such as major dislocations and pervious or cavernous formations running across the divide of the reservoir should be identified at this stage of investigation for further detailed investigations.

Major unstable zones, particularly in the vicinity of the dam in tight gorges, should be identified at this stage for carrying out detailed investigations for the stability of the reservoir rim.

The locations for suitable construction material available in the reservoir area should be pin pointed at this stage so that after detailed surveys such materials can be exploited for proper utilisation during the construction stage prior to impounding of reservoir.

The rate of silting of the reservoir is vital for planning the height of the dam and working out the economic life of the project. Since the rate of silting, in addition to other factors, is dependent on the type of terrain in the catchment area of the reservoir, the major geological formations and the ecological set up should be recognized at this stage to enable a more accurate estimation of the rate of silting of the reservoir. For example, it should be possible to estimate at this stage that forty percent of the catchment of a storage dam project is covered by Quaternary sediment and that this is a condition which is likely to a yield a high silt rate or that ninety percent of the catchment of another storage dam project is composed of igneous and metamorphic rocks and is likely to yield a relatively low sediment rate. This information will also be useful in examining whether or not tributaries flowing for long distances through soft or unconsolidated formations, prior to forming the proposed reservoir, can be avoided and if not, what remedial measures can be taken to control the silt load brought by these tributaries.

The impounding of a reservoir may submerge economic/strategic mineral deposits occurring within the reservoir area or the resultant rise in the water table around the reservoir may cause flooding, increased seepage in quarries and mines located in the area and water logging in other areas. It is, therefore, necessary that the economic mineral deposits, which are likely to be adversely affected by the reservoir area, are identified at this stage of the investigation. For example, if an underground working is located close to a proposed storage reservoir area, it should be identified for regular systematic geo-hydrological studies subsequently. These studies would establish whether the impoundment of the water in the reservoir had adversely affected the underground working or not. References should also be made to various agencies dealing with the economic minerals likely to be affected by the impoundment in the reservoir for proper evaluation of the problem and suitable necessary action.

A dam and its reservoir are affected by the environment in which they are located and in turn they also change the environment. Impoundment of a reservoir sometimes results in an increase of seismic activity at, or near the reservoir. The seismic activity may lead to microtremors and in some cases lead to earthquakes of high magnitude. It is, therefore, necessary to undertake the regional seismotechtonic study of the project area. The faults having active seismic status should be delineated at this stage.

Simultaneous action to plan and install a network of seismological observatories encompassing the reservoir area should also be taken.

Preliminary Investigation

The object of preliminary investigation of the reservoir area is to collect further details of the surface and subsurface geological conditions, with reference to the likely problems identified during the reconnaissance stage of investigation by means of surface mapping supplemented by photo geological interpretation of aerial photographs, hydro geological investigations, geophysical investigations, preliminary subsurface exploration and by conducting geo-seismological studies of the area.

On the basis of studies carried out during the reconnaissance stage it should be possible to estimate the extent of exploration that may be required during the preliminary stage of investigation including the total number of holes required to be drilled and the total number and depth of pits, trenches and drifts as also the extent of geophysical surveys which may be necessary. For exploration by pits, trenches, drifts and shafts guidelines laid down in IS 4453: 1980 Name of IS code should be followed.

The potential zones of leakage from the reservoir and the lateral extent of various features, such as extent of aeolian sand deposits, glacial till, land slides, major dislocations or pervious and cavernous formations running across the divide, should be delineated on a scale of 1: 50000.

The geo-hydrological conditions of the reservoir rim should be established by surface and sub-surface investigation as well as inventory, as a free ground water divide rising above the proposed level of the reservoir is a favourable condition against leakage from the reservoir. The level of water in a bore hole should be determined as given in IS 6935: 1973.

The extension of various features at depth, wherever necessary, is investigated by geophysical exploration and by means of pits, trenches, drifts and drill holes. For example, the resistivity survey should be able to identify water saturated zones. The nature of the material is investigated by means of laboratory and in situ tests, to determine permeability and assess the quantum of leakage which may take place through these zones on impoundment of the reservoir. Moreover, permeability of rocks/overburden in the reservoir area is determined from water table fluctuations and pumping tests in wells. For determining in situ permeability in overburden and rock, reference should be made to IS 5529 (Part I): 1985 and IS: 5529 (Part II): 1985 respectively. The information about permeability would enable the designers to estimate the treatment cost for controlling leakage/seepage from the reservoir and to decide whether it would be desirable to change the location of height of the dam to avoid these zones.

Major unstable zones along the reservoir identified during the reconnaissance stage and which are of consequence to the storage scheme should be investigated in detail at this stage by means of surface and sub-surface exploration.

The areas should be geologically mapped in detail on a scale of 1: 2000. The suspect planes/zones of failure should be identified and explored by means of drifts, trenches and pits. Disturbed and undisturbed samples of the plastic material should be tested for cohesion (c) and angle of internal friction (ϕ) as well as for other relevant properties. The stability of slopes should also be evaluated considering the reservoirs operational conditions. These studies should provide the designers with an idea of the magnitude of the problems that may be encountered, so that they may be able to take remedial measures to stabilize zones or to abandon the site altogether, if the situation demands.

The areas having potential economic mineral wealth and which are likely to be adversely affected by the impoundment of the reservoir should be explored by means of surface and sub-surface investigation to establish their importance both in terms of their value as well as strategic importance. This information would be necessary for arriving at a decision regarding the submergence, or otherwise, of the mineral deposit. The nature and amount of the existing seepage, if any, in the existing mines and quarries in the adjacent areas of the reservoir should be recorded and monitored regularly. This data is necessary, to ascertain whether or not there has been any change in the quantum of seepage in the mines and quarries due to the impoundment of water in the reservoir, directly or indirectly.

Large scale geological mapping and terrace matching across the faults with seismically active status, delineated during the reconnaissance stage, should be carried out on a scale of 1 : 2000 and the trend, and behaviour of the fault plane should be investigated in detail by means of surface studies and sub-surface exploration by pits, trenches and drifts etc. A network of geodetic survey points should be established on either side of the suspected faults to study micro-movements along these suspected faults, if any, both prior to and after impoundment of the reservoir. Micro earthquake studies should be carried out using portable 3-station or 4-station networks in areas with proven seismically active fault features.

On the basis of the studies carried out during the preliminary stage it should be possible to estimate the quantum of exploration which may be required during the detailed stage of investigation including the total number of holes required to be drilled and the total number and depth of pits, trenches and drifts as also the extent of geophysical survey which may be necessary.

Detailed surface and sub-surface investigation of all features connected with the reservoir should be carried out to provide information on leakage of water through the periphery and/or basin of the reservoir area.

Based on these investigations and analysis of data it should be possible to decide as to whether the reservoir area in question would hold water without undue leakage. If, not, the dam site may have to be abandoned in favour of suitable alternative site.

The zones, which on preliminary investigation are found to be potential zones of leakage/seepage from the reservoir, and which due to other considerations cannot be avoided are geologically mapped on a scale of 1 : 2 000 and investigated in detail at this stage by means of a close spaced sub-surface exploration programme. The purpose of this stage of investigation is to provide the designers sufficient data to enable them to plan the programme of remedial treatment. The sub-surface explorations are carried out by means of pits and trenches, if the depth to be explored is shallow, say up to 5 meters, and by drill holes and drifts, if the depth to be explored is greater than 5 meters.

The unstable zones around the reservoir rim, specially those close to the dam sites in tight gorges, should be explored in detail by means of drifts, pits and trenches so that the likely planes of failures are located with precision. The physical properties including angle of internal friction and cohesion of representative samples of the material along which movement is anticipated should be determined. The above information would enable the designers to work out details for preventive measures, for example, it may be possible to unload the top of the slide area or to load the toe of the slide with well drained material, within economic limits.

Sub-surface explorations by drill holes, drifts, pits and trenches should be carried out at possible locations of check dams and at the locations of other preventive structures proposed to restrict the flow of silt into the reservoir. These studies would enable the designers to assess the feasibility of such proposals.

Detailed plans, regarding the economic mineral deposits within the zones of influence of the reservoir should be finalized during this stage by the concerned agencies. The seepage investigations in the quarries and mines within the zone of influence of the reservoir should be continued

4.5.7 Fixing the capacity of the reservoirs

Once it is decided to build a reservoir on a river by constructing a dam across it, it is necessary to arrive at a suitable design capacity of the reservoir. As has been discussed in section 4.5.1, the reservoir storage generally consists of there main parts which may be broadly classified as:

1. Inactive storage including dead storage
2. Active or conservation storage, and
3. Flood and surcharge storage.

In general, these storage capacities have to be designed based on certain specified considerations, which have been discussed separately in the following Bureau of Indian Standard codes:

IS: 5477 Fixing the capacities of reservoirs- Methods
(Part 1): 1999 General requirements
(Part 2): 1994 Dead storage
(Part 3): 1969 Line storage
(Part 4): 1911 Flood storage

The data and information required for fixing the various components of the design capacity of a reservoir are as follows:

a) Precipitation, run-off and silt records available in the region;
b) Erodibility of catchment upstream of reservoir for estimating sediment yield;
c) Area capacity curves at the proposed location;
d) Trap efficiency;
e) Losses in the reservoir;
f) Water demand from the reservoir for different uses;

g) Committed and future upstream uses;
h) Criteria for assessing the success of the project;
i) Density current aspects and location of outlets;
j) Data required for economic analysis; and
k) Data on engineering and geological aspects.

These aspects are explained in detail in the following sections.

4.5.7.1 *Precipitation, Run-Off and Silt Record*

The network of precipitation and discharge measuring stations in the catchment upstream and near the project needs to be considered to assess the capacity of the same to adequately sample both spatially and temporally the precipitation and the stream flows.

The measurement procedures and gap filling procedures in respect of missing data as also any rating tables or curves need to be critically examined so that they are according to guidelines of World Meteorological Organization (WMO). Long-term data has to be checked for internal consistency between rainfall and discharges, as also between data sets by double mass analysis to highlight any changes in the test data for detection of any long term trends as also for stationarity. It is only after such testing that the data should be used for generating the long term inflows of water (volumes in 10 days, 15 days, monthly or yearly inflow series) into the reservoir.

Sufficiently long term precipitation and run-off records are required for preparing the water inflow series. For working out the catchment average sediment yield, long-term data of silt measurement records from existing reservoirs are essential. These are pre-requisites for fixing the storage capacity of reservoirs.

If long term run-off records are not available, concurrent rainfall and run-off data may be used to convert long term rainfall data (which is generally available in many cases) into long-term run-off series adopting appropriate statistical/conceptual models. In some cases regression analysis may also be resorted to for data extension.

4.5.7.2 *Estimation of average Sediment Yield from the catchment area above the reservoir*

It is usually attempted using river sediment observation data or more commonly from the experience of sedimentation of existing reservoirs with similar characteristics. Where observations of stage/flow data is available for only short periods, these have to be suitably extended with the help of longer data on rainfall to estimate as far as possible sampling errors due to scanty records. Sediment discharge rating curve may also be prepared from hydraulic considerations using any of the standard sediment load formulae, such as, Modified Einstein's procedure, Young's stream power, etc. It is also necessary to account for the bed load which may not have been measured. Bed load measurement is preferable and when it is not possible, it is often estimated as a percentage generally ranging from 5 to 20 percent of the suspended sediment load. However, actual measurement of bed load needs to be undertaken particularly in cases where high bed loads are anticipated. To assess the volume of sediment that would deposit in the reservoir, it is further necessary to make estimates of average trap efficiency of the reservoir and the likely unit weight of sediment deposits, along with time average over the period selected. The trap efficiency would depend on the capacity inflow ratio but would also vary with the locations of controlling outlets and reservoir operating procedures. Computations of reservoir trap efficiency may be made using the trap efficiency curves such as those developed by Brune and by Churchill (see IS: 12182-1987).

4.5.7.3 *Elevation Area Capacity Curves*

Topographic survey of the reservoir area should form the basis for obtaining these curves, which are respectively the plots of elevation of the reservoir versus surface area and elevation of the reservoir versus volume. For preliminary studies, in case suitable topographic map with contours, say at intervals less than 2.5 m is not available, stream profile and valley cross sections taken at suitable intervals may form the basis for computing the volume. Aerial survey may also be adopted when facilities are available.

4.5.7.4 *Trap Efficiency*

Trap efficiency of reservoir, over a period, is the ratio of total deposited sediment to the total sediment inflow. Figures 1 and 2 given in Annex A of IS 12182 cover relationship between sedimentation index of the reservoir and percentage of incoming sediment and these curves may be used for calculation of trap efficiency.

4.5.7.5 *Losses in Reservoir*

Water losses mainly of evaporation and seepage occur under pre-project conditions and are reflected in the stream flow records used for estimating water yield. The construction of new reservoirs and canals is often accompanied by additional evaporation and infiltration. Estimation of these losses may be based on measurements at existing reservoirs and canals. The measured inflows and outflows and the rate of change of storage are balanced by computed total loss rate.

The depth of water evaporated per year from the reservoir surface may vary from about 400 mm in cool and humid climate to more than 2500 mm in hot and arid regions. Therefore, evaporation is an important consideration in many projects and deserves careful attention. Various methods like water budget method, energy budget method, etc may be applied for estimating the evaporation from reservoir. However, to be more accurate, evaporation from reservoir is estimated by using data from pan-evaporimeters or pans exposed to atmosphere with or without meshing in or near the reservoir site and suitably adjusted.

Seepage losses from reservoirs and irrigation canals may be significant if these facilities are located in an area underlain by permeable strata. Avoidance in full or in part of seepage losses may be very expensive and technical difficulties involved may render a project unfeasible. These are generally covered under the conveyance losses in canals projected on the demand side of simulation studies.

4.5.7.6 *Demand, Supply and Storage*

The demand should be compared with supplies available year by year. If the demand is limited and less than the available run-off, storage may be fixed to cater to that particular demand which is in excess of the run-off. The rough and ready method is the mass curve method for initial sizing.

Even while doing the above exercise, water use data are needed to assess the impact of human activities on the natural hydrological cycle. Sufficient water use information would assist in implementing water supply projects, namely, evaluating the effectiveness of options for demand management and in resolving problems inherent in competing uses of water, shortages caused by excessive withdrawal, etc. Water demands existing prior to construction of a water resource project should be considered in the design of project as failure to do so may result in losses apart from legal and social problems at the operation stage.

4.5.7.7 *Committed and future upstream uses*

The reservoir to be planned should serve not only the present day requirements but also the anticipated future needs. The social, economic and technological developments may bring in considerable difference in the future needs/growth rate as compared to the present day need/growth rate. Committed and upstream future uses should also be assessed in the same perspective.

4.5.7.8 *Criteria for assessing the success of the project*

Water Resources Projects are to be designed for achieving specified success. Irrigation projects are to be successful for 75 percent period of simulation. Likewise power projects and water supply projects are to be successful for 90 percent and nearly 100 percent period of simulation respectively.

4.5.7.9 *Density Current aspects and location of outlets*

Density current is defined as the gravitational flow of one fluid under another having slightly different density. The water stored in reservoir is generally free from silt but the inflow during floods is generally muddy. There are, thus two layers having different densities resulting in the formation of density currents. The density currents separate the water from the clearer water and make the turbid water flow along the river bottom. The reservoir silting rate can be reduced by venting the density currents by properly locating and operating the outlets and sluice ways.

4.5.7.10 *Data Required for Economic Analysis*

Economic Analysis is carried out to indicate the economic desirability of the project. Benefit cost ratio, Net benefit, Internal Rate of Return are the parameters in this direction. It is desirable to have the benefit cost ratio in the case of irrigation projects and flood mitigation projects to be above 1.5 and 1.1 respectively. Benefit functions for reservoir and water utilisation for irrigation, power, water supply etc., are also to be determined judiciously. Cost benefit functions are obtained as continuous functions using variable cost/benefit against reservoir storage/net utilisation of water and from benefit functions the benefit from unit utilisation of water can be determined. The spillway capacity has to be adequate to pass the inflow design flood using moderation possible with surcharge storage or any other unobstructed capacity in the reservoir without endangering the structural safety as provided elsewhere in the standard. In the event of the inflow design flood passing the reservoir, the design needs to ensure that dam break situation does not develop or induce incremental damage downstream.

4.5.7.11 *Data on Engineering and Geological Aspects*

Under engineering and geological aspects the following items of work shall invariably be carried out:

a) Engineering

1) Preliminary surveys to assess the catchment and reservoir,
2) Control surveys like topographical surveys,
3) Location of nearest Railway lines/Roads and possible access, and
4) Detailed survey for making area capacity curves for use in reservoir flood routing.

b) Geology

1) General formations and foundation suitability;
2) Factors relating to reservoir particularly with reference to water tightness;
3) Contributory springs;
4) Deleterious mineral and salt deposits; and
5) Location of quarry sites, etc.

Important aspects of the methods are briefly described in the subsequent sections.

4.5.8 Fixing of Inactive Storage including Dead Storage

Inactive storage including dead storage pertains to storage at the lowest level up to which the reservoir can be depleted. This part of the storage is set apart at the design stage for anticipated filling, partly or fully, by sediment accumulations during the economic life of the reservoir and with sluices/outlets so located that it is not susceptible to full depletion. In case power facility is provided, it is also the storage below the minimum draw down level (MDDL).

Sill level of lowest outlets for any reservoir is fixed from command considerations in case of irrigation purposes and minimum draw down level on considerations of efficient turbine operation in the case of power generation purpose. The lowest sill level should be kept above the new zero elevation expected after the feasible service period according to IS 12182 which is generally taken as 100 years for irrigation projects and 70 years for power projects supplying power to a grid.

By providing extra storage volume in the reservoir for sediment accumulation, in addition to live storage, it is ensured that the live storage although it contains sediment, will function at full efficiency for an assigned number of years. The distribution pattern of sediments in the entire depth of a reservoir depends upon many factors, such as slope of the valley, length of reservoir, constriction in the reservoir, particle size of the suspended sediment and capacity inflow ratio, but the reservoir operation has an important control over the factors. However, the knowledge of the pattern is essential, especially, in developing areas, in order to have an idea about the formation of delta and recreational spots.

The dead storage of a reservoir depends upon the sediment yield of the catchment. The measurement of sediment yield is done as follows:

4.5.8.1 *Measurement of sediment yields*

The sediment yield in a reservoir may be estimated by any one of the following two methods:
a) Sedimentation surveys of reservoirs with similar catchment characteristics, or
b) Sediment load measurements of the stream.

4.5.8.2 *Reservoir Sedimentation Survey*

The sediment yield from the catchment is determined by measuring the accumulated sediment in a reservoir for a known period, by means of echo sounders and other electronic devices since the normal sounding operations give erroneous results in large depths. The volume of sediment accumulated in a reservoir is computed as the difference between the present reservoir capacity and the original capacity after the completion of the dam. The unit weight of deposit is determined in the laboratory front the representative undisturbed samples or by field determination using a calibrated density probe developed for this purpose. The total sediment volume is then converted to dry- weight of sediment on the basis of average unit weight of deposits. The total sediment yield for the period of record covered by the survey will then be equal to the total weight of the sediment deposited in the reservoir plus that which has passed out of the reservoir based on the trap efficiency. In this way, reliable records may be readily and economically obtained on long-term basis.

The density of deposited sediment varies with the composition of the deposits, location of the deposit within the reservoir, the flocculation characteristics of clay content and water, the age of deposit, etc. For coarse material (0.0625 mm and above) variation of density with location and age may be unimportant.

Normally a time and space average density of deposited materials applicable for the period under study is required for finding the overall volume of deposits. For this purpose the trapped sediment for the period under study would have to be classified in different fractions. Most of the sediment escape front getting deposited into the reservoir should be front the silt and clay fractions. In some special cases local estimates of densities at points in the reservoir may be required instead of average density over the whole reservoir.

The trap efficiency mainly depends upon the capacity-in-flow ratio but may vary with location of outlets and reservoir operating procedure. Computation of reservoir trap efficiency may be made using trap efficiency curves, such as those developed by Brune and by Churchill (see IS: 12182-1987).

4.5.8.3 *Sediment Load Measurements*

Periodic samples front the stream should be taken at various discharges along with the stream gauging observations and the suspended sediment concentration should be measured as detailed in IS 4890: 1968. A sediment rating curve which is a plot of sediment concentration against the discharge is then prepared and is used in conjunction with stage duration curve (or flow duration) based on uniformly spaced daily or shorter time units data in case of smaller river basins to assess sediment load. For convenience, the correlation between sediment concentration against discharge, may be altered to the relation of sediment load against run-off for calculating sediment yield. Where observed stage/flow data is available for only shorter periods, these have to be suitably extended with the help of longer data on rainfall. The sediment discharge rating curves may also be prepared from hydraulic considerations using sediment load formula, that is, modified Einstein's procedure.

The bed load measurement is preferable. How- ever, where it is not possible, it may be estimated using analytical methods based on sampled data or as a percentage of suspended load (generally ranging from 10 to 20 percent). This should be added to the suspended load to get the total sediment load.

4.5.9 Fixing of Live Storage

The storage of reservoir includes the Active Storage (or Conservation Storage) and the Buffer Storage.

Active or conservation storage assures the supply of water from the reservoir to meet the actual demand of the project whether it is for power, irrigation, or any other demand water supply.

The active or conservation storage in a project should be sufficient to ensure success in demand satisfaction, say 75 percent of the simulation period for irrigation projects, whereas for power and water supply projects success rates should be 90 percent and 100 percent respectively. These percentages may be relaxed in case of projects in drought prone areas. The simulation period is the feasible service period, but in no case be less than 40 years. Storage is also provided to satisfy demands for maintaining draft for navigation and also maintaining water quality for recreation purpose as envisaged in design.

Live storage capacity of a reservoir is provided to impound excess waters during periods of high flow, for use during periods of low flow. It helps the usage of water at uniform or nearly uniform rate which is greater than the minimum flow live storage has to guarantee a certain quantity of water usually called safe (or firm) yield with a predetermined reliability. Though sediment is distributed to some extent in the space for live storage, the capacity of live storage is generally taken as the useful storage between the full reservoir level and the minimum draw-down level in the case of power projects and dead storage in the case of irrigation projects.

The design of the line storage include certain factors, of which the most important in the availability of flow, since, without an adequate flow, it is not possible to cope up with the demand at all periods and seasons throughout the year. When adequate flow is available, there may still be certain problems like the possible maximum reservoir capacity from physical considerations may be limited and then this becomes the governing criteria. Even if an adequate reservoir capacity may be possible to be built, the governing factor may have to be based on the demand.

For fixing the live storage capacity, the following data should be made use of:

a) Stream flow data for a sufficiently long period at the site;
b) Evaporation losses from the water-spread area of the reservoir and seepage losses and also recharge into reservoir when the reservoir is depleting;
c) The contemplated irrigation, power or water supply demand;
d) The storage capacity curve at the site.

Stream flow records are required at proposed reservoir site. In the absence of such records the records from a station located upstream or downstream of the site on the stream or .on a nearby stream should be adjusted to the reservoir site. The run off records are often too short to include a critical drought period. In such a case the records should be extended by comparison with longer stream flow records in the vicinity or by the use of rainfall run off relationship.

The total evaporation losses during a period are generally worked out roughly as the reduction in the depth of storage multiplied by the mean water-spread area between the full reservoir level and the minimum draw-down level. For accurate estimation, monthly working tables should be prepared and the mean exposed area during the month is found out and the losses should be then worked out on the basis of this mean exposed area, and the evaporation data from pan evaporimeter at the reservoir site. The details are expected to be covered in the draft ‘Indian Standard criteria for determination of seepage and evaporation losses including the code for minimizing them. In the absence of’ actual data these may be estimated from the records of an existing reservoir with similar characteristics, like elevation, size, etc, in the neighbourhood.

Of the various methods available for fixing the live storage capacity, the Working Table method may be used which is prepared on the basis of preceding long term data on discharge observation at the site of the proposed reservoir, inclusive of at least one drought period. A typical format for carrying out the working table computation, is given in the following table:

Working Table

Months	Beginning of period			Inflow in M-Ha-m	Total M-Ha-m	Demand during period				End of period			Spilled Water	Tail Race Level	Effective Head in m	Discharge in m^3/s	Re-generation	Power Potential P=7.4QH
	Reservoir level	Capacity in M-Ha-m	Water Spread area			For irrigation	For Power	Evaporation	Total	Reser-voir Level	Capacity M-Ha-m	Water Spread area						
(1)	(2)	(3)	(4)	(5)	(6) = (3) +(5)	(7)	(8)	(9)	(10) = (7)+(8)+(9)	(11)	(12) = (6)-(10)	(13)	(14)	(15)	(16)	(17)	(18)	(19)

The working table calculations may be represented graphically by plotting the cumulative net reservoir inflow exclusive of upstream abstraction as ordinate against time as abscissa. This procedure is commonly called the Mass Curve Technique, where the ordinate may be denoted by depth in centimeters or in hectare meters or in any other unit of volume. Discharge, with units of 10 days or a month may be used culmination in the mass curve. A segment of the mass curve is shown in Figure 3.

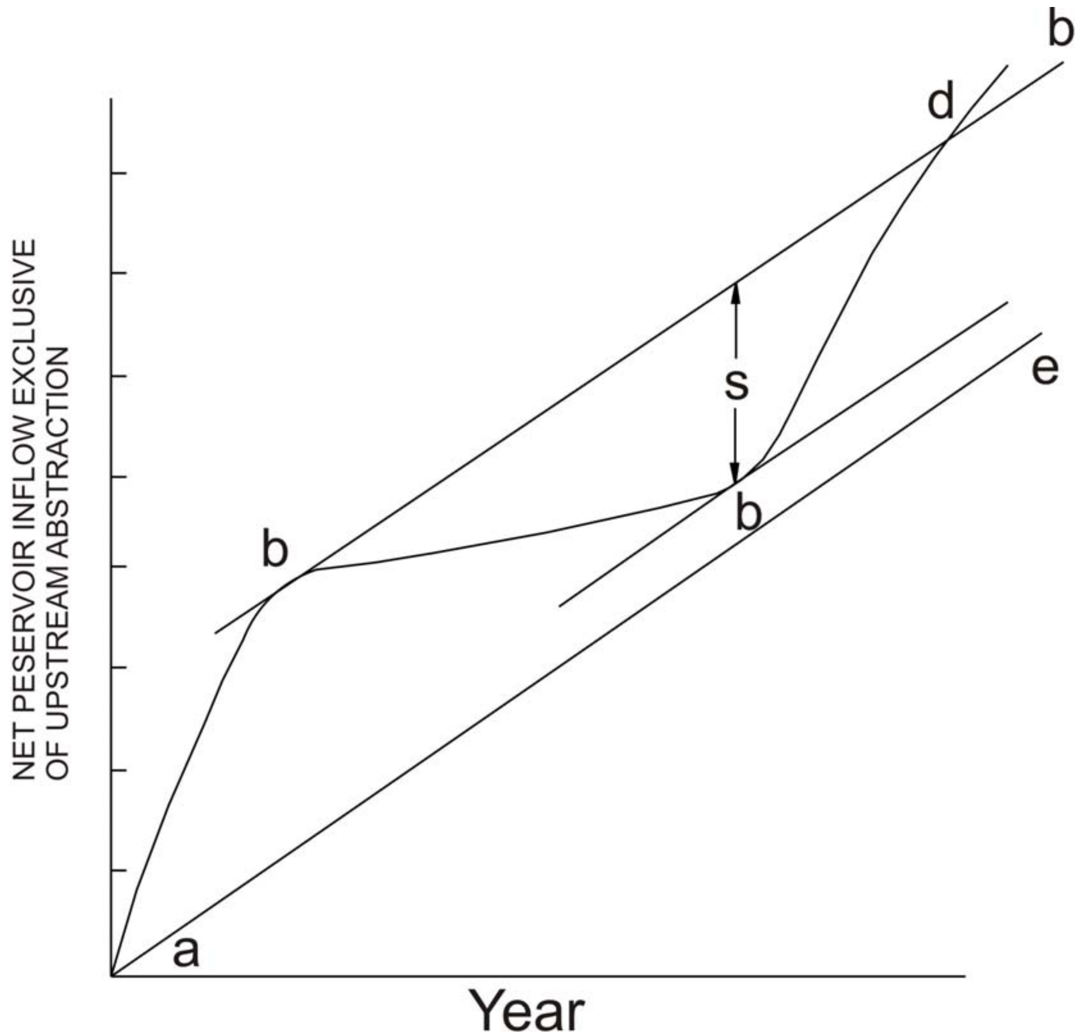


FIGURE3.SEGMENT OF NET INFLOW MASS CURVE

The difference in the ordinate at the end of a segment of the mass curve gives the inflow volume during that time interval. Lines parallel to the lines of uniform rate of demand are drawn at the points ***b*** and ***c*** of the mass curve. At ***d***, the following inferences can be made:

- The inflow rate between ***a*** to ***b*** is more than the demand rate and the reservoir is full.
- Reservoir is just full as the inflow rate is equal to the demand rate.
- Reservoir storage is being drawn down between ***b*** and ***c*** since the demand rate exceeds the inflow rate.
- Draw down, S, is maximum at ***c*** due to demand rate being equal to inflow rate.
- Reservoir is filling or in other words draw down is decreasing from ***c*** to ***d*** as the inflow rate is more than the demand rate.
- Reservoir is full at ***d*** and from ***d*** to *b* again the reservoir is over flowing because the inflow rate exceeds the demand rate. The greatest vertical distance, S at ***c*** is the storage required to make up the proposed demand.

The withdrawals from the reservoir to meet the irrigation demand are generally variable and in such cases the demand line becomes a curve instead of a straight line. The demand mass curve should be super-imposed on the inflow mass curve on the same time scale. When the inflow and demand mass curves intersect, the reservoir may be assumed to be full. For emptying conditions of the reservoir the demand curve would be above the inflow curve and the maximum ordinate between the two would indicate the live storage capacity required.

4.5.9.1 *Fixation of Live Storage Capacity for a Given Demand*

Lines parallel to the demand lines are drawn at all the peak points of the mass inflow curve exclusive of upstream abstraction obtained from a long run off record on 10-day (or monthly) basis as shown in Figure 4. When the demand line cuts the mass curve the reservoir may be assumed to be full. The maximum ordinate between the demand line and the mass curve will give the live storage to meet the required demand. The vertical distance between the successive lines parallel to the demand line represents the surplus water from the reservoir.

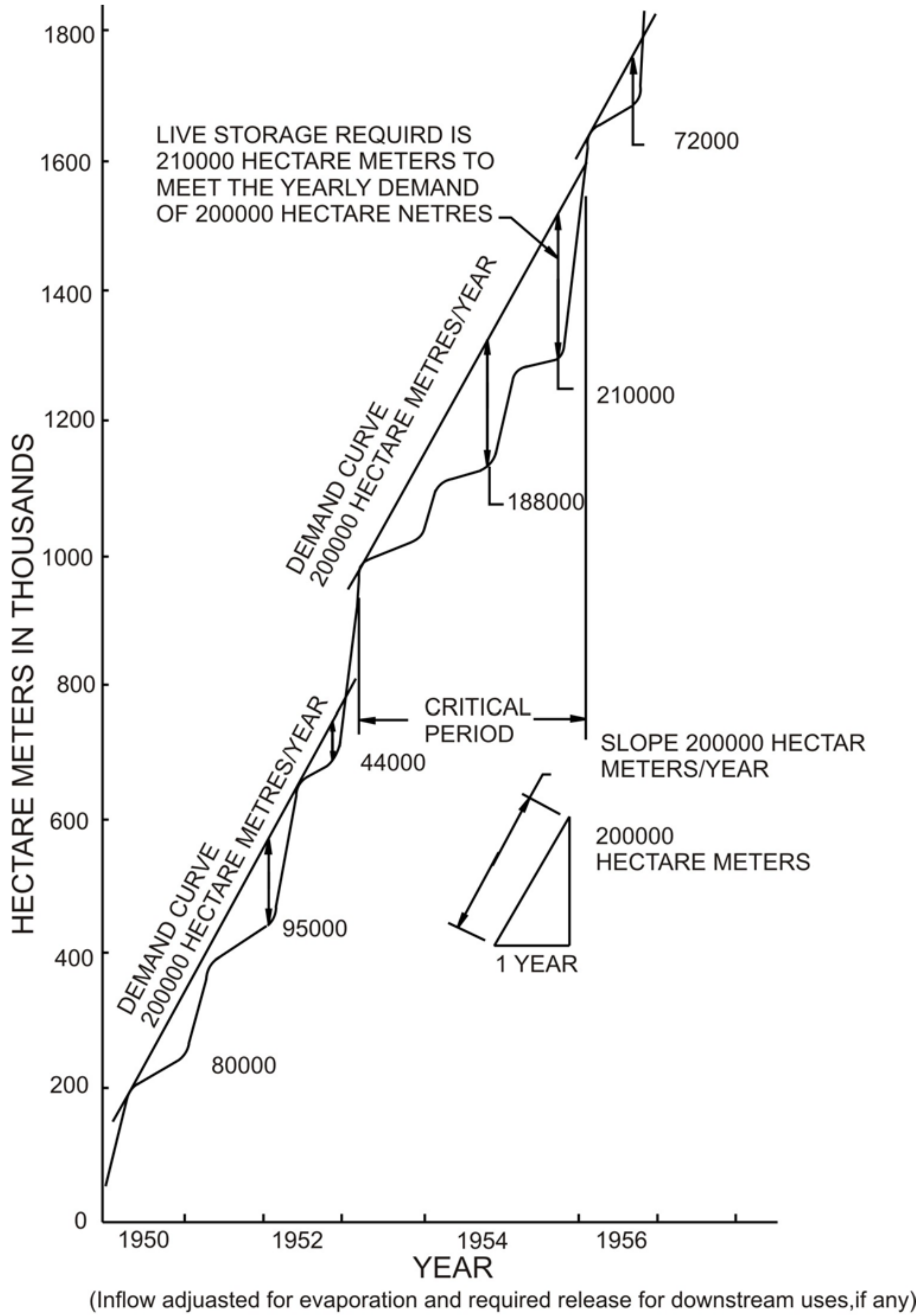


FIGURE 4. A TYPICAL MASS CURVE

4.5.9.2 *Estimation of a Demand from a Given Live Storage Capacity*

The net inflow mass curve is plotted from the available records. The demand lines arc, drawn at peak points of the mass curve in such a way that the maximum ordinate between the demand line and the mass curve is equal to the specified live storage. The demand lines shall intersect the mass curve when extended forward. The slop of the flattest line indicates the film demand that could he met by the given live storage rapacity.

Before fixing the reservoir capacity, it would be desirable to plot a curve between the net annual drafts and the required live storage capacities for these drafts. This curve will give an indication of the required live storage capacity. However, the economics of the capacity will have to be considered before deciding final capacity.

4.5.10 Fixing of flood and surcharge storage

In case of reservoirs having flood control as one of the purposes, a separate flood control storage is to be set apart above the storage meant for power, irrigation and water supply. Flood control storage is meant for storing flood waters above a particular return period temporarily and to attenuate discharges up to that flood magnitude to minimise effects on downstream areas from flooding. Flood and surcharge storage between the full reservoir level (FRL), and maximum water level (MWL) attainable even with full surplussing by the spillway takes care of high floods and moderates them.

4.5.10.1 *Flood Control Storage*

Storage space is provided in the reservoir for storing flood water temporarily in order to reduce peak discharge of a specified return period flood and to minimize flooding of downstream areas for all floods IS: 5477 (Part 1) : 1999 equal to or lower than the return period flood considered. In the case of reservoirs envisaging flood moderation as a purpose and having separate flood control storage, the flood storage is provided above the top of conservation pool.

4.5.10.2 *Surcharge Storage*

Surcharge storage is the storage between the full reservoir level (FRL) and the maximum water level (MWL) of a reservoir which may be attained with capacity exceeding the reservoir at FRL to start with. The spillway capacity has to be adequate to pass the inflow design flood making moderation possible with surcharge storage.

The methods that are generally used for estimate of the Design Flood for computing the Flood Storage are broadly classified as under:

1. Application of a suitable factor of safety to maximum observed flood or maximum historical flood.
2. Empirical flood formulae.
3. Envelope curves.
4. Frequency analysis.
5. Rating method of derivation of design flood from storm studies and application of the Unit Hydrograph principle.

The important methods amongst the above have been explained in module 2. Nevertheless, these methods are briefly reiterated below:

4.5.10.3 *Application of a Suitable Factor of Safety to Maximum Observed Flood or Maximum Historical Flood*

The design flood is obtained by applying a safety factor which depends upon the judgement of the designer to the observed or estimated maximum historical flood at the project site or nearby site on the same stream. This method is limited by the highly subjective selection of a safety factor and the length of available stream flow record which may give an inadequate sample of flood magnitudes likely to occur over a long period of time.

Empirical Flood Formulae : The empirical formulae commonly used in the country are the Dicken's formula, Ryve's formula and Inglis' formula in which the peak flow is given as a function of the catchment area and a coefficient. The values of the coefficient vary within rather wide, limits and have to be selected on the basis of judgement. They have limited regional application, should be used with caution and only, when a more accurate method cannot be applied for lack of data.

Envelope Curves: In the envelope curve method maximum flood is obtained from the envelope curve of all the observed maximum floods for a number of catchments in a homogeneous meteorological region plotted against drainage area. This method, although useful for generalizing the limits of floods actually experienced in the region under consideration, cannot be relied upon for estimating maximum probable floods for the determination of spillway capacity except as an aid to judgement.

Frequency Analysis: The frequency method involves the statistical analysis of observed data of a fairly long (at least 25 years) period. It is a purely statistical approach and when applied to derive design floods for long recurrence intervals, several times larger than the data, has many limitations. Hence this method has to be used with caution.

4.5.10.4 Rational Method of Derivation of Design Flood from Storm Studies and Application of Unit Hydrograph Principle

The steps involved, in brief, are:

a. Analysis of rainfall and run-off data for derivation of loss rates under critical conditions;
b. Derivation of unit hydrograph by analysis (or by synthesis, in cases where data are not available);
c. Derivation of the design storm; and
d. Derivation of design flood from the design storm by the application of the rainfall excess increments to the unit hydrograph.

The Maximum Water Level of a reservoir is obtained by routing the design flood through the reservoir and the spillway. This process of computing the reservoir storages, storage volumes and outflow rates corresponding to a particular hydrograph of inflow is commonly referred to as flood routing. The routing is carried out with the help of the following data,

1. Initial reservoir stage
2. The design flood hydrograph
3. Rate of outflow including the flow over the crest, through sluices or outlets and through power units, and
4. Incremental storage capacity of the reservoir.

Typical values of the last two types of data, is shown in Figure 5.

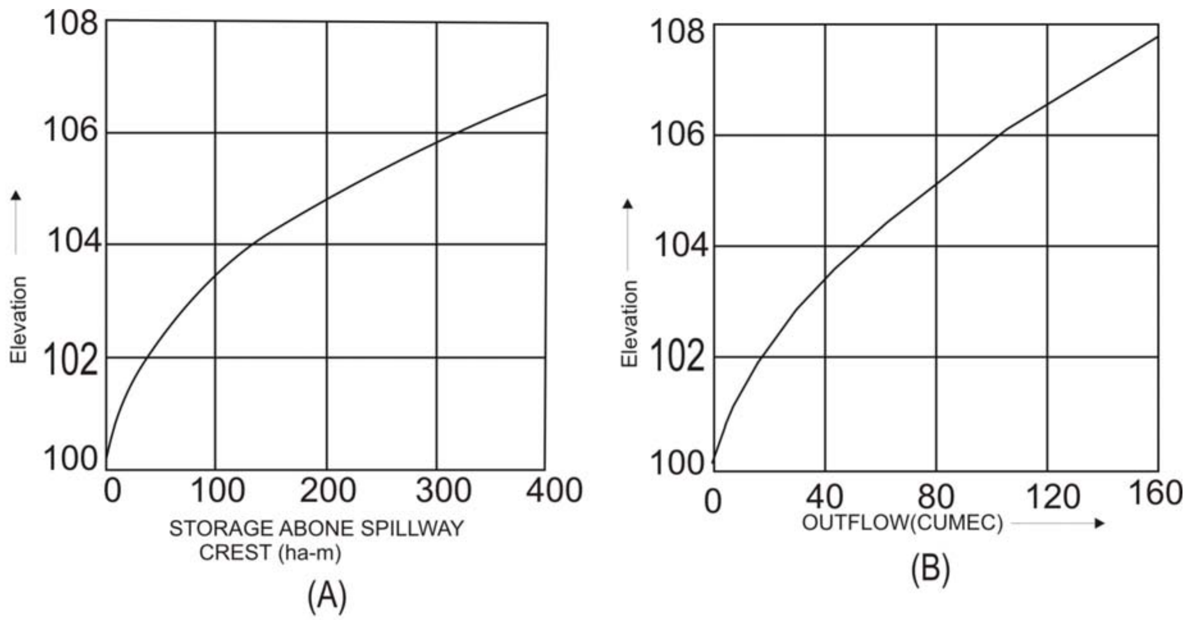


FIGURE 5. TYPICAL CURVES FOR (A) STORAGE Vs ELEVATION
(B) OUTFLOW Vs ELEVATION

The routing of flood through the reservoir and the spillway is done by solving the continuity of flow within reservoir, which may simply be stated as:
Inflow to reservoir - Outflow to reservoir = Rise is water surface of the reservoir, that is an increase in the storage of the reservoir. That is,

$$(I - O) * \Delta t = \Delta S \qquad (1)$$

Where, I is the inflow discharge (m^3/s), O is the Outflow discharge (m^3/s), Δs is the in storage volume (m^3/s) in time interval Δt (h).
If the inflow hydrograph is known, then we may read out the inflow ordinates at every time interval (t. Suppose, the following values are known:

I1 = Inflow (m3/s) at the beginning of a time interval
I2 = Inflow (m3/s) at the end of the time interval

O1= Outflow (m3/s) at the beginning of the time interval

S1= Total storage volume of the reservoir (m3/s) at the beginning of the time interval

And the unknown values are

S2= Total storage volume of the reservoir (m3/s) at the end of the interval

O2= Outflow (m3/s) at the end of the time interval

Then, we rewrite the continuity equation as,

$$\Delta t\left(\frac{I_1 + I_2}{2}\right) - \Delta t\left(\frac{O_1 + O_2}{2}\right) = S_2 - S_1 = \Delta S \qquad (2)$$

Taking the known values to the left side of the equation, one obtains,

$$\left(\frac{I_1 + I_2}{2}\right) + \left(\frac{S_1}{\Delta t} - \frac{O_1}{2}\right) = \left(\frac{S_2}{\Delta t} + \frac{O_2}{2}\right) \qquad (3)$$

Now, for solving the above equation, a table of $\left(\frac{S}{\Delta t} - \frac{O}{2}\right)$ and $\left(\frac{S}{\Delta t} + \frac{O}{2}\right)$ versus Reservoir Level, are prepared with the help of the Spillway Capacity (Outflow) curve and the Reservoir Storage Capacity Curve, which were shown in Figure 5. From the table, two graphs are prepared which typically looks as shown in Figure 6.

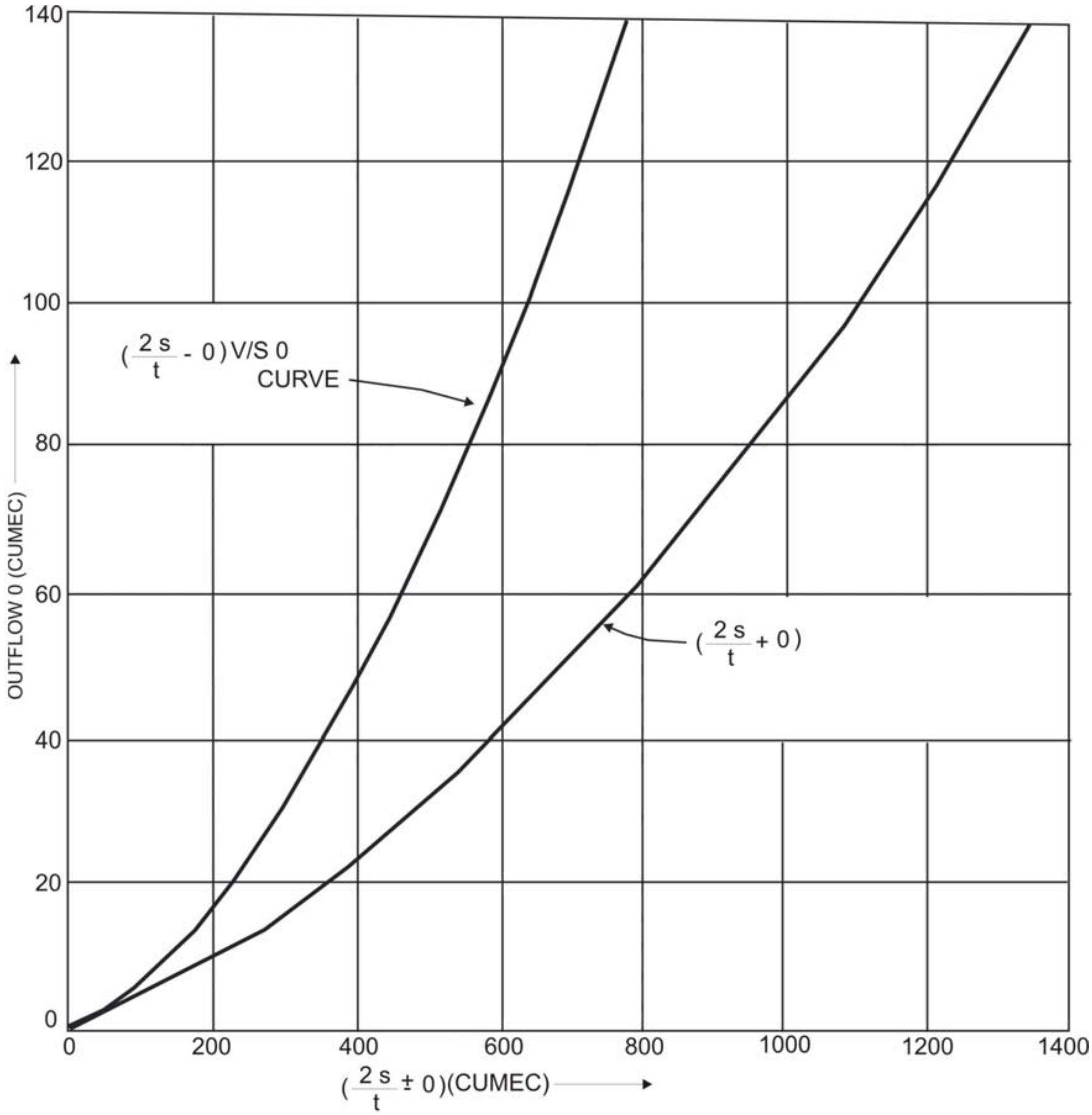


FIGURE 6. TYPICAL COMPUTATION CURVES

At the start of the inflow or the beginning of the first interval of time, to account for the worst condition, the water surface in the reservoir is normally taken to be at the maximum conservation level or the full reservoir level and hence, both the storage S_1 and outflow O_1 at the beginning are equal to zero. Thus, one obtains from equation (3), the following at the beginning of the flood routing procedure:

$$\left(\frac{I_1+I_2}{2}\right)=\frac{S_2}{\Delta t}+\frac{O_2}{2} \tag{4}$$

where, the suffix 1 stands for the time at the beginning of routing and 2 for the time at the end of the routing after a time interval Δt.

For the first interval of time, the inflow rates I_1 and I_2 at the beginning and end of the interval are known (in fact, they are known at all times), and introducing these values in equation (4), $\left(\frac{S_2}{\Delta t}+\frac{O_2}{2}\right)$ becomes known. From Figure 5, corresponding to this value of $\left(\frac{S_2}{\Delta t}+O_2\right)$, one obtain the value of outflow O_2 at the end of the time interval. Corresponding to this value of O_2, one may also find $\left(\frac{S_2}{\Delta t}-O_2\right)$ from Figure 6, which may be used for the second time interval from the following expression:

$$\left(\frac{I_2+I_3}{2}\right)+\left(\frac{S_2}{\Delta t}-\frac{O_2}{2}\right)=\left(\frac{S_3}{\Delta t}+\frac{O_3}{2}\right) \tag{5}$$

In the equation (5), I_2 and I_3 are the inflow rates at the beginning and end of the second interval of time which are known from the given inflow hydrograph. Knowing $\left(\frac{S_2}{\Delta t}-O_2\right)$ from the calculation of the previous step, one may calculate $\left(\frac{S_3}{\Delta t}+\frac{O_3}{2}\right)$ from equation (5) and obtain O_3 and $\left(\frac{S_3}{\Delta t}-O_3\right)$ both from Figure 6. This procedure goes on till the inflow hydrograph gets fully routed through the reservoir.

4.5.11 Reservoir losses and their minimization

Loss of reservoir water would mainly take place due to evaporation and a number of methods have been suggest for controlling such loss. The Bureau of Indian Standard code IS: 14654 - 1999 “Minimizing evaporation losses from reservoirs- guidelines” describes the cause of evaporation reduction methods in detail, some important aspects of which are described in the subsequent paragraphs. As such, percolation or seepage loss is small for most of the reservoirs and progressively gets lowered with the passage of time since the sediment getting deposited at the reservoir bottom helps to reduce percolation losses. Of course, in some hills and valleys forming the reservoir, there may be continuous seams of porous rock strata or limestone caverns which cause huge amount of water to get drained out of the reservoirs. The reservoir of the Kopili Hydroelectric Project in Assam-Meghalaya border had faced similar problems due to the presence of large caverns which had to be sealed later at quite large cost at a later stage.

A number of factors affect the evaporation from open water surface, of which, the major factors are water spread area and frequent change of speed and direction of wind over the water body. Other meteorological factors like.

a) Vapour pressure difference_ between water surface and the layer of air above;
b) Temperature of water and air;
c) Atmospheric pressure;
d) Radiation;
e) Heat storage in water body; and
f) Quality of water,

have direct influence on the rate of evaporation.

Since the meteorological factors affecting evaporation cannot be controlled under normal conditions, efforts are made for inhibition of evaporation by control of flow of wind over water surface or by protection of the water surface area by physical or chemical methods. The methods generally used are as follows:

a) Wind breakers,
b) Covering the water surface,
c) Reduction of exposed water surface,
d) Integrated operation of reservoirs, and
e) Treatment with chemical water evaporetardants (WERs).

4.5.11.1 *Wind Breakers*

Wind is one of the most important factors which affect rate of evaporation loss from water surface. The greater the movement of air over the water surface, greater is the evaporation loss. Planting of trees normal to windward direction is found to be an effective measure for checking of evaporation loss. Plants (trees, shrubs or grass) should be grown around the rim of tanks in a row or rows to act as wind breaker.
These wind breakers are found to influence the temperature, atmospheric humidity, soil moisture, evaporation and transpiration of the area protected.

Plants to act as wind breakers are usually arranged in rows, with tallest plants in the middle and the smallest along the end rows, so that more or less conical formation is formed.

4.5.11.2 *Covering the Water Surface*

Covering the surface of water bodies with fixed or floating covers considerably retards evaporation loss. These covers reflect energy inputs from atmosphere, as a result of which evaporation loss is reduced. The covers literally trap the air and prevent transfer of water vapour to outer atmosphere.

Fixed covers are suitable only for relatively small storages. For large storages, floating covers or mat or spheres may be useful and effective. However, for large water surfaces the cost of covering the surface with floats is prohibitive, Further in case of reservoirs with flood outlets, there is also the danger of floats being lost over spillway or through outlets. The floating covers are thus of limited utility to larger water bodies.

4.5.11.3 *Reduction of Exposed Water Surface*

In this method shallow portions of the reservoirs are isolated or curtailed by construction of dykes or bunds at suitable locations. Water accumulated during the monsoon season in such shallow portions IS diverted or pumped to appropriate deeper pockets in summer months, so that the shallow water surface area exposed to evaporation is effectively reduced.

4.5.12 Control of sedimentation in reservoirs

Sedimentation of a reservoir is a natural phenomenon and is a matter of vital concern for storage projects in meeting various demands, like irrigation, hydroelectric power, flood control, etc. Since it affects the useful capacity of the reservoir based on which projects are expected to be productive for a design period. Further, the deposited sediment adds to the forces on structures in dams, spillways, etc.

The rate of sedimentation will depend largely on the annual sediment load carried by the stream and the extent to which the same will be retained in the reservoir. This, in turn, depends upon a number of factors such as the area and nature of the catchment, level use pattern (cultivation practices, grazing, logging, construction activities and conservation practices), rainfall pattern, storage capacity, period of storage in relation to the sediment load of the stream, particle size distribution in the suspended sediment, channel hydraulics, location and size of sluices, outlet works, configuration of the reservoir, and the method and purpose of releases through the dam. Therefore, attention is required to each one of these factors for the efficient control of sedimentation of reservoirs with a view to enhancing their useful life and some of these methods are discussed in the Bureau of Indian Standard code IS: 6518-1992 “Code of practice for control of sediment in reservoirs”. In this section, these factors are briefly discussed.

There are different techniques of controlling sedimentation in reservoirs which may broadly be classified as follows:

- Adequate design of reservoir
- Control of sediment inflow
- Control of sediment deposition
- Removal of deposited sediment.

Each of these methods is briefly described as follows:

4.5.12.1 *Design of reservoirs*

The capacity of reservoirs is governed by a number of factors which are covered in IS: 5477 (Parts 1 to 4). From the point of view of sediment deposition, the following points may be given due consideration:

a) The sediment yield which depends on the topographical, geological and geo-morphological set up,meteorological factors, land use/land cover, intercepting tanks, etc;
b) Sediment delivery characteristics of the channel system;
c) The efficiency of the reservoir as sediment trap;
d) The ratio of capacity of reservoir to the inflow;
e) Configuration of reservoir;
f) Method of operation of reservoir;
g) Provisions for silt exclusion.

The rate of sediment delivery increases with the volume of discharge. The percentage of sediment trapped by a reservoir with a given drainage area increases with the capacity. In some cases an increased capacity will however, result in greater loss of water due to evaporation. However, with the progress of sedimentation, there is decrease of storage capacity which in turn lowers the trap efficiency of the reservoir.

The capacity of the reservoir and the size and characteristics of the reservoir and its drainage area are the most important factors governing the annual rate of accumulation of sediment. Periodical reservoir sedimentation surveys provide guidance on the rate of sedimentation. In the absence of observed data for the reservoir concerned, data from other reservoirs of similar capacity and catchment characteristics may be adopted.

Silting takes place not only in the dead storage but also in the live storage space in the reservoir. The practice for design of reservoir is to use the observed suspended sediment data available from key hydroIogica1 networks and also the data available from hydrographic surveys of other reservoirs in the same region. This data be used to simulate sedimentation status over a period of reservoir life as mentioned in IS 12182: 1987.

4.5.12.2 *Control of sediment inflow*

There are many methods for controlling sediment inflows and they can be divided as under:

a) Watershed management/soil conservation measures to check production and transport of sediment in the catchment area.

b) Preventive measures to check inflow of sediment into the reservoir.

The soil conservation measures are further sub-divided as:

a) Engineering,

b) Agronomy, and
c) Forestry.

The engineering methods include:

a) Use of check dams formed by building small barriers or dykes across stream channels.
b) Contour bounding and trenching;
c) Gully plugging;
d) Bank protection.

The agronomic measures include establishment of vegetative screen, contour farming, strip cropping and crop rotation.

Forestry measures include forest conservancy, control on grazing, lumbering, operations and forest fires along with management and protection of forest plantations.

Preventive measures to check inflow of sediment into the reservoir include construction of by-pass channels or conduits.

Check Dams

Check dams are helpful for the following reasons:

a) They help arrest degradation of stream bed thereby arresting the slope failure;
b) They reduce the veIocity of stream flow, thereby causing the deposition of the sediment load.

Check dams become necessary, where the channel gradients are steep and there is a heavy inflow of sediment from the watershed. They are constructed of local material like earth, rock, timber, etc. These are suitable for small catchment varying in size from 40 to 400 hectares. It is necessary to provide small check dams on the subsidary streams flowing into the main streams besides the check dams in the main stream. Proper consideration should be given to the number and location of check dams required. It is preferable to minimize the height of the check dams. If the stream ha, a very-steep slope, it is desirable to start with a smaller height for the check dams than may ultimately be necessary.

Check dams may generally cost more per unit of storage than the reservoirs they protect. Therefore, it may not always be possible to adopt them as a primary method of sediment control in new reservoirs. However, feasibility of providing check dams at a later date should not be overlooked while planning the protection of-a new reservoir.

Contour Bunding and Trenching

These are important methods of controlling soil erosion on the hills and sloping lands, where gradients of cultivated fields or terraces are flatter, say up to 10 percent. By these methods the hill side is split up into small compartments on which the rain is retained and surface run-off is modified with prevention of soil erosion. In addition to contour bunding, side trenching is also provided sometimes.

Gully Plugging

This is done by small rock fill dams. These dams will be effective in filling up the gullies with sediment coming from the upstream of the catchment and also prevent further widening of the gully.

4.5.12.3 *Control of sediment deposition*

The deposition of sediment in a reservoir may be controlled to a certain extent by designing and operating gates or other outlets in the dam in such a manner as to permit selective withdrawals of water having a higher than average sediment content. The suspended sediment content of the water in reservoirs is higher during and just after flood flow. Thus, more the water wasted at such times, the smaller will be the percentage of the total sediment load to settle into permanent deposits. There are generally two methods: (a) density currents, and (b) waste-water release, for controlling the deposition and both will necessarily result in loss of water.

Density Current

Water at various levels of a reservoir often contains radically different concentrations of suspended sediment particularly during and after flood flows and if all waste-water could be withdrawn at those levels where the concentration is highest, a significant amount of sediment might be removed from the reservoir. Because a submerged outlet draws water towards it from all directions, the vertical dimension of the opening should be small with respect to the thickness of the layer and the rate of withdrawal also should be low. With a view to passing the density current by sluices that might be existed, it is necessary to trace the movement of density currents and observation stations (consisting of permanently anchored rafts from which measurements could be made of temperature and conductivity gradient from the surface of the lake to the bottom, besides collecting water samples at various depths) at least one just above the dam and two or more additional stations in the upstream (one in the inlet and one in the middle) should be located.

Waste-Water Release

Controlling the sedimentation by controlling waste-water release is obviously possible only when water can be or should be wasted. This method is applicable only when a reservoir is of such size that a small part of large flood flows will fill it.

In the design of the dam, sediment may be passed through or over it as an effective method of silt control by placing a series of outlets at various elevations. The percentage of total sediment load that might be ejected from the reservoir through proper gate control will differ greatly with different locations. It is probable that as much as 20 percent of the sediment inflow could be passed through many reservoirs by venting through outlets designed and con- trolled.

Scouring Sluicing

This method is somewhat similar to both the control of waste-water release and the draining and flushing methods. The distinction amongst them care the following:

1) The waste-water release method ejects sediment laden flood flows through deep spillway gates or large under sluices at the rate of discharge that prevents sedimentation.

2) Drainage and flushing method involves the slow release of stored water from the reservoir through small gates or valves making use of normal or low flow to entrain and carry the sediment, and

3) Scouring sluicing depends for its efficiency on either the scouring action exerted by the sudden rush of impounded water under a high head through under sluices or on the scouring action of high flood discharge coming into the reservoir.

Scouring sluicing method can be used in the following:

a) Small power dams that depend to a great extent on pondage but not on storage;

b) Small irrigation reservoirs, where only a small fraction of the total annual flow can be stored;

c) Any reservoir in narrow channels, gorges, etc, where water wastage can be afforded; and

d) When the particular reservoir under treatment is a unit in an interconnected system so that the other reservoirs can supply the water needed.

4.5.12.4 *Removal of deposited sediment*

The most practical means of maintaining the storage capacity are those designed to prevent accumulation of permanent deposits as the removal operations are extremely expensive, unless the material removed is usable. Therefore, the redemption of lost storage by removal should be adopted as a last resort. The removal of sediment deposit implies in general, that the deposits are sufficiently compacted or consolidated to act as a solid and, therefore, are unable to flow along with the water. The removal of sediment deposits may be accomplished by a variety of mechanical and hydraulic or methods, such as excavation, dredging, siphoning, draining, flushing, flood sluicing, and sluicing aided by such measures as hydraulic or mechanical agitation or blasting of the sediment. The excavated sediments may be suitably disposed off so that, these do not find the way again in the reservoir.

Excavation

The method involves draining most of or all the water in the basin and removing the sediment by hand or power operated shovel, dragline scraper or other mechanical means. The excavation of silt and clay which constitute most of the material in larger reservoirs is more difficult than the excavation of sand and gravel. Fine-textured sediment cannot be excavated easily from larger reservoirs unless it is relatively fluid or relatively compact.

Dredging

This involves the removal of deposits from the bottom of a reservoir and their conveyance to some other point by mechanical or hydraulic means, while water storage is being maintained.

Dredging practices are grouped as:

a) Mechanical dredging by bucket, ladder, etc;

b) Suction dredging with floating pipeline and a pump usually mounted on a barrage; and

c) Siphon dredging with a floating pipe extending over the dam or connected to an opening in the dam and usually with a pump on a barrage.

Draining and Flushing

The method involves relatively slow release of all stored water in a reservoir through gates or valves located near bottom of the dam and the maintenance thereafter of open outlets for a shorter or longer period during which normal stream flow cuts into or directed against the sediment deposits. Therefore, this method may be adopted in flood control reservoirs.

Sluicing with Controlled Water

This method differs from the flood sluicing in that the controlled water supply permits choosing the time of sluicing more advantageously and that the water may be directed more effectively against the sediment deposits. While the flood sluicing depends either on the occurrence of flood or on being able to release rapidly all of a full or nearly full supply of water in the main reservoir is empty. The advantage of this method is that generally more sediment can be removed per unit of water used than in flood scouring or draining and flushing.

Sluicing with Hydraulics and Mechanical Agitation

Methods that stir up, break up or move deposits of a sediment into a stream current moving through a drained reservoir basin or into a full reservoir will tend to make the removal of sediment from the reservoir more complete. Wherever draining, flushing or sluicing appear to be warranted, the additional use of hydraulic means for stirring up the sediment deposits, or sloughing them off, into a stream flowing through the reservoir basin should be considered. It has, however, limited application.

4.5.13 Reservoir operation

The flow in the river changes seasonally and from year to year, due to temporal and spatial variation in precipitation. Thus, the water available abundantly during monsoon season becomes scarce during the non-monsoon season, when it is most needed. The traditional method followed commonly for meeting the needs of water during the scarce period is construction of storage reservoir on the river course. The excess water during the monsoon season is stored in such reservoirs for eventual use in lean period. Construction of storages will also help in control of flood, as well as generation of electricity power. To meet the objective set forth in planning a reservoir or a group of reservoirs and to achieve maximum benefits out of the storage created, it is imperative to evolve guidelines for operation of reservoirs. Without proper regulation schedules, the reservoir may not meet the full objective for which it was planned and may also pose danger to the structure itself.

Control of flood is better achieved if the reservoir level is kept low in the early stages of the monsoon season. However, at a later stage, if the anticipated inflows do not result the reservoir may not get filled up to FRL in the early stages of monsoon, to avoid the risk of reservoir remaining unfilled at later stage, there may be problem of accommodating high floods occurring at later stage. In some cases while planning reservoirs, social and other considerations occasionally result in adoption of a plan that may not be economically the best.

4.5.13.1 *Operation of Single Purpose Reservoirs*

The common principles of single purpose reservoir operation are given below:

a) Flood control- Operation of flood control reservoirs is primarily governed by the available flood storage capacity of damage centers to be protected, flood characteristics, ability and accuracy of flood/ storm forecast and size of the uncontrolled drainage area. A regulation plan to cover all the complicated situations may be difficult to evolve, but generally it should be possible according to one of the following principles:

1) *Effective use of available flood control storage:* Operation under this principle aims at reducing flood damages of the locations to be protected to the maximum extent possible, by effective use of flood event. Since the release under this plan would obviously be lower than those required for controlling the reservoir design flood, there is distinct possibility of having a portion of the flood control space occupied during the occurrence of a subsequent heavy flood. In order to reduce this element of risk, maintenance of an adequate network of flood forecasting stations both in the upstream and down stream areas would be absolutely necessary.

2) *Control of reservoir design flood:* According to this principle, releases from flood control reservoirs operated on this concept are made on the same hypothesis as adopted for controlling the reservoir design flood, that is the full storage capacity would be utilized only when the flood develops into the reservoir design flood. However, as the design flood is usually an extreme event, regulation of minor and major floods, which occur more often, is less satisfactory when this method is applied.

3) *Combination of principle (1) and (2):* In this method, a combination of the principles (1) and (2) is followed. The principle (1) is followed for the lower portion of the flood reserve to achieve the maximum benefits by controlling the earlier part of the flood. Thereafter releases are made as scheduled for the reservoir design flood as in principle (2). In most cases this plan will result in the best overall regulation, as it combines the good points of both the methods.

4) *Flood control in emergencies:* It is advisable to prepare an emergency release schedule that uses information on reservoir data immediately available to the operator. Such schedule should be available with the operator to enable him to comply with necessary precautions under extreme flood conditions.

b) Conservation: Reservoirs meant for augmentation of supplies during lean period should usually be operated to fill as early as possible during filling period, while meeting the requirements. All water in excess of the requirements of the filling period shall be impounded. No spilling of water over the spillway will normally be permitted until the FRL is reached. Should any flood occur when the reservoir is at or near the FRL, release of flood waters should be affected, so as not to exceed the discharge that would have occurred had there been no reservoir. In case the year happens to be dry, the draft for filling period should be curtailed by applying suitable factors. The depletion period should begin thereafter. However, in case the reservoir is planned with carry-over capacity, it is necessary to ensure that the regulation will provide the required carry-over capacity at the end of the depletion period.

Operation of multi purpose reservoirs: The general principles of operation of reservoirs with these multiple storage spaces are described below:

1. Separate allocation of capacities- When separate allocations of capacity have been made for each of the conservational uses, in addition to that required for flood control, operation for each of the function shall follow the principles of respective functions. The storage available for flood control could, however be utilized for generation of secondary power to the extent possible. Allocation of specific storage space to several purposes with the conservation zone may some times be impossible or very costly to provide water for the various purposes in the quantities needed and at the time they are needed.

2. Joint use of storage space- In multi-purpose reservoir where joint use of some of the storage space or storage water has been envisaged, operation becomes complicated due to competing and conflicting demands. While flood control requires low reservoir level, conservation interests require as high a level as is attainable. Thus, the objectives of these functions are not compatible and a compromise will have to be effected in flood control operations by sacrificing the requirements of these functions. In some cases parts of the conservational storage space is utilized for flood moderation, during the earlier stages of the monsoon. This space has to be filled up for conservation purpose towards the end of monsoon progressively, as it might not be possible to fill up this space during the post-monsoon periods, when the flows are insufficient even to meet the current requirements. This will naturally involve some sacrifice of the flood control interests towards the end of the monsoon.

4.5.13.2 *Operation of system of reservoirs*

It is not very uncommon to find a group or 'system' of reservoirs either in a single river or in a river and its tributaries. An example of the former are the dams proposed on the river Narmada (Figure 7) and an example of the latter are the dams of the Damodar Valley project (Figure 8).

In case of system of reservoirs, it is necessary to adopt a strategy for integrated operated of reservoirs to achieve optimum utilization of the water resources available and to benefit the best out of the reservoir system.

In the preparation of regulation plans for an integrated operation of system of reservoirs, principles applicable to separate units are first applied to the individual reservoirs. Modifications of schedule so developed should then be considered by working out several alternative plans. In these studies optimization and simulation techniques may be extensively used with the application of computers in water resources development.

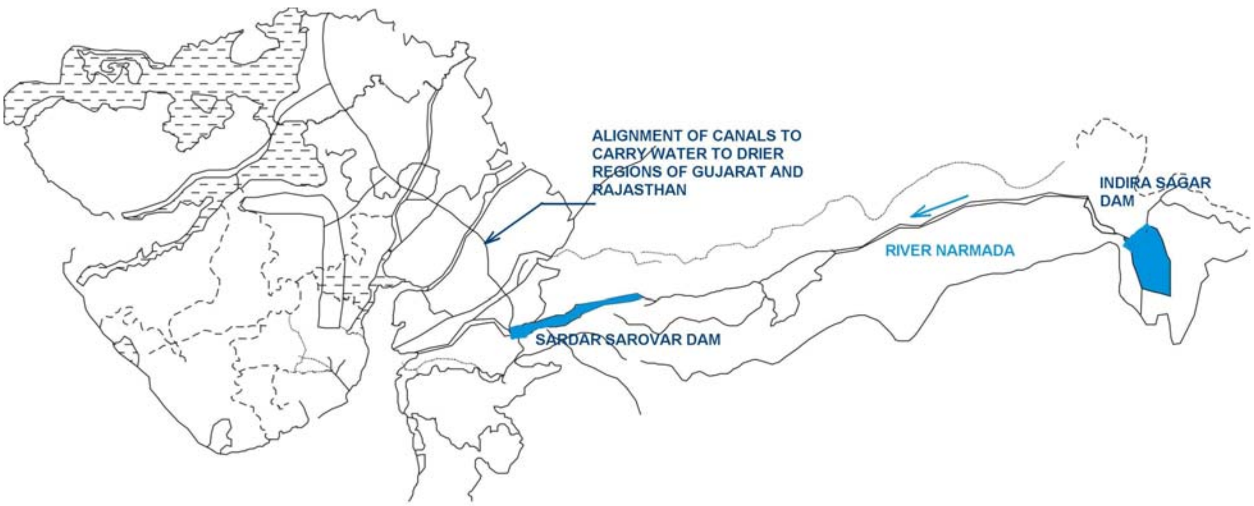


FIGURE 7. NARMADA VALLEY DEVELOPMENT PROJECT

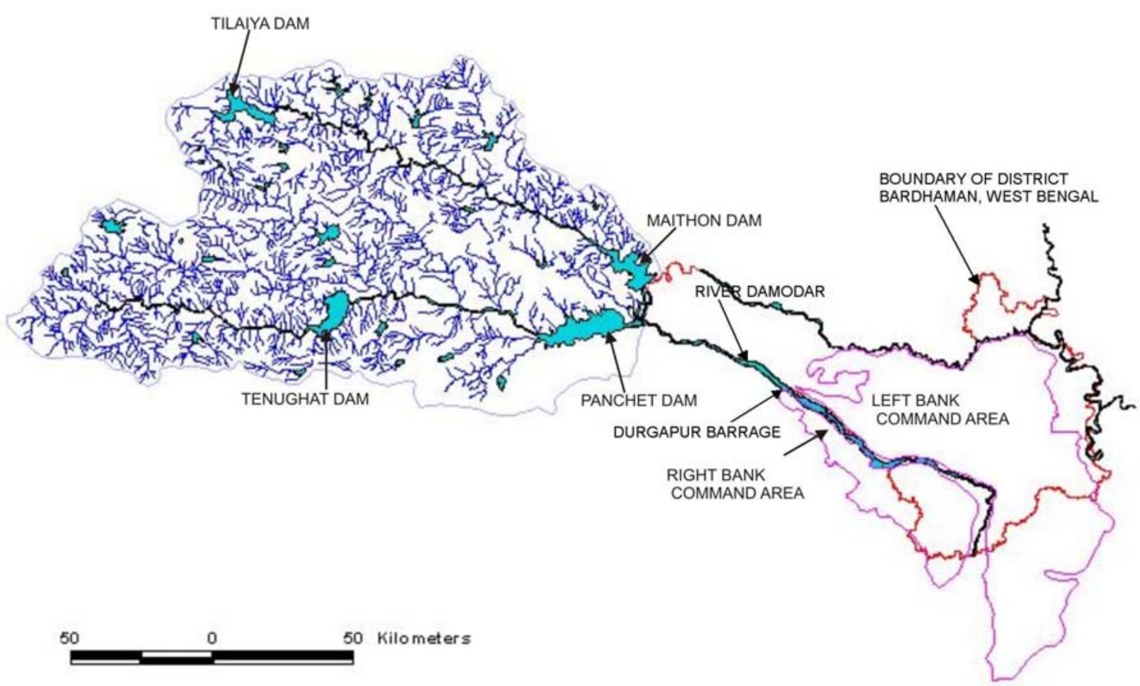


FIGURE.8 DAMODAR VALLEY DEVELOPMENT PROJECTS

Module
4
Hydraulic Structures for Flow Diversion and Storage

Lesson
6
Design and Construction of Concrete Gravity Dams

Instructional objectives

On completion of this lesson, the student shall learn:

1. The different components of concrete gravity dams and their layouts
2. Design steps for of concrete gravity dam sections
3. The expected loadings for gravity dams
4. Stability analysis of gravity dam sections
5. Construction processes for gravity dams
6. Foundation preparation for gravity dams
7. Temperature control for mass concrete dams
8. Instrumentations in concrete dams

4.6.0 Introduction

Dams constructed out of masonry or concrete and which rely solely on its self weight for stability fall under the nomenclature of gravity dams. Masonary dams have been in use in the past quite often but after independence, the last major masonry dam structure that was built was the N***agarjunsagar Dam*** on river ***Krishna*** which was built during 1958-69. Normally, coursed rubble masonry was used which was bonded together by lime concrete or cement concrete. However masonry dam is no longer being designed in our country probably due to existence of alternate easily available dam construction material and need construction technology. In fact, gravity dams are now being built of mass concrete, whose design and construction aspects would be discussed in this chapter. There are other dams built out of concrete like the Arch/Multiple Arch or Buttress type. These have however not been designed or constructed in India, except the sole one being the arch dam at ***Idukki*** on river ***Periyar***. In India the trend for concrete dam is only of the gravity type and therefore the design other types of concrete dams have not been discussed in this course. Interested readers may know more about such dams from standard books on the subject like Engineering of Large Dams by Henry H. Thomas, Volumes I and II published by John Wiley and Sons (1976). A slightly outdated publication, Engineering of Dams, Volumes I, II and III by W P Creager, J D Justin, and J Hinds published by John Wiley and Sons (1917) has also been long considered a classic in dam engineering, though many new technologies have do not find mention here.

It is important to note that, it is not just sufficient to design a strong dam structure, but it is equally important to check the foundation as well for structural integrity. For concrete dams, the stress developed at the junction of the base becomes quite high, which the foundation has to resist. Usually concrete gravity dams are constructed across a river by excavating away the loose overburden till firm rock is encountered which is considered as the actual foundation. Nevertheless not all rocks are of the same quality; they vary with different geological materials and the process by which they have been formed over the years. For example, the hills of the Himalayan range of the mountains are considered geologically young, as well as weaker than the massif of the Deccan plateau. The quality of foundation not only affects the design, it also guides the type of dam that would be suited at a design site. Hence, discussions on the ground foundation aspects have been introduced in this lesson as well.

It may also be realized that designing a dam based on field data (like the geometry of the river valley, the foundation allowable bearing capacity .etc) is not the only part that a water resource engineer has to do. He has to get it constructed at the design site which may easily take anywhere between 5 to 10 years or even more depending on the complexity of the work and the volume and type of the structure. It may easily be appreciated that constructing a massive structure across a flowing river is no easy task. In fact tackling of the monsoon flows during the years of construction is a difficult engineering task.

4.6.1 Concrete gravity dam and apparent structures- basic layout

The basic shape of a concrete gravity dam is triangular in section (Figure 1a), with the top crest often widened to provide a roadway (Figure 1b).

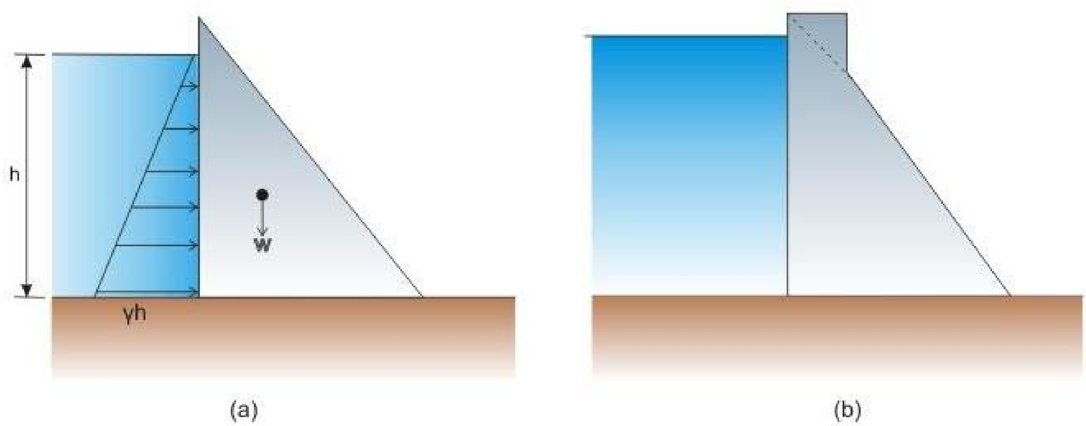


FIGURE 1 : Concrete gravity dam section (a) Basic triangular shape (b) Modified shape

The increasing width of the section towards the base is logical since the water pressure also increases linearly with depth as shown in Figure 1a. In the figure, ***h*** is assumed as the depth of water and ***γh*** is the pressure at base, where ***γ*** is the unit weight of water (9810 N/m³), ***W*** is the weight of the dam body. The top portion of the dam (Figure 1b) is widened to provide space for vehicle movement.

A gravity dam should also have an appropriate spillway for releasing excess flood water of the river during monsoon months. This section looks slightly different from the other non-overflowing sections. A typical section of a spillway is shown in Figure 2.

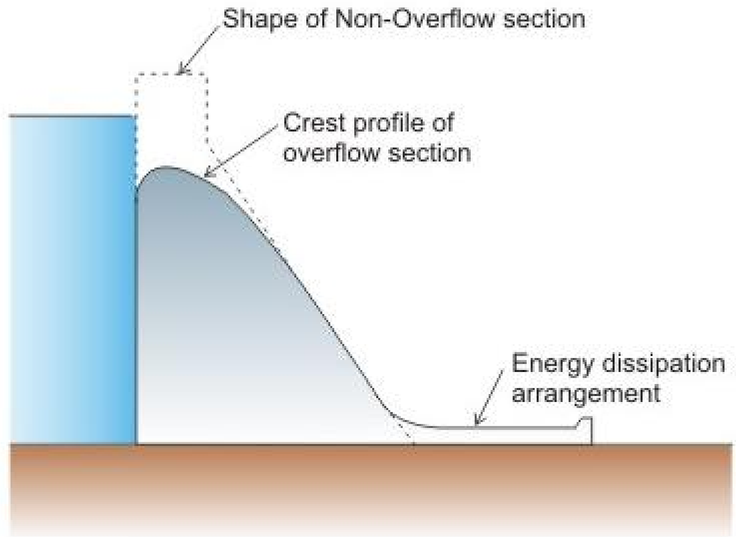


FIGURE 2: Typical overflow section of a gravity dam

The flood water glides over the crest and downstream face of the spillway and meets an energy dissipating structure that helps to kill the energy of the flowing water, which otherwise would have caused erosion of the river bed on the downstream. The type of energy dissipating structure shown in Figure 2 is called the stilling basin which dissipates energy of the fast flowing water by formation of hydraulic jump at basin location. This and other types of spillway and energy dissipators are discussed in a subsequent section. Figure 3 shows the functioning of this type of spillway

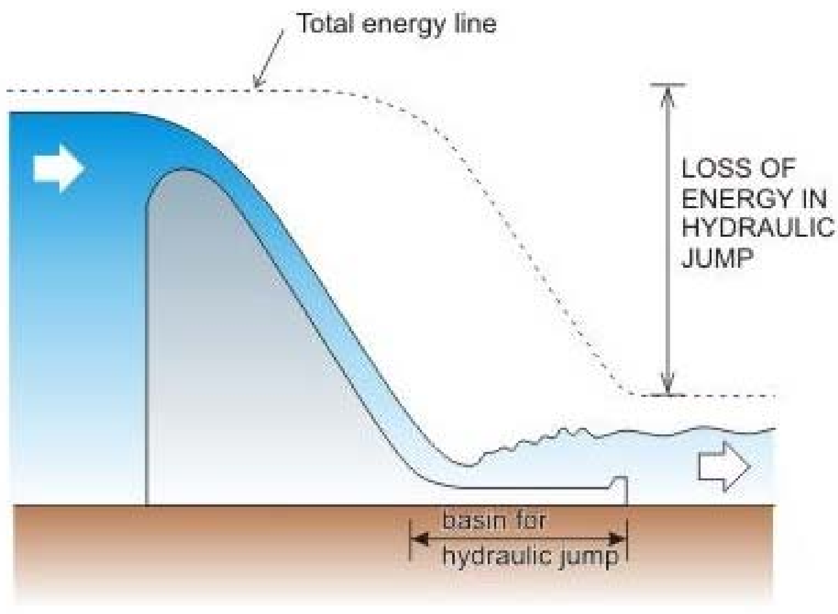


FIGURE 3: Water flowing over a spillway

Usually, a spillway is provided with a gate, and a typical spillway section may have a radial gate as shown in Figure 4. The axis or ***trunnion*** of the gate is held to ***anchorages*** that are fixed to ***piers***.

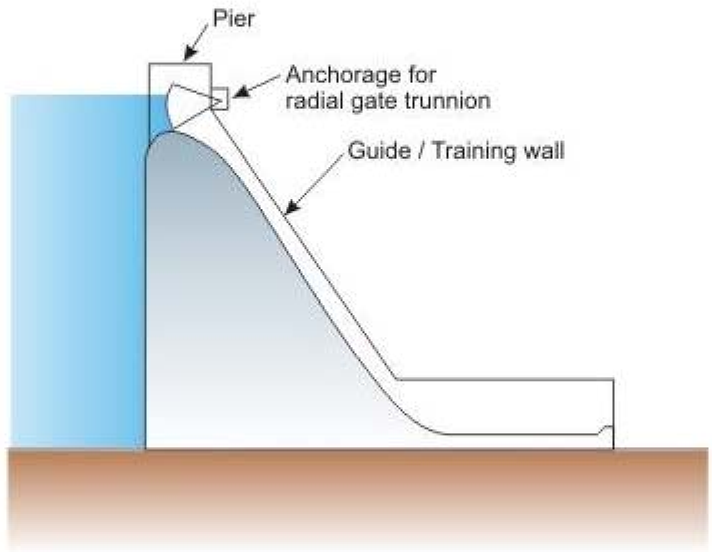


FIGURE 4. A gated spillway section

Also shown in the figure is a ***guide wall*** or ***training wall*** that is necessary to prevent the flow crossing over from one bay (controlled by a gate) to the adjacent one. Since the width of a gate is physically limited to about 20m (limited by the availability of hoisting motors), there has to be a number of bays with corresponding equal number of gates separated by guide walls in a practical dam spillway.

The upstream face of the overflowing and non-overflowing sections of a gravity dam are generally kept in one plane, which is termed as the dam axis or sometimes referred to as the dam base line (Figure 5).

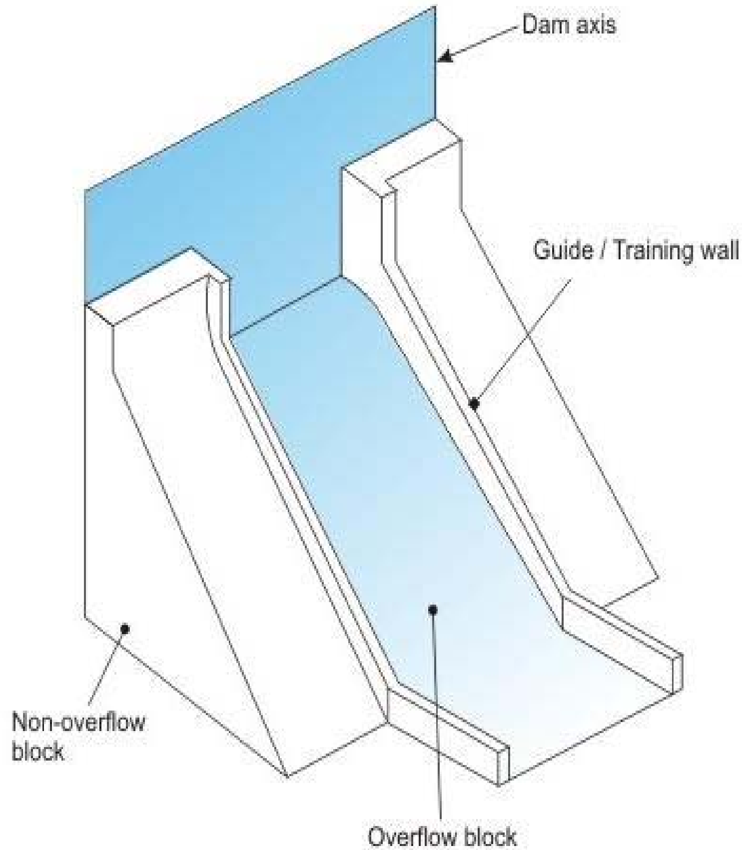


FIGURE 5. Co-planar upstream faces of overflow and non-overflow blocks.

Since the downstream face of the dam is inclined, the plane view of a concrete gravity dam with a vertical upstream face would look like as shown in Figure 6.

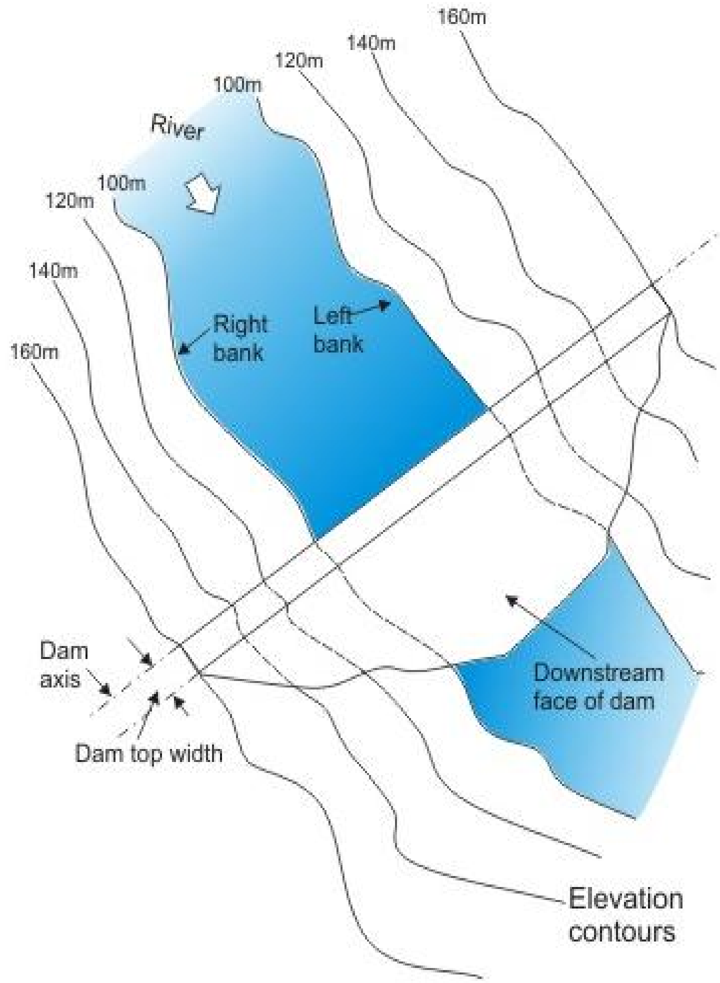


FIGURE 6. A typical layout of a concrete gravity dam in plan.

If a concrete gravity dam is appreciably more than 20 m in length measured along the top of the dam from one bank of the river valley to the other, then it is necessary to divide the structure into blocks by providing ***transverse contraction joints***. These joints are in vertical planes that are at the right angle to the dam axis and separated about 18-20 m. The spacing of the joints is determined by the capacity of the concreting facilities to be used and considerations of volumetric changes and attendant cracking caused by shrinkage and temperature variations. The possibilities of detrimental cracking can be greatly reduced by the selection of the proper type of the cement and by careful control of mixing and placing procedures. The contraction joints allow relieving of the thermal stresses. In plan, therefore the concrete gravity dam layout would be as shown in Figure 7, where the dam is seen to be divided into blocks separated by the contraction joints.

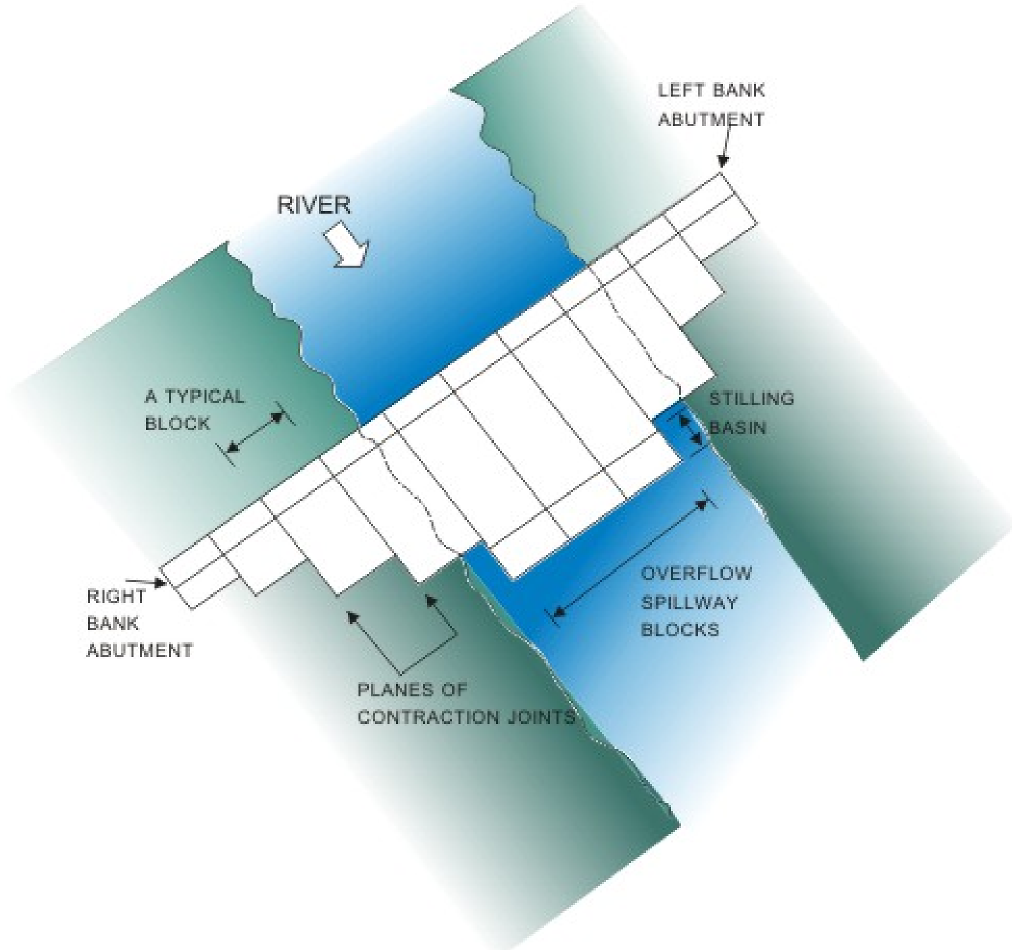


FIGURE 7. Layout of blocks for concrete gravity dams.

The base of each block of the dam is horizontal and the blocks in the centre of the dam are seen to accommodate the spillway and ***energy dissipators***. The blocks with maximum height are usually the spillway blocks since they are located at the deepest portion of the river gorge, as shown in Figure 7. The upstream face of the dam is sometimes made inclined (Figure 8a) or kept vertical up to a certain elevation and inclined below (Figure 8b).

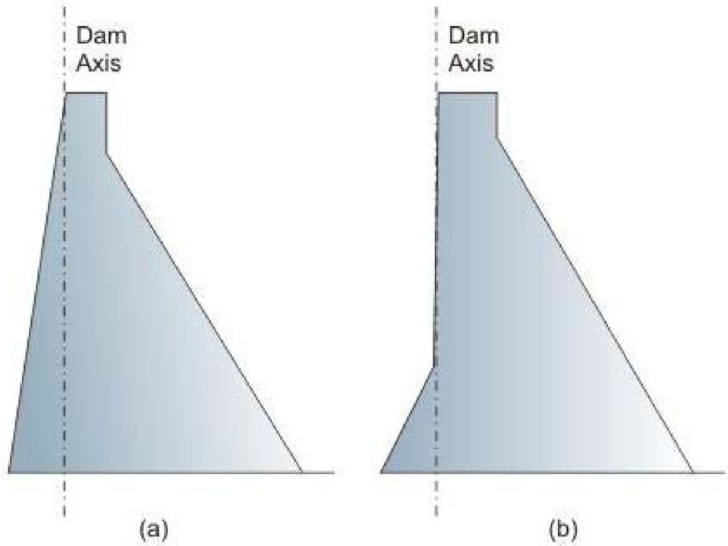


FIGURE 8. Upstream inclined face for concrete gravity dams. (a) Full face inclined; (b) Partly inclined

In plan, the dam axis may be curved as for the ***Indira Sagar Dam*** (Figure 9), but it does not provide any arch action since each block is independent being separated by a construction joint.

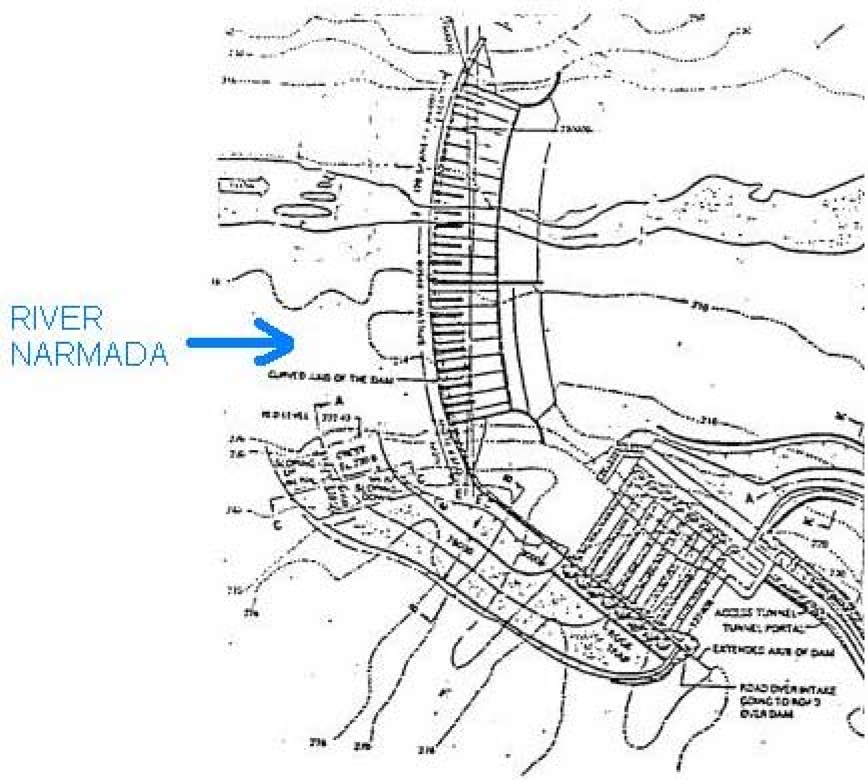


FIGURE 9. Indira Sagar dam curved layout

The construction joints in a concrete gravity dams provide passage through the dam which unless sealed, would permit the leakage of water from the reservoir to the downstream face of the dam. To check this leakage, water stops are installed in the joints adjacent to the upstream face (Figure 10).

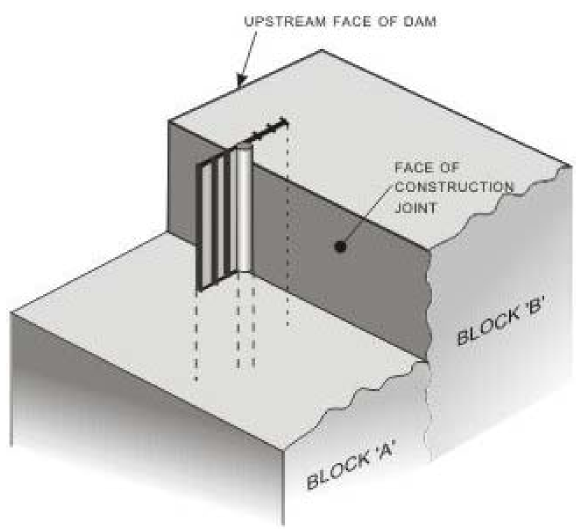


FIGURE 10. Typical installation of a water stop near upstream face of dam

Not very long ago, different types of water – stops were being used like copper strips, asphalt grouting, etc. apart from rubber seals. However, unsatisfactory performance of the copper strips and asphalt seals and advancement in the specifications and indigenous manufacture of good quality Polyvinyl Chloride (PVC) water stops have led to the acceptance of only the PVC water stops for all future dam construction. This has been recommended by the following Bureau of Indian Standard codes:

- IS 12200-2001 "Provision of water stops at transverse construction joints in masonry and concrete dams- code of practice"
- IS 15058-2002 " PVC water stops at transverse construction joints in masonry and concrete dams- code of practice"

The recommended cross section of a PVC water stop is shown in Figure 11.

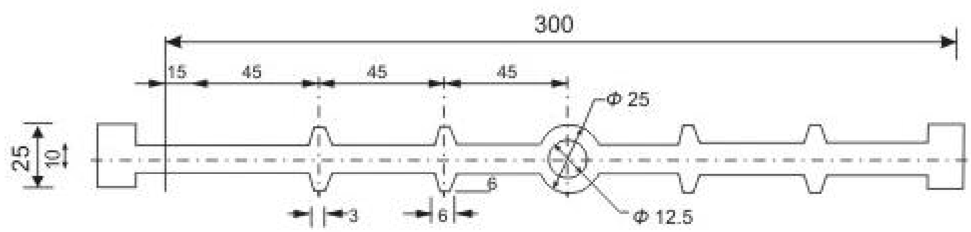


FIGURE 11. Cross section of a PVC water stop
(All dimensions are in mm)

According to the recommendations of IS 1220-2001, there needs to be more than one layer of PVC water stop at each joint between two blocks, as shown in Figure 12

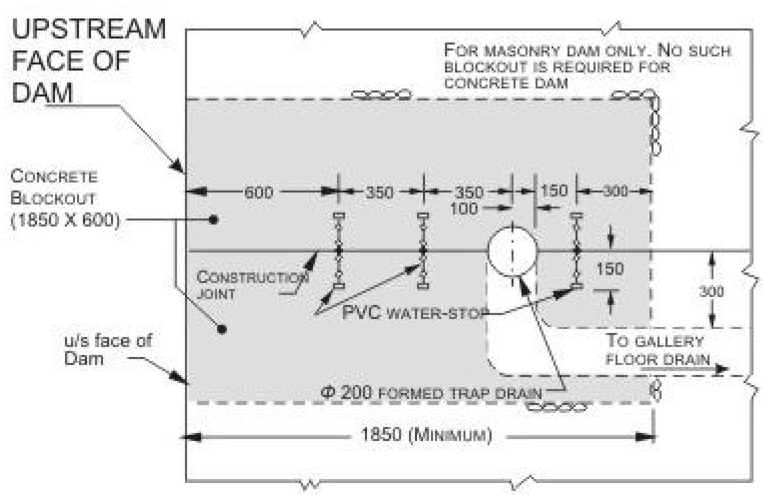


FIGURE 12. Sectional plan through two adjacent blocks showing relative placement of PVC water stops. Note the formed trap drain for removing any leakage water

In the vertical plane, the water stop needs to continue right up to the elevation of the maximum water level plus at least 1000 mm as shown in Figures 13 and 14.

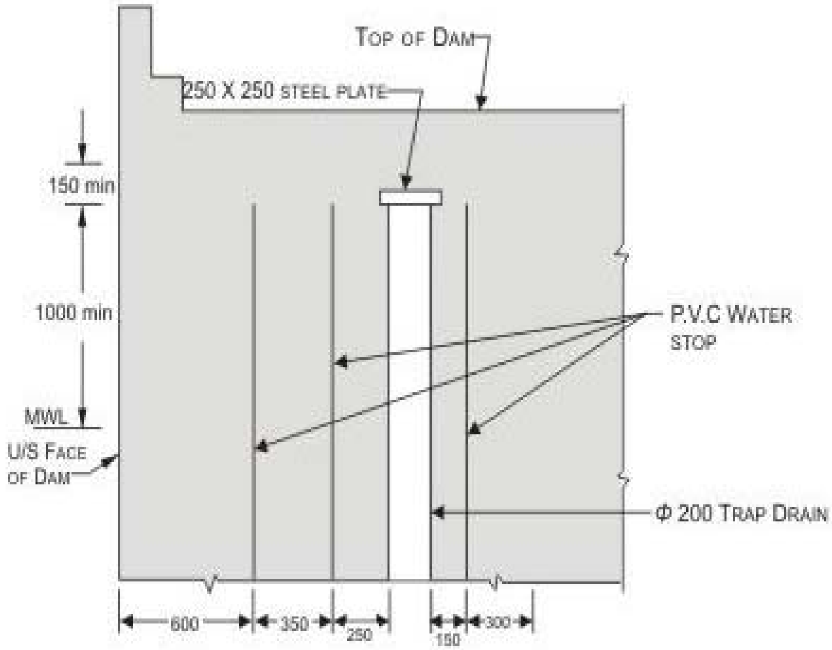


FIGURE 13. Water stop details near top of non-overflow section of concrete gravity dam. The formed trap drain also extends upto the same elevation as water stops.

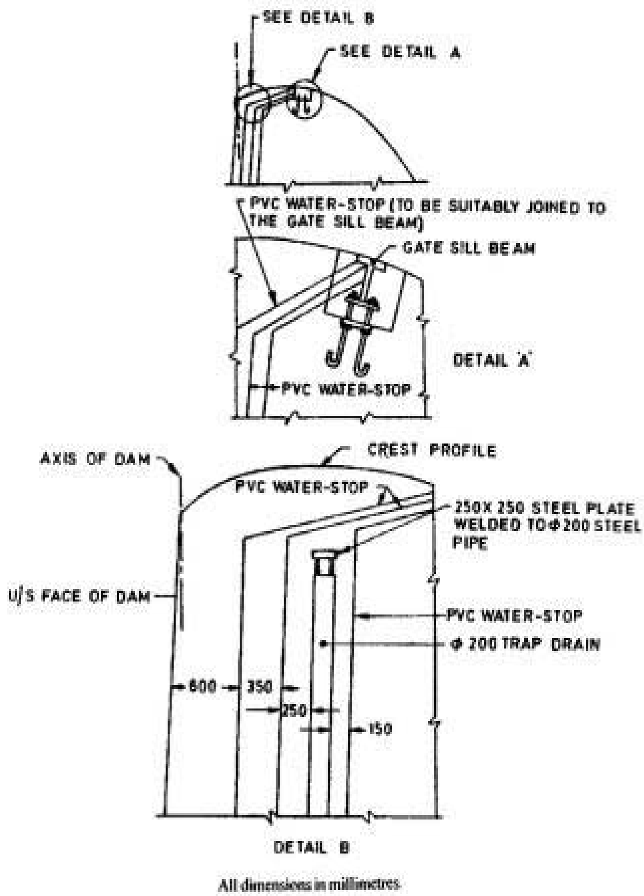


FIGURE 14. Water stop details near top of the crest of spillway (overflow) section of a concrete gravity dam.

In spite of the provision of water stops there may be leakage through the body of the dam due to the pressure of water from the upstream. In order to remove this water, vertical formed drains to trap the seeping water through the contraction joint is recommended as may be observed from Figures 12 to 13. These vertical drains convey the drainage water to a drainage gallery at some lower level within the dam body as shown in Figure 15.

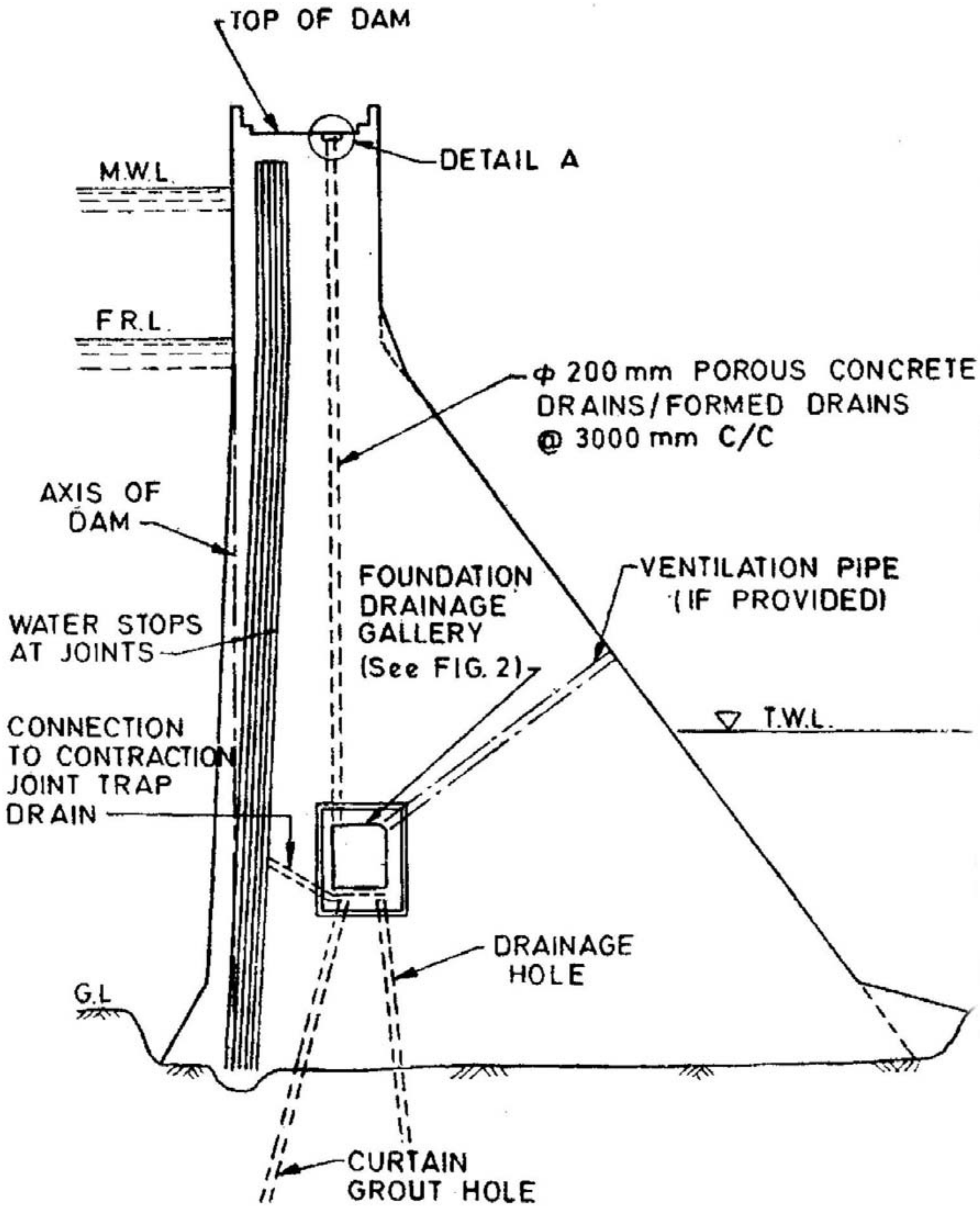


FIGURE 15. Vertical formed drain connected to drainage gallery.

It may be noticed that the formed drain is provided not only at the transverse contraction joint between two adjacent blocks of a concrete gravity dam, they are placed at an equal interval of about 3metres centre to centre in all the dam blocks, as recommended by the Bureau of Indian Standard code IS 10135-1985" Code of practice for drainage system for gravity dams, their foundations and abutments".

These drains are required to intercept any seeping water from the reservoir through the upstream face of the concrete dam. The vertical drains may be formed drains or may be filled with porous concrete, which is formed by mixing 1 part cement with 5 parts of 5 to 20 mm size aggregates. It is also recommended in IS 10135 that the permeability of a 200 mm thick slab of this concrete under a head of 100 mm should be such that the discharge should not be less than 30litres/min/m². It is important to note that the general mass concrete of a dam, though not as porous as the lean concrete mentioned above also seeps water but to a very lesser quantity.

The foundation drainage gallery (shown in Figure 15) is connected to all the vertical drains passing through the body of the dam (Figure 16).

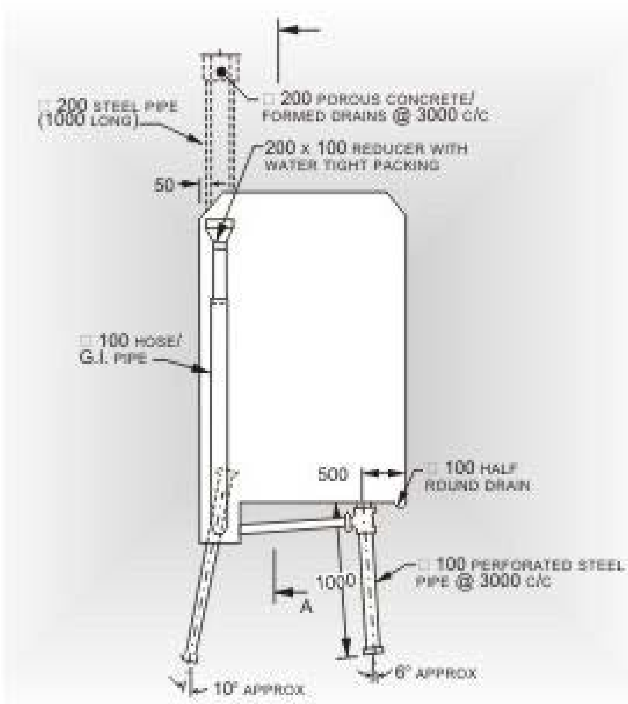


FIGURE 16. Details of a foundation drainage gallery

This gallery, of a size large enough for a person to walk comfortably, extends throughout the length of the dam at about the same height above.

In plan the gallery is near and parallel to the axis of the dam. The dam foundation is as shown in Figure 17.

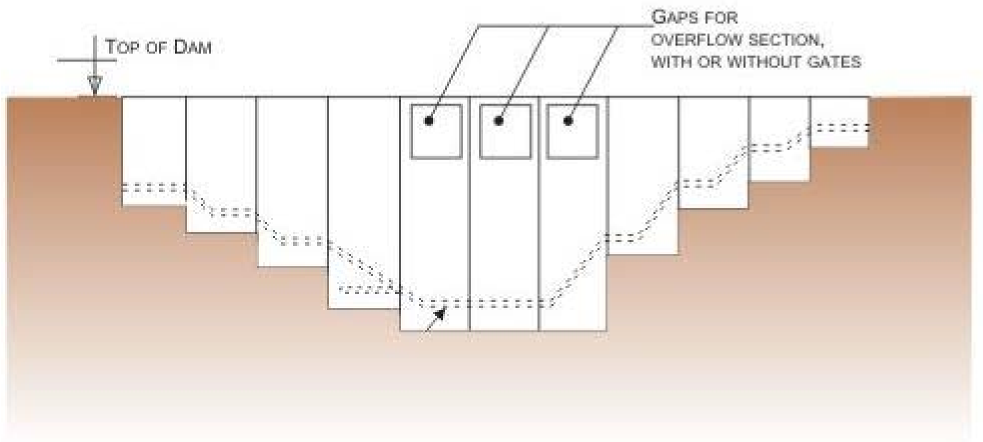


FIGURE 17. Alignment of drainage gallery shown in an elevation view of a concrete gravity dam

The foundation drainage gallery should have a small slope, about 1 in 1000 to drain away the collected water in the side drain up to a sump at the lowest level from where the water may be pumped out to the downstream side of the dam. As such, it is compulsorily recommended to provide a drainage gallery whose normal foundation level is more than 10metres measured from the crest level of the overflow portion of the dam. For dams with maximum height less than 10metres between the deepest foundation level and the overflow section crest level, the provision of a drainage gallery is optional.

Another important location of water seeping through is the bottom of a concrete gravity dam. This water seeps through the foundation material, like through the joints of a fractured rock upon which the dam is founded. This seepage water also causes uplift at the base of the dam and produces an upward force that must be countered by the weight of the dam apart from countering other forces discussed in the next section. Thus drainage of foundation material is an important consideration in concrete dam design.

It may be observed from the details of a foundation drainage gallery that 100mm diameter perforated steel pipes are usually provided through the floor of the gallery that penetrates into the foundation. These drainage holes are drilled once the foundation gallery base has been constructed and the foundation grouting (explained later) is completed. The size, spacing and the depth of these holes are assumed on the basis of physical characteristics of the foundation rock, foundation condition and the depth of the storage reservoir. The diameter of the hole may be kept at 75mm and the spacing of holes may be kept at 3m centre to centre. The depth of holes may be kept between 20 and 40 percent of the maximum reservoir depth and between 30 and 75 percent depth of the curtain grouting (explained later). The drainage holes of 75mm diameter are drilled through 100 mm diameter pipe embedded in concrete portion of the dam. For foundation drainage holes in soft foundation, the arrangement shown in Figure 18 may be adopted.

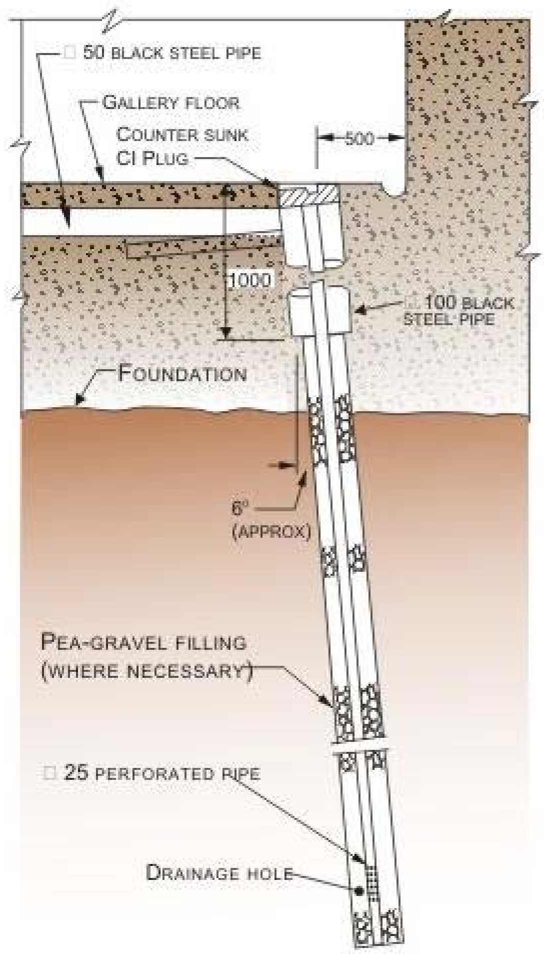


FIGURE 18. Details of foundation drainage gallery in soft foundation

Another function of the foundation gallery is to provide a space for drilling holes for providing what is called grout curtain, which is nothing but a series of holes drilled in a line deep inside the foundation and filled with pressurized cement mortar. The location of a grout hole within a foundation gallery may be seen from Figure 16. The purpose of providing these holes and injecting them with cement mortar is to create a barrier in the foundation rock at the heel of the dam (Figure 19) which will prevent leakage of water from the reservoir and thus reduce uplift pressure at the bottom of the dam.

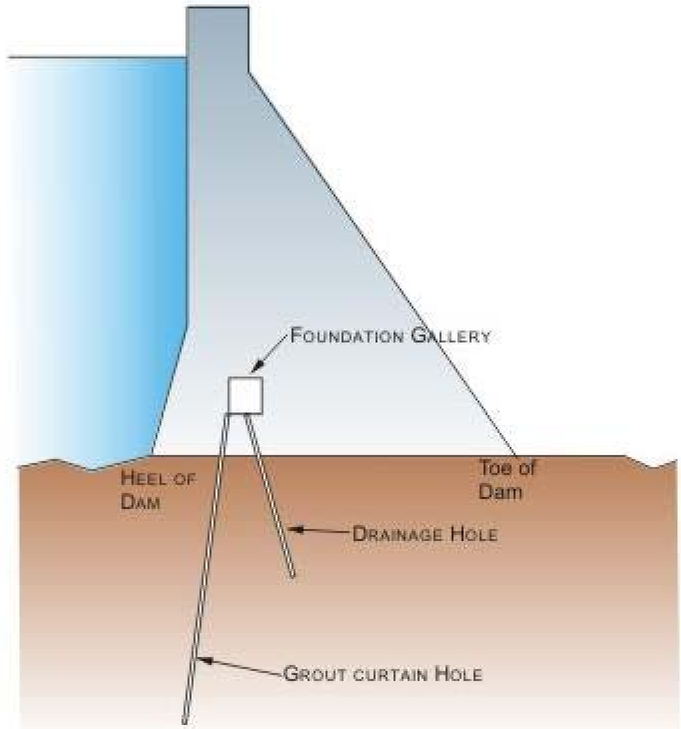


FIGURE 19. Drainage and grout curtain holes

The depth of the grout curtain holes depends upon the nature of the rock in foundation and in general, it may range from 30 to 40 percent of the head of the water on good foundation and to 70 percent of head on poor foundations.

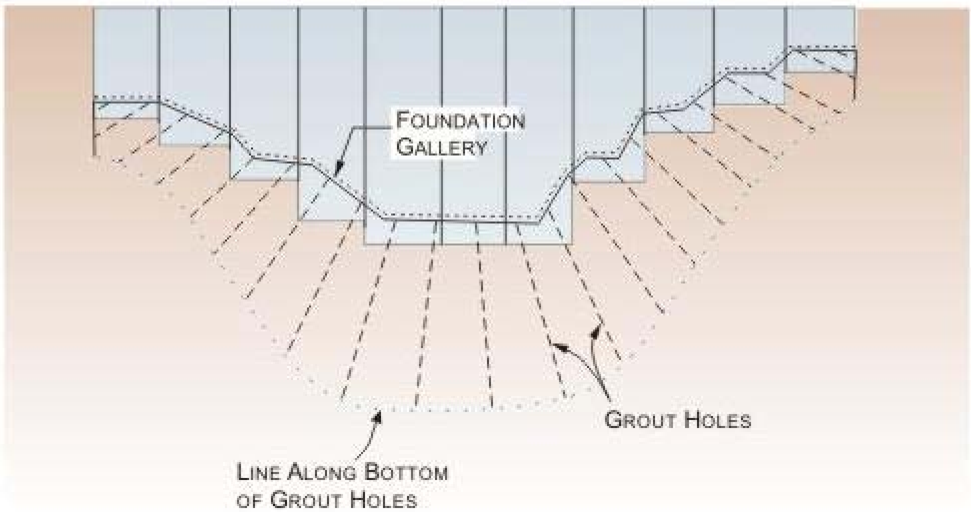


FIGURE 20. Series of grout holes forming a grout curtain shown in elevation of a concrete gravity dam

Figure 20 shows a typical layout of grout holes for a concrete gravity dam shown in an elevation view.

According to IS: 11293 (Part2)-1993 "Guidelines for the design of grout curtains", the following empirical criteria may be used as a guide:

$$D= (2/3)\, H + 8 \qquad (1)$$

Where ***D*** is the depth of the grout curtain in meters and ***H*** is the height of the reservoir water in meters.

The grout holes may be either vertical or inclined.

The orientation, plan and inclination of grout holes depend upon the type of joints and the other discontinuities in the foundation rock. The most common practice is to drill holes inclined towards the upstream at 5 to 10 degrees to the vertical.

Apart from the gallery at the foundation level, there could be other galleries located at intermediate levels as shown Figures 21 and 22.

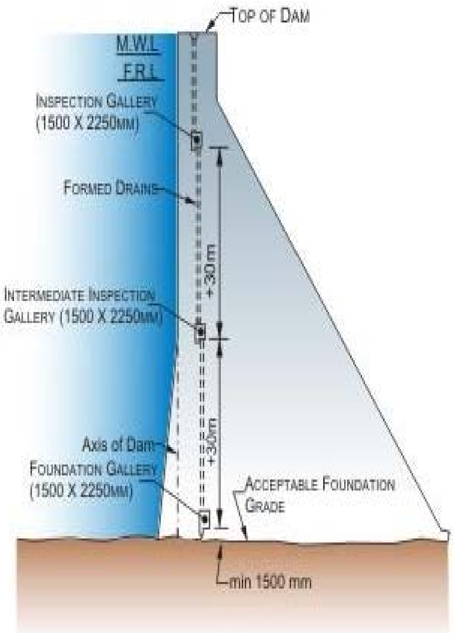


FIGURE 21. Galleries in a non-overflow section of a typical concrete gravity dam.

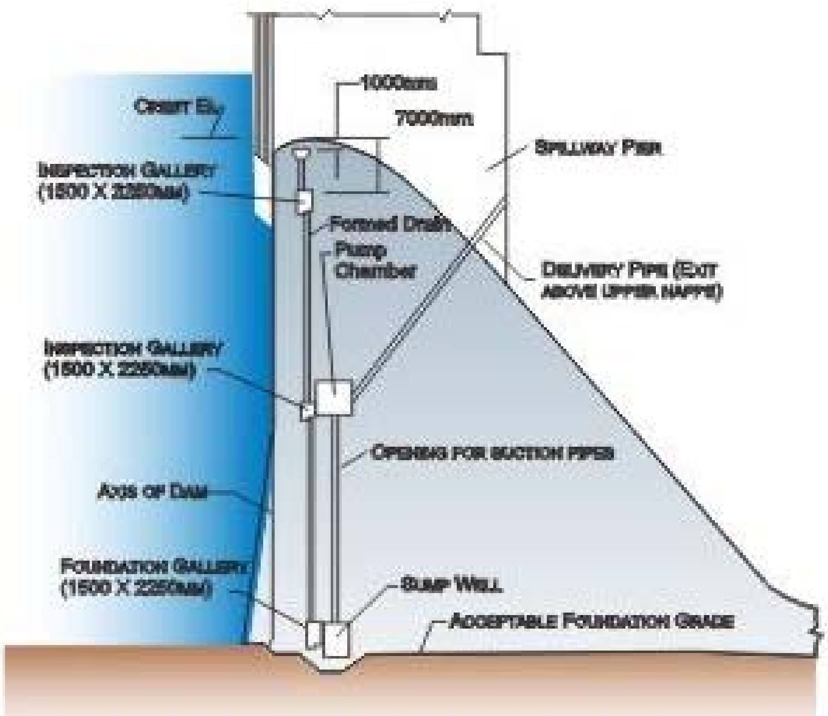


FIGURE 22: Galleries in a overflow-section of a typical concrete gravity dam

At times, gate galleries have to be provided in a dam to give access to and room or the mechanical and electrical equipment required for the operation of gates in outlet conduits, penstocks, etc. Inspection galleries are also sometimes provided to give access to the interior mass of the dam after completion. Foundation, drainage and gate galleries also serve as inspection galleries. In order to connect these galleries that are parallel to the dam axis, transverse galleries called adits are also provided in a dam. Adits providing access to the galleries from outside the dam are also called access gallery or entrance gallery.

It may be interesting to note that all spillways of dams (including gravity dams) may not be necessarily be gated. Ungated spillways have been provided in dams located in remote areas where spillway operation by manual control is difficult. For the gated spillways, radial gates are more common nowadays, though vertical lift gates have been used in some dams earlier. More details about gates and hoists have been presented in a subsequent lesson.

Apart from the openings in dams discussed earlier sluices are also provided in the body of the dam to release regulated supplies of water for a variety of purposes. Details about the types of sluices and their uses are provided separately.

Sometimes the non overflow blocks of a concrete gravity dams are used to accommodate the penstocks (large diameter pipes) which carry water from the reservoir to the powerhouse. Design of the intakes and conveyance systems for power generation have been discussed in another lesson.

4.6.2 Design of concrete gravity Dam sections

Fundamentally a gravity dam should satisfy the following criteria:

1. It shall be safe against overturning at any horizontal position within the dam at the contact with the foundation or within the foundation.
2. It should be safe against sliding at any horizontal plane within the dam, at the contact with the foundation or along any geological feature within the foundation.
3. The section should be so proportional that the allowable stresses in both the concrete and the foundation should not exceed.

Safety of the dam structure is to be checked against possible loadings, which may be classified as primary, secondary or exceptional. The classification is made in terms of the applicability and/or for the relative importance of the load.

1. Primary loads are identified as universally applicable and of prime importance of the load.
2. Secondary loads are generally discretionary and of lesser magnitude like sediment load or thermal stresses due to mass concreting.
3. Exceptional loads are designed on the basis of limited general applicability or having low probability of occurrence like inertial loads associated with seismic activity.

Technically a concrete gravity dam derives its stability from the force of gravity of the materials in the section and hence the name. The gravity dam has sufficient weight so as to withstand the forces and the overturning moment caused by the water impounded in the reservoir behind it. It transfers the loads to the foundations by cantilever action and hence good foundations are pre requisite for the gravity dam.

The forces that give stability to the dam include:

1. Weight of the dam
2. Thrust of the tail water

The forces that try to destabilize the dam include:

1. Reservoir water pressure
2. Uplift
3. Forces due to waves in the reservoir
4. Ice pressure
5. Temperature stresses
6. Silt pressure
7. Seismic forces
8. Wind pressure

The forces to be resisted by a gravity dam fall into two categories as given below:

1. Forces, such as weight of the dam and water pressure which are directly calculated from the unit weight of materials and properties of fluid pressure and
2. Forces such as uplift, earthquake loads ,silt pressure and ice pressure which are assumed only on the basis of assumptions of varying degree of reliability. In fact to evaluate this category of forces, special care has to be taken and reliance placed on available data, experience and judgement.

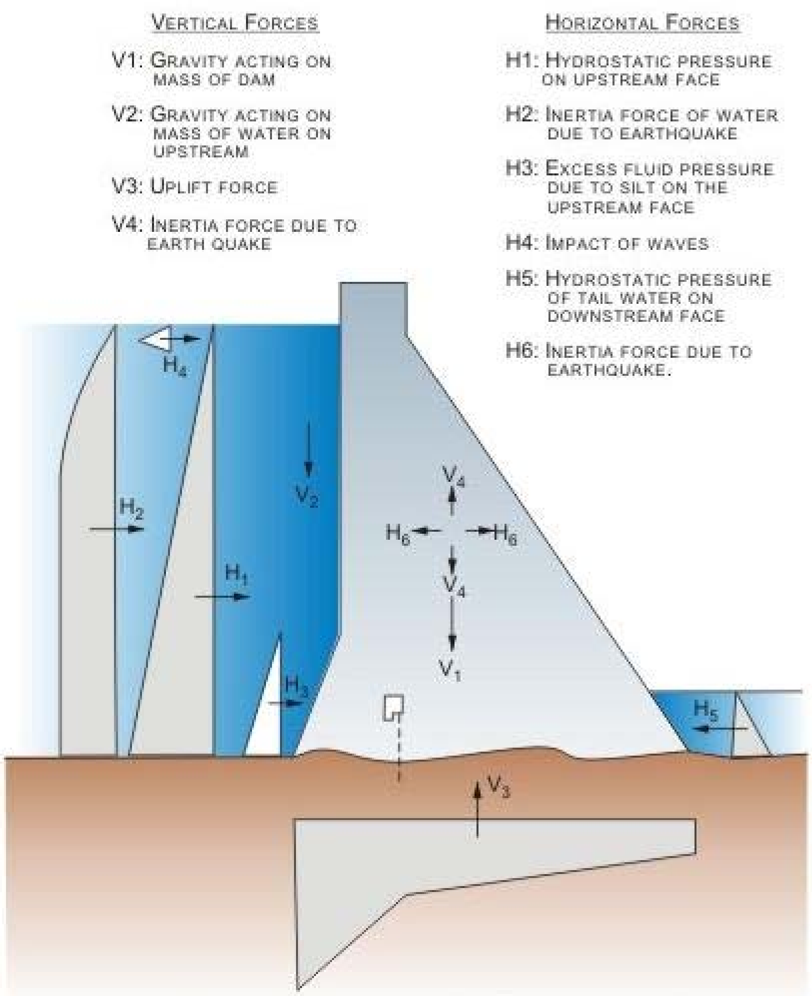


FIGURE 23: Different forces acting on a concrete gravity dam

Figure 23 shows the position and direction of the various forces expected in a concrete gravity dam. Forces like temperature stresses and wind pressure have not been shown. Ice pressures being uncommon in Indian context have been omitted.

For consideration of stability of a concrete dam, the following assumptions are made:

1. That the dam is composed of individual transverse vertical elements each of which carries its load to the foundation without transfer of load from or to adjacent elements. However for convenience, the stability analysis is commonly carried out for the whole block.
2. That the vertical stress varies linearly from upstream face to the downstream face on any horizontal section.

The Bureau of Indian Standards code IS 6512-1984 "Criteria for design of solid gravity dams" recommends that a gravity dam should be designed for the most adverse load condition of the seven given type using the safety factors prescribed.

Depending upon the scope and details of the various project components, site conditions and construction programme one or more of the following loading conditions may be applicable and may need suitable modifications. The seven types of load combinations are as follows:

1. Load combination A (construction condition): Dam completed but no water in reservoir or tailwater
2. Load combination B (normal operating conditions): Full reservoir elevation, normal dry weather tail water, normal uplift, ice and silt (if applicable)
3. Load combination C: (Flood discharge condition) - Reservoir at maximum flood pool elevation ,all gates open, tailwater at flood elevation, normal uplift, and silt (if applicable)
4. Load combination D: Combination of A and earthquake
5. Load combination E: Combination B, with earthquake but no ice
6. Load combination F: Combination C, but with extreme uplift, assuming the drainage holes to be Inoperative
7. Load combination G: Combination E but with extreme uplift (drains inoperative)

It would be useful to explain in a bit more detail the different loadings and the methods required to calculate them. These are explained in the following sections.

4.6.3 Loadings for concrete Gravity Dams

The significant loadings on a concrete gravity dam include the self-weight or dead load of the dam, the water pressure from the reservoir, and the uplift pressure from the foundation. There are other loadings, which either occur intermittently, like earthquake forces, or are smaller in magnitude, like the pressure exerted by the waves generated in the reservoir that his the upstream of the dam face. These loadings are explained in the following section.

4.6.3.1 *Dead load*

The dead load comprises of the weight of the concrete structure of the dam body in addition to pier gates and bridges, if any over the piers. The density of concrete may be considered as 2400 kg/m³. Since the cross section of a dam usually would not be simple, the analysis may be carried out by dividing the section into several triangles and rectangles and the dead load (self weight) of each of these sections (considering unit width or the block width) computed separately and then added up. For finding out the moment of the dead load (required for calculating stresses), the moments due to the separate sub–parts may be calculated individually and then summed up.

4.6.3.2 *Water pressure on dam*

The pressure due to water in the reservoir and that of the tailwater acting on vertical planes on the upstream and downstream side of the dam respectively may be calculated by the law of hydrostatics. Thus, the pressure at any depth **h** is given by **γh** kN/m² acting normal to the surface. When the dam has a sloping upstream face, the water pressure can be resolved into its horizontal and vertical componenets, the vertical component being given by the weight of the water prism on the upstream face and acts vertically downward through the centre of gravity of the water area supported on the dam face.

In spillway section, when the gates are closed, the water pressure can be worked out in the same manner as for non–overflow sections except for vertical load of water on the dam itself. During overflow, the top portion of the pressure triangle gets truncated and a trapezium of pressure acts (Figure 24).

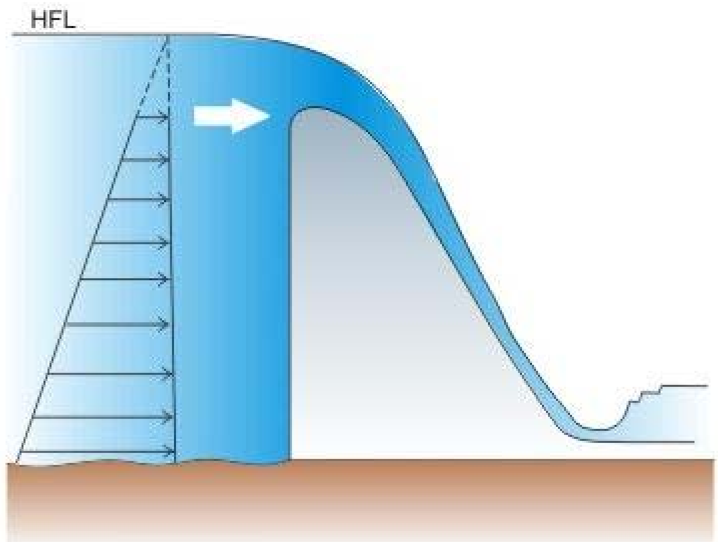


FIGURE 24. Horizontal water force on spillway block during flood water overflow

The pressure due to tailwater is obtained in a similar manner as for the upstream reservoir water.

In case of low overflow dams, the dynamic effect of the velocity of approach may be significant and deserve consideration.

4.6.3.3 *Uplift pressures*

Uplift forces occur as internal pressure in pores, cracks and seams within the body of the dam, at the contact between the dam and its foundation and within the foundation. The recent trends for evaluating uplift forces is based on the phenomenon of seepage through permeable material. Water under pressure enters the pores and fissures of the foundation material and joints in the dam. The uplift is supposed to act on the whole width plane, that is being considered, either at the base or at any position within the dam. The uplift pressure on the upstream end of the considered horizontal plane is taken as γh_u where h_u is the depth of water above the plane. On the downstream the value is γh_d where h_d is again the depth of water above the plane.

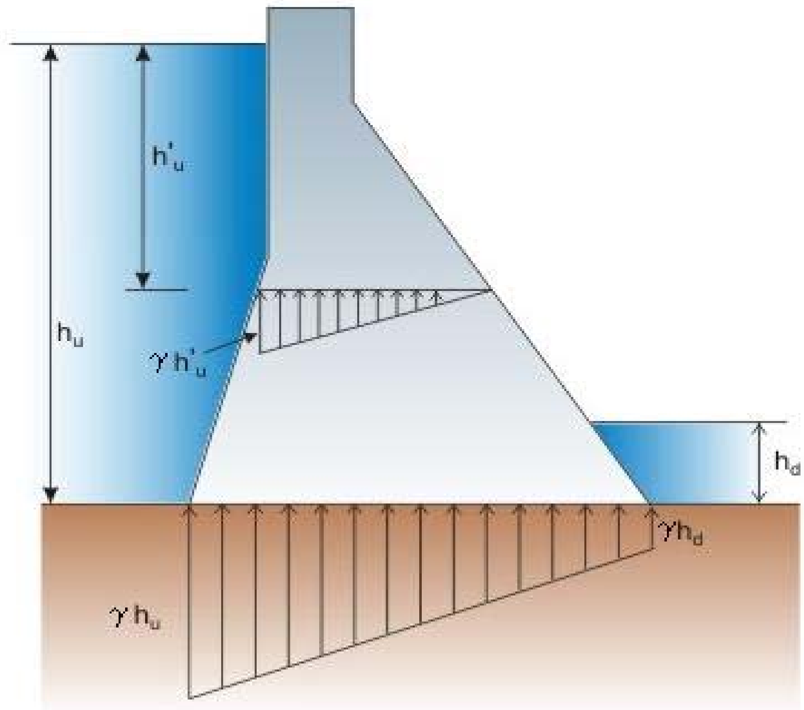


FIGURE 25. Uplift pressure at base and at any general plane in the dam body. Drainage holes are not considered.

Figure 25 illustrates the uplift pressure on a concrete gravity dam's non overflow section through two planes – one at the base and the other at the horizontal plane which is above the tail water level. In Figure 25, the drainage holes either in the body of the dam, or within the foundation has not been considered. If the effects of the drainage holes are considered, then the uplift pressure diagram gets modified as shown in Figure 26. If there is crack at any plane of the dam, or at the base then the uplift pressure diagram gets further modified as shown in Figure27.

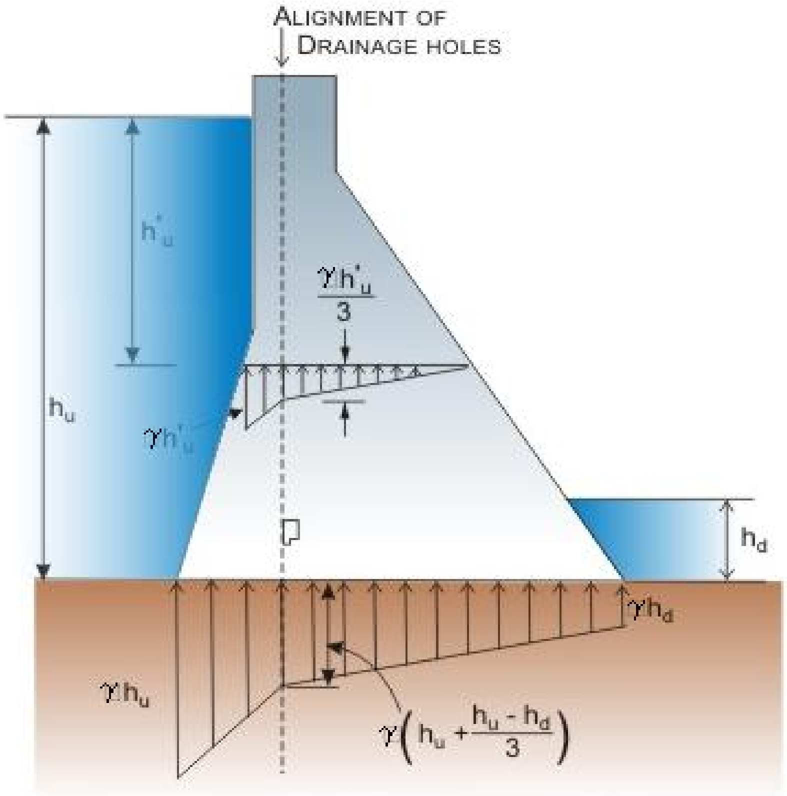


FIGURE 26: Assumed uplift pressure considering presence of drainage holes

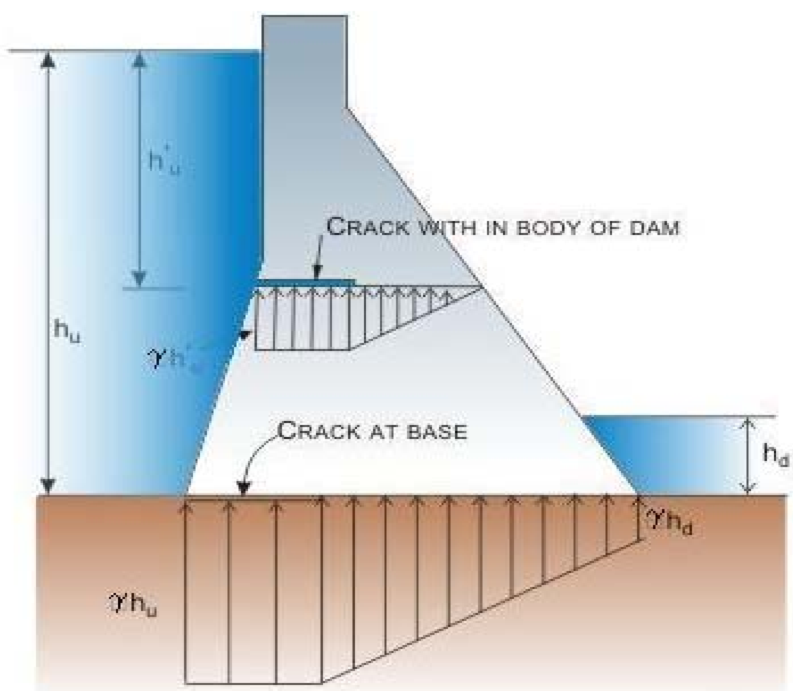


FIGURE 27: Uplift pressure diagrams considering horizontal cracks at any general plane/at the base.

As such, the uplift pressure is assumed to act throughout the base area. Further it is also assumed that they remain unaffected by earthquakes.

4.6.3.4 *Silt pressure*

The weight and the pressure of the submerged silt are to be considered in addition to weight and pressure of water. The weight of the silt acts vertically on the slope and pressure horizontally, in a similar fashion to the corresponding forces due to water. It is recommended that the submerged density of silt for calculating horizontal pressure may be taken as 1360 kg/m^3. Equivalently, for calculating vertical force, the same may be taken as 1925 kg/m^3.

4.6.3.5 *Earthquake (seismic) forces*

Earthquake or seismic activity is associated with complex oscillating patterns of acceleration and ground motions, which generate transient dynamic loads due to inertia of the dam and the retained body of water. Horizontal and vertical accelerations are not equal, the former being of greater intensity.

The earthquake acceleration is usually designated as a fraction of the acceleration due to gravity and is expressed as $\alpha \cdot g$, where α is the ***Seismic Coefficient***. The seismic coefficient depends on various factors, like the intensity of the earthquake, the part or zone of the country in which the structure is located, the elasticity of the material of the dam and its foundation, etc. For the purpose of determining the value of the seismic coefficient which has to be adopted in the design of a dam, India has been divided into five seismic zones, depending upon the severity of the earthquakes which may occur in different places. A map showing these zones is given in the Bureau of Indian Standards code IS: 1893-2002 (Part-1) “Criteria for earthquake resistant design of Structures (fourth revision)”, and has been reproduced in Figure 28.

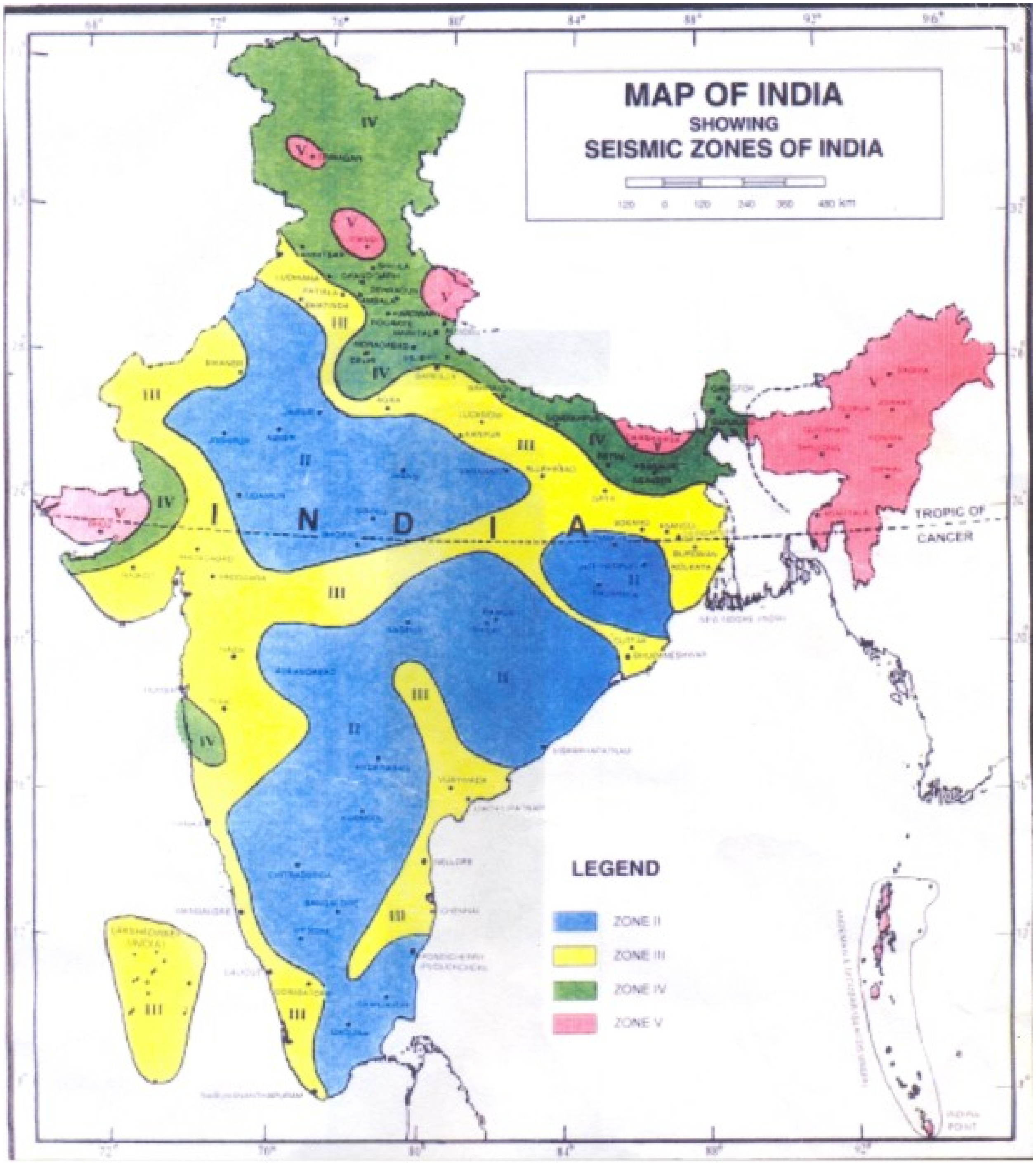


FIGURE 28. Seismic zones of India as per IS : 1893 - 2002 (Part 1)

The BIS code also indicates two methods that may be used for determining the coefficient α.

These are:

1. The Seismic Coefficient Method (for dam height up to 100m)

$$\alpha=\beta I\alpha_0 \qquad (2)$$

2. The Response Spectrum Method (for dams taller than 100m)

$$\alpha=\beta I\Phi_0\,(\Sigma_\alpha/\gamma) \qquad (3)$$

In the above expressions,

β = Soil-foundation system factor, which may be taken as 1.0 for dams
I = Importance factor, which may be taken as 2.0 for dams
α_0=The basic seismic Coefficient, the value of which for each of the five zones is given the following table:

Zone	[illegible]
I	0.01
II	0.02
III	0.04
IV	0.05
V	0.08

F_0 = The seismic Zone Factor for average acceleration spectra, the value of which for each of the five zonesis given in the following table:

Zone	F
I	0.05
II	0.10
III	0.20
IV	0.25
V	0.40

S_a/g = the average acceleration coefficient that has to be read from Figure 29, corresponding to the appropriate natural period of vibration and damping of the structure.

The natural (or fundamental) period of vibration of a gravity dam may be determined by the following expression:

$$T = 5.59\,\frac{H^2}{B}\sqrt{\frac{\gamma_m}{gE_s}} \qquad (4)$$

Where

T = The natural period of vibration of the dam, in seconds

H = The height of the dam, in m

B = The base width of the dam, in m

γ_m = Specific weight of the material with which the dam is constructed. For concrete dams, it may be taken as about 26.5KN/m^3

g = Acceleration due to gravity (=9.8m/s^2)

E_s = Modulus of elasticity of the dam material. For concrete dams, it may be taken as about 32.5 GPa.

Using the value obtained for the natural period of vibration (T) of the dam, and assuming the recommended value of 5 percent damping, as per IS: 1893-1984, the value of (S_a/g) may be obtained from Figure 29, and the value of the seismic coefficient computed using the appropriate equation.

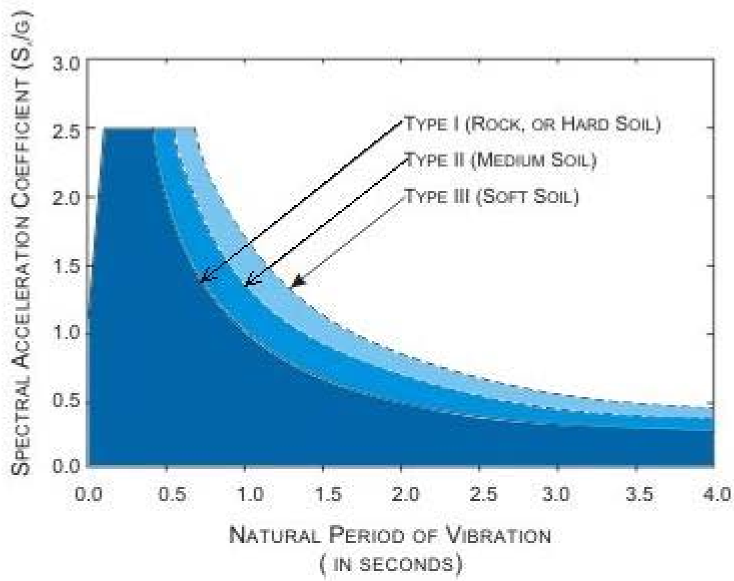


FIGURE 29. Average acceleration coefficient in terms of the natural period of vibration of the structure considering 5 percent damping.
(as per IS: 1893-2002, Part 1)

As mentioned earlier, the earthquake forces cause both the dam structure as well as the water stored in the reservoir to vibrate. The forces generated in the dam is called the Inertia Force and that in the water body, Hydrodynamic Force. Since the earthquake forces are generated due to the vibration of the earth itself, which may be shaking horizontally in the two directions as well as vibrating vertically. For design purpose, one has to consider the worst possible scenario, and hence the combination that is seen to be the least favourable to the stability of the dam has to be considered.

When the dam has been newly constructed, and the reservoir has not yet been filled, then the worst combination of vertical and horizontal inertia forces would have to be taken that cause the dam to topple backward as shown in Figure 30. The notations used in the figure are as follows:

H_u: Horizontal earthquake force acting in the upstream direction
H_D: Horizontal earthquake force acting in the downstream direction
V_u: Vertical earthquake force acting upwards
V_D: Vertical earthquake force acting downwards

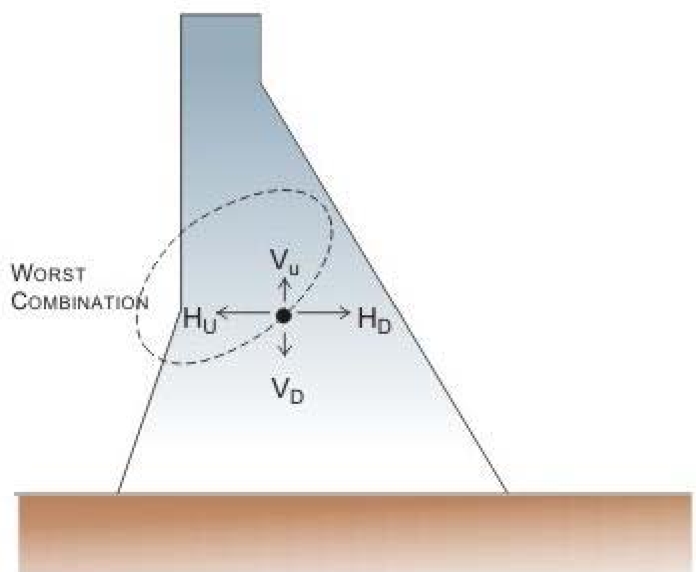


FIGURE 30. Worst combination of earthquake forces under reservoir empty condition

Under the reservoir full condition, the worst combination of the inertia forces is the one which tries to topple the dam forward, as shown in Figure 31.

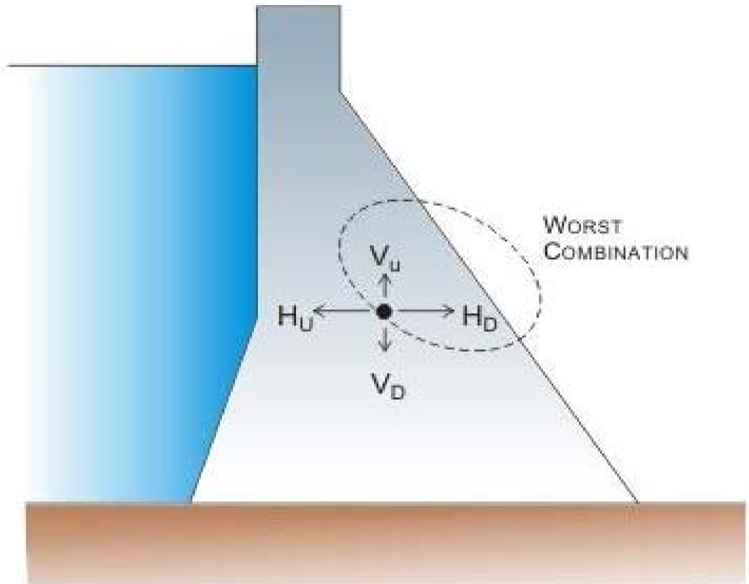


FIGURE 31. Worst combination of earthquake forces under reservoir full condition

In the Seismic Coefficient method, the horizontal and vertical acceleration coefficients, α_h and α_v, respectively, are assumed to vary linearly from base of the dam to its top as shown in Figure 32.

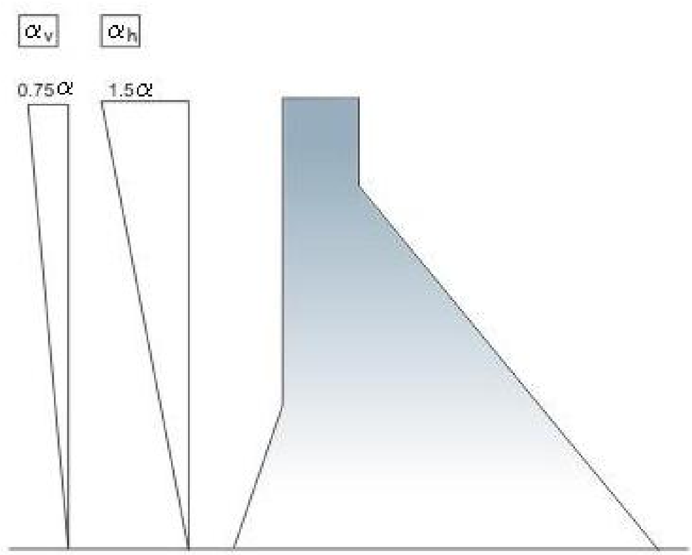


FIGURE 32. Variation of horizontal acceleration (α_h) and vertical acceleration (α_v) in terms of the Basic Seismic Coefficient α

In order to find the force generated due to the acceleration, it would be necessary to divide the dam into horizontal strips, finding out the force on each strip, and then integrating for the total dam height (Figure 33). This has to be done for both horizontal force **H** and vertical force **V**. Taking moment of these forces for each strip about any point in the dam body (say the heel or the toe) and integrating over the dam height would give the moment due to horizontal and vertical earthquake forces.

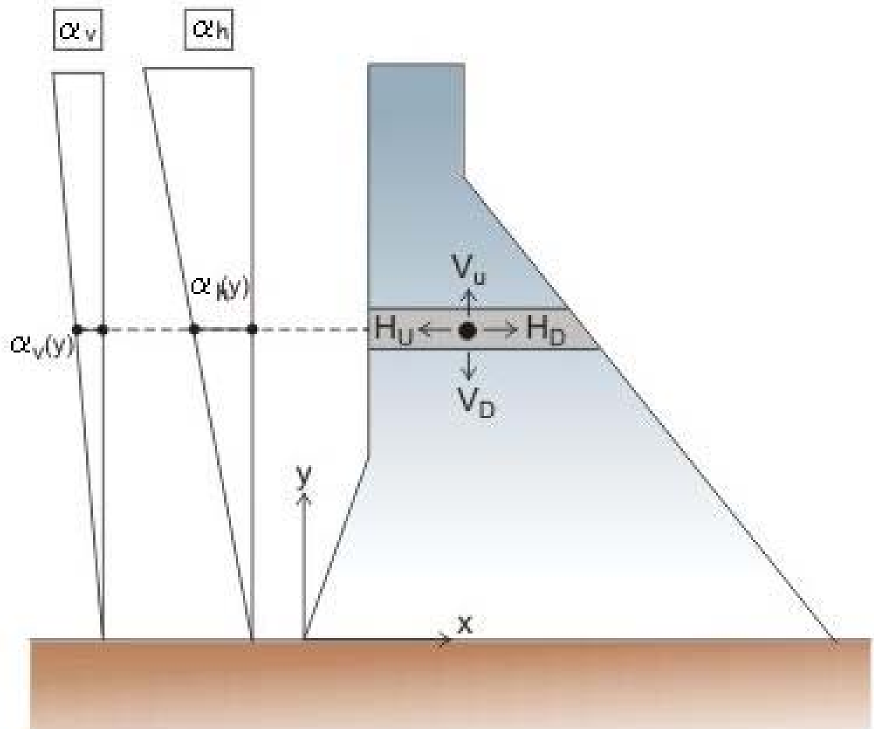


Figure 33. Earthquake acceleration forces in an infinitesimal hozontal strip in the body of the dam.

In the Response Spectrum method, the horizontal seismic coefficient α_h is assumed to be equal to the value of the seismic coefficient α obtained by the appropriate equation. The horizontal force H_B per unit length of the dam and its moment M_B about any point in the base of the dam is obtained by the following expressions:

$$\mathrm{H_B} = 0.6W\alpha_h \tag{5}$$

$$M_B = 0.9W\bar{h}\alpha_h \tag{6}$$

Where

W = Weight of the dam per unit length in KN/m

α_h = Seismic coefficient as obtained by the appropriate equation, and

$\bar{h}$ = Height of the centre of gravity of the dam above the base, in m.

For any horizontal section within the dam body, lying at a depth **y** from the top of the dam, the horizontal force $\mathbf{H_y}$ per unit length of the dam and the bending moment $\mathbf{M_y}$ may be obtained as follows:

$$\mathrm{H}_y = C_\mathrm{H} \cdot \mathrm{H}_B \tag{7}$$

$$\mathrm{M}_y = C_\mathrm{M} \cdot \mathrm{M}_B \tag{8}$$

Where $\mathbf{C_H}$ and $\mathbf{C_M}$ are coefficients that may be read out from Figure 34.

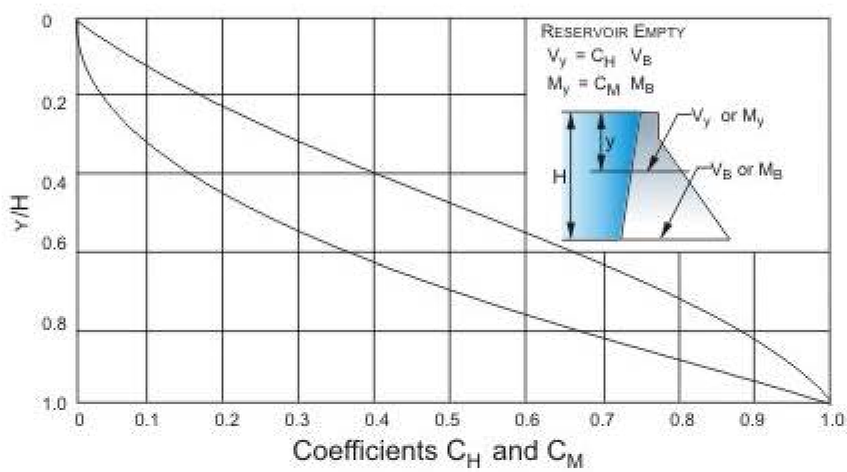


FIGURE.34: Variation of coefficients C_H and C_M

As for the vertical earthquake force calculation by the Response Spectrum method, the vertical seismic coefficient $\alpha_\mathbf{u}$ has to be assumed to vary from zero at base up to $\mathbf{0.75\alpha}$ at the top, where α is the seismic coefficient calculated appropriately. The method for calculating the vertical force and its corresponding moment has to proceed as for the seismic coefficient method.

The hydrodynamic pressure generated due to the horizontal movement of the water body in the reservoir and its consequent impinging against the dam may to be calculated by the following formula.

$$\mathrm{P} = C_s \cdot \alpha \cdot \gamma \cdot h \tag{5}$$

Where

P = Hydrodynamic pressure, in KN/m^2 at any depth y below the reservoir surface

C_s = Coefficient which varies with the shape of the dam and the depth of the reservoir, which may be found by the method indicated below

. γ = Unit weight of water, in KN/m^3

h = Total water depth in reservoir, in m

The variation of the coefficient C_s may approximately be found for a dam with vertical or constant upstream slope as:

$$C_s = \frac{C_m}{2}\left\{\frac{y}{n}\left(2-\frac{y}{n}\right)+\sqrt{\frac{y}{n}\left(2-\frac{y}{n}\right)}\right\} \qquad (6)$$

Where

C_m = Maximum value of C_s, obtained from Figure 35

y = Depth of horizontal section under consideration below the water surface, in m

h = Total depth of water in reservoir

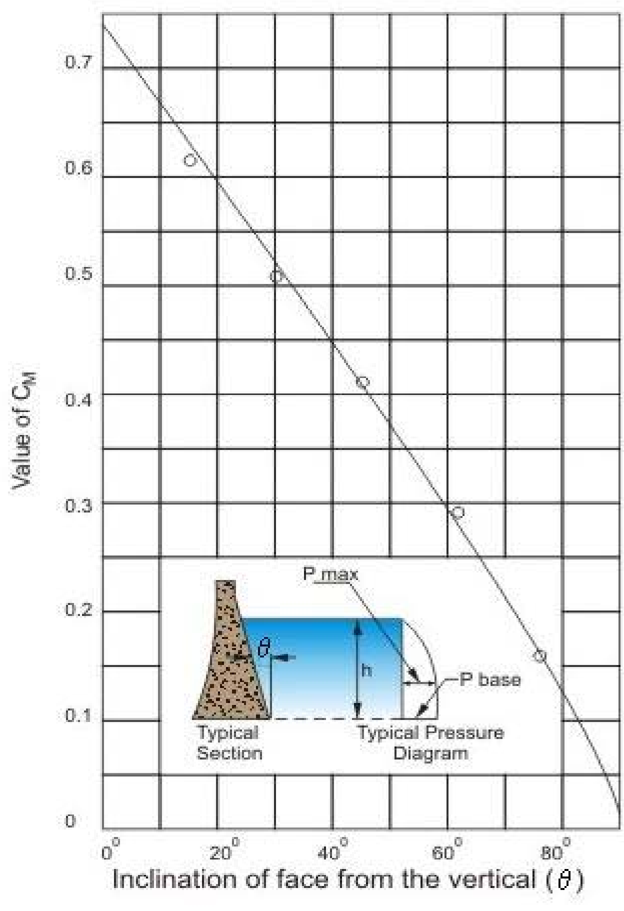


FIGURE.35: Maximum values of pressure coefficient (C_M) for dams with constant sloping faces

For dams with combination of vertical and sloping faces, an equivalent slope may be used for obtaining the approximate value of $\mathbf{C_s}$. If the height of the vertical portion of the upstream face of the dam is equal to or greater than one-half the total height of the dam, analyze it as if vertical throughout. If the height of the vertical portion of the upstream face of the dam is less than one-half the total height of the dam, use the pressure on the sloping line connecting the point of intersection of the upstream face of the dam and the reservoir surface with the point of intersection of the upstream face of the dam with the foundation.

The approximate values of horizontal pressure $\mathbf{PH_y}$ and moment due to the horizontal force $\mathbf{MH_y}$ due to hydrodynamic forces at any depth **y** (in m) below the reservoir surface is given by the following formulae.

$$\mathrm{PH}_y = 0.726 \cdot p \cdot y \quad \text{(in KN per m length)} \tag{7}$$

$$\mathrm{MH}_y = 0.299 \cdot p \cdot y \quad \text{(in KN.m per m length)} \tag{8}$$

Where **p** is the hydrodynamic pressure at any depth **y**.
If there is tail water on the downstream, then there would be appropriate hydrodynamic pressure on the downstream face of the dam.

Wave pressure

The reservoir behind a dam is prone to generation of waves produced by the shearing action of wind blowing over the surface. Of course, the pressure of the waves against massive dams of appreciable height is not of much consequence. The height of wave is generally more important in determination of the free board requirements of dams to prevent overtopping of the dam crest by wave splash. The force and dimensions of waves depend mainly on the extent and dimensions of waves depend mainly on the extent and configuration of the surface area of the reservoir, the depth of the reservoir, and the velocity of the wind. The procedure to workout the height of waves generated, and consequently derive the safe free board, may be done according to the method described in IS: 6512-1984 "Criteria for design of solid gravity dams". However, since it is a bit involved, a simpler method is prescribed as that given by the Stevenson formula (Davis and Sorenson 1969).

$$H_w = 0.34\sqrt{F} + 0.76 - 0.26\sqrt[4]{F} \tag{9}$$

Where

H_w = Height of wave, crest to trough, in m

F = Fetch of the reservoir, that is, the longest straight distance of the reservoir from the dam up to the farthest point of the reservoir.

When the fetch exceeds 20Km, the above formula can be approximated as

$$H_w = 0.34\sqrt{F} \quad (10)$$

Since the height of the generated waves must be related to the wind velocity, the original formula has been modified to

$$H_w = 0.032\sqrt{(VF)} + 0.76 - 0.26\sqrt[4]{F} \quad (11)$$

Where V = wind speed along the fetch, in km/h

Stevenson's approximate formula is applicable for wind speeds of about 100km/hour, which is a reasonable figure for many locations. It is conservative for low wind speeds but under estimates waves for high wind speeds.

The pressure intensity due to waves (**P_w**, in KN/m^2) is given by the following expression

$$P_w = 23.544H_w \quad (12)$$

Where **H_w** is the height of wave in m. and occurs at 1/8H_w above the still water level (Figure36).

The total wave pressure **P_w** per unit length (in KN/m) of the dam is given by the area of the triangle 1-2-3 as given in Figure 36, and is given as

$$P_w = 20H_w^2 \quad (13)$$

The centre of application is at a height of 0.375**H_w** above the still water level.

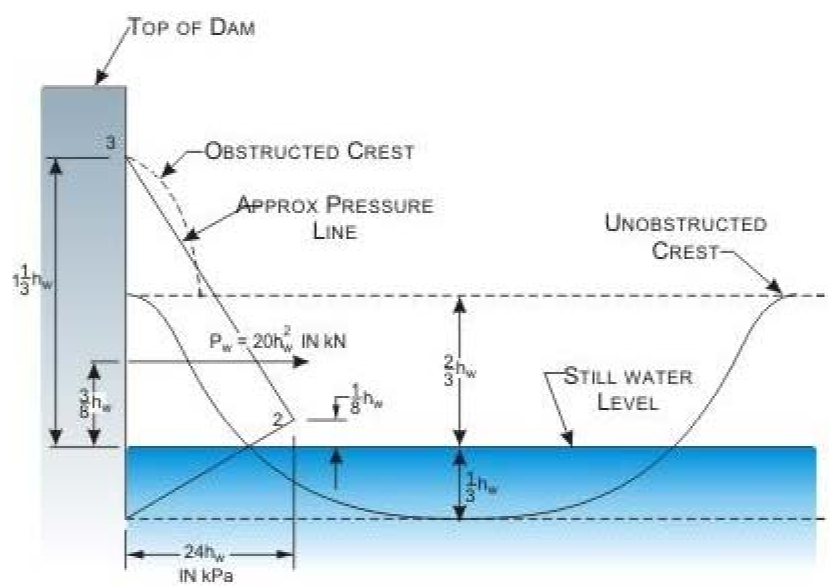


FIGURE. 36: Wave height, pressure and center of action

4.6.4 Free board

Free board is the vertical distance between the top of the dam and the sill water level. IS:6512-1984 recommends that the free board shall be wind set-up plus 4/3 times wave height above normal pool elevation or above maximum reservoir level corresponding to design flood, whichever gives higher crest elevation. Wind set-up is the shear displacement of water towards one end of a reservoir by wind blowing continuously – or in repeated regular gusts – from one direction. The Zuider Zee formula (Thomas, 1976) and recommended by IS: 6512-1984 may be used as a guide for the estimation of set-up(S):

$$S = \frac{V^2 F Cos A}{kD} \tag{14}$$

Where

- S = Wind set-up, in m
- V = Velocity of wind over water in m/s
- F = Fetch, in km
- D = Average depth of reservoir, in m, along maximum fetch
- A = Angle of wind to fetch, may be taken as zero degrees for maximum set-up
- K = A constant, specified as about 62000

Set-up of the reservoir will depend upon the period of time over which the wind blows, that is, at least 1hour, for a fetch of 3km or 3hours for a fetch of 20km. On a 80km fetch, a wind speed of 80 km/hour must last for at least 4hours, whereas for a wind speed of 40km/hour it must last around 8hours for maximum set-up.

The free-board shall not be less than 1.0m above Maximum Water Level (MWL) corresponding to the design flood. If design flood is not same as Probable Maximum Flood (PMF), then the top of the dam shall not be lower than MWL corresponding to PMF.

4.6.5 Stability analysis of gravity dams

The stability analysis of gravity dams may be carried out by various methods, of which the gravity method is described here. In this method, the dam is considered to be made up of a number of vertical cantilevers which act independently for each other. The resultant of all horizontal and vertical forces including uplift should be balanced by an equal and opposite reaction at the foundation consisting of the total vertical reaction and the total horizontal shear and friction at the base and the resisting shear and friction of the passive wedge, if any. For the dam to be in static equilibrium, the location of this force is such that the summation of moments is equal to zero. The distribution of the vertical reaction is assumed as trapezoidal for convenience only. Otherwise, the problem of determining the actual stress distribution at the base of a dam is complicated by the horizontal reaction, internal stress relations, and other theoretical considerations. Moreover, variation of foundation materials with depth, cracks and fissures which affect the resistance of the foundation also make the problem more complex. The internal stresses and foundation pressures should be computed both with and without uplift to determine the worst condition.

The stability analysis of a dam section is carried out to check the safety with regard to

1. Rotation and overturning
2. Translation and sliding
3. Overstress and material failure

Stability against overturning

Before a gravity dam can overturn physically, there may be other types of failures, such as cracking of the upstream material due to tension, increase in uplift, crushing of the toe material and sliding. However, the check against overturning is made to be sure that the total stabilizing moments weigh out the de-stabilizing moments. The factor of safety against overturning may be taken as 1.5. As such, a gravity dam is considered safe also from the point of view of overturning if there is no tension on the upstream face.

Stability against sliding

Many of the loads on the dam act horizontally, like water pressure, horizontal earthquake forces, etc. These forces have to be resisted by frictional or shearing forces along horizontal or nearly-horizontal seams in foundation. The stability of a dam against sliding is evaluated by comparing the minimum total available resistance along the critical path of sliding (that is, along that plane or combination of plans which mobilizes the least resistance to sliding) to the total magnitude of the forces tending to induce sliding.

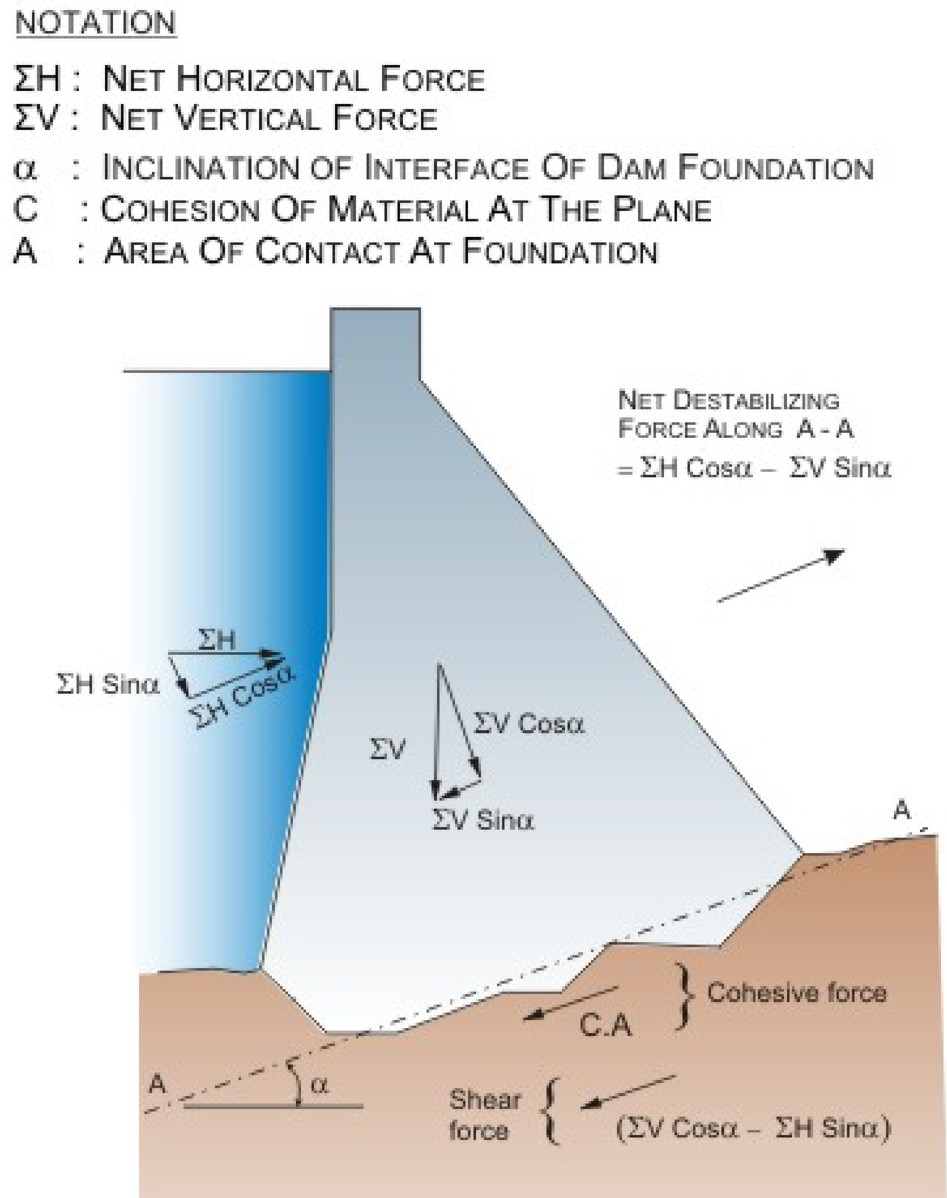


FIGURE. 37: Stability against sliding along concrete dam - rock base interface. Good rock is assumed to exist below

Sliding resistance is also a function of the cohesion inherent in the materials at their contact and the angle of internal friction of the material at the surface of sliding. The junction plane between the dam and rock is rarely smooth. In fact, special efforts are made during construction to keep the interface as rough as possible. There may, however be some lower plane in the foundation where sliding is resisted by friction alone especially if the rock is markedly stratified and horizontally bedded. Figure 37 shows a typical dam profile with the bed-rock and foundation interface inclined at an angle **α**. Factor of Safety against sliding (F) along a plane may be computed from the following formula:

$$F = \frac{\dfrac{\text{Net shear force along the plane}}{F_\varphi} + \dfrac{\text{Net cohesive force along the plane}}{F_c}}{\text{Net horizontal destabilizing force along the plane}} \qquad (15)$$

Where F_ϕ and F_c are the Partial Factor of Safety in respect of friction and Partial Factor of Safety of cohesion. IS: 6512-1984 recommends these values to be as given in the following table:

Loading Condition	F_ϕ	F_c		
		For dams and the contact plane with foundation	For foundation	
			Thoroughly investigated	Others
A,B,C	1.5	3.6	4.0	4.5
D,E	1.2	2.4	2.7	3.0
F,G	1.0	1.2	1.35	1.5

The value of cohesion and internal friction may be estimated for the purpose of preliminary designs on the basis of available data on similar or comparable materials. For final designs, however, the value of cohesion and internal friction has to be determined by actual laboratory and field tests, as specified in the Bureau of Indian Standards code IS: 7746-1975 “Code of practice for in-situ shear test on rocks”.

In the presence of a horizon with low shear resistance, for example, a thin clay seam or clay infill in a discontinuity (Figure 38), then it would be advisable to include downstream passive wedge resistance P, as a further component of the total resistance to sliding which can be mobilized. In this case, the Factor of Safety along sliding has to be found along plane B-B computing the net shear force and net cohesive force along this plane. The net shear force would now be equal to:

$$\left(W Cos\alpha + \sum \text{H} Sin\alpha\right)\tan\phi \qquad (16)$$

Where W is the weight of the wedge; $\alpha$is the assumed angle of sliding failure, $\sum$Ḥ is the net destabilizing horizontal moment; and ϕ is the internal friction within the rock at plane

B-B. The net cohesive force along plane B-B is determined as equal to C.$A_{B-B.}$ Here, C is the cohesion of material and A_{B-B}, the area, along plane B-B.

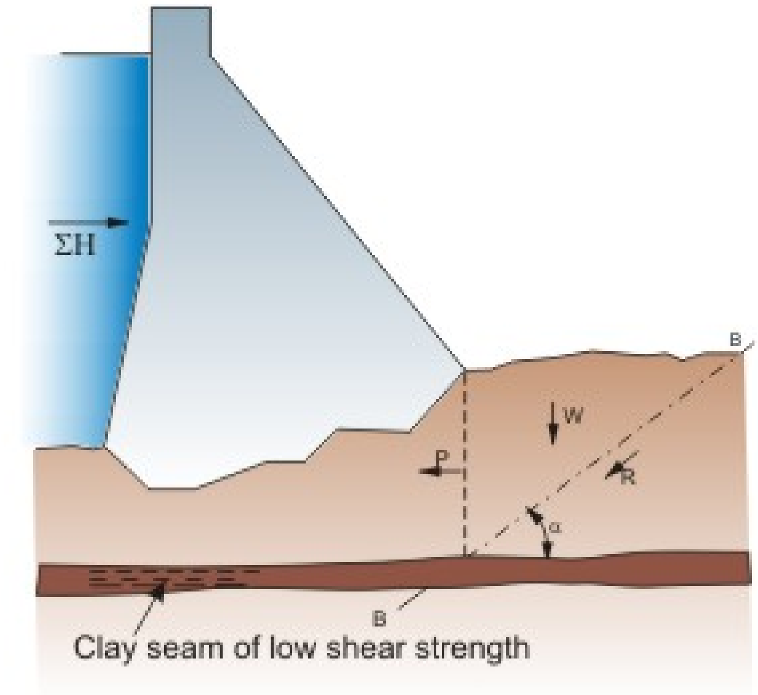


FIGURE.38: Sliding against presence of weak seams resisted by passive wedge resistance

Failure against overstressing

A dam may fail if any of its part is overstressed and hence the stresses in any part of the dam must not exceed the allowable working stress of concrete. In order to ensure the safety of a concrete gravity dam against this sort of failure, the strength of concrete shall be such that it is more than the stresses anticipated in the structure by a safe margin. The maximum compressive stresses occur at heel (mostly during reservoir empty condition) or at toe (at reservoir full condition) and on planes normal to the face of the dam. The strength of concrete and masonary varies with age, the kind of cement and other ingredients and their proportions in the work can be determined only by experiment.

The calculation of the stresses in the body of a gravity dam follows from the basics of elastic theory, which is applied in a two-dimensional vertical plane, and assuming the block of the dam to be a cantilever in the vertical plane attached to the foundation. Although in such an analysis, it is assumed that the vertical stresses on horizontal planes vary uniformly and horizontal shear stresses vary parabolically, they are not strictly correct. Stress concentrations develop near heel and toe, and modest tensile stresses may develop at heel. The basic stresses that are required to be determined in a gravity dam analysis are discussed below:

Normal stresses on horizontal planes

On any horizontal plane, the vertical normal stress (σ_Z) may be determined as:

$$\sigma_Z = \frac{\sum V}{T} \pm \frac{12 \sum Ve}{T^3} y \tag{17}$$

Where

$\sum V$ = Resultant vertical load above the plane considered

T = Thickness of the dam block, that is, the length measured from heel to toe

e = Eccentricity of the resultant load

y = Distance from the neutral axis of the plane to the point where (□□) is being determined

At the heel, y = -T/2 and at toe, y = +T/2. Thus, at these points, the normal stresses are found out as under:

$$\sigma_{z_{heel}} = \frac{\sum V}{T}\left(1 - \frac{6e}{T}\right) \tag{18}$$

$$\sigma_{z_{toe}} = \frac{\sum V}{T}\left(1 + \frac{6e}{T}\right) \tag{19}$$

The eccentricity e may be found out as:

$$e = \frac{Net\ moment}{Net\ vertical\ \ force} \tag{20}$$

Naturally, there would be tension on the upstream face if the overturning moments under the reservoir full condition increase such that e becomes greater than T/6. The total vertical stresses at the upstream and downstream faces are obtained by addition of external hydrostatic pressures.

Shear stresses on horizontal planes

Nearly equal and complimentary horizontal stress (τ_{zy}) and shear stresses (τ_{yz}) are developed at any point as a result of the variation in vertical normal stress over a horizontal plane (Figure 39). The following relation can be derived relating the stresses with the distance y measured from the centroid:

$$\tau_{zy} = \tau_{yz} = \tau_{yzD} - \frac{2}{T}\left[\frac{3H}{T} + \tau_{yzU} + 2\tau_{yzD}\right]y + \frac{3}{T^2}\left[\frac{2H}{T}\tau_{yzD} + \tau_{yzU}\right]y^2 \qquad (21)$$

Where

$\tau_{yz\,D} = (\sigma_{zD} - p_D)\tan\phi_D$, the shear stress at downstream face
$\tau_{yz\,U} = -(\sigma_{zU} - p_U)\tan\phi_D$, the shear stress at upstream face
H = the height of the dam

The shear stress is seen to vary parabolically from $\tau_{yz\,U}$ at the upstream face up to $\tau_{yz\,D}$ at the downstream face.

Normal stresses on vertical planes

These stresses, σ_y can be determined by consideration of the equilibrium of the horizontal shear forces operating above and below a hypothetical element within the dam (Figure 39). The difference in shear forces is balanced by the normal stresses on vertical planes. Boundary values of σ_y at upstream and downstream faces are given by the following relations:

$$\sigma_{y_U} = p_u + \left(\sigma_{z_U} - p_y\right)\tan^2\phi_U \qquad (22)$$

$$\sigma_{y_D} = p_y + \left(\sigma_{z_D} - p_y\right)\tan^2\phi_D \qquad (23)$$

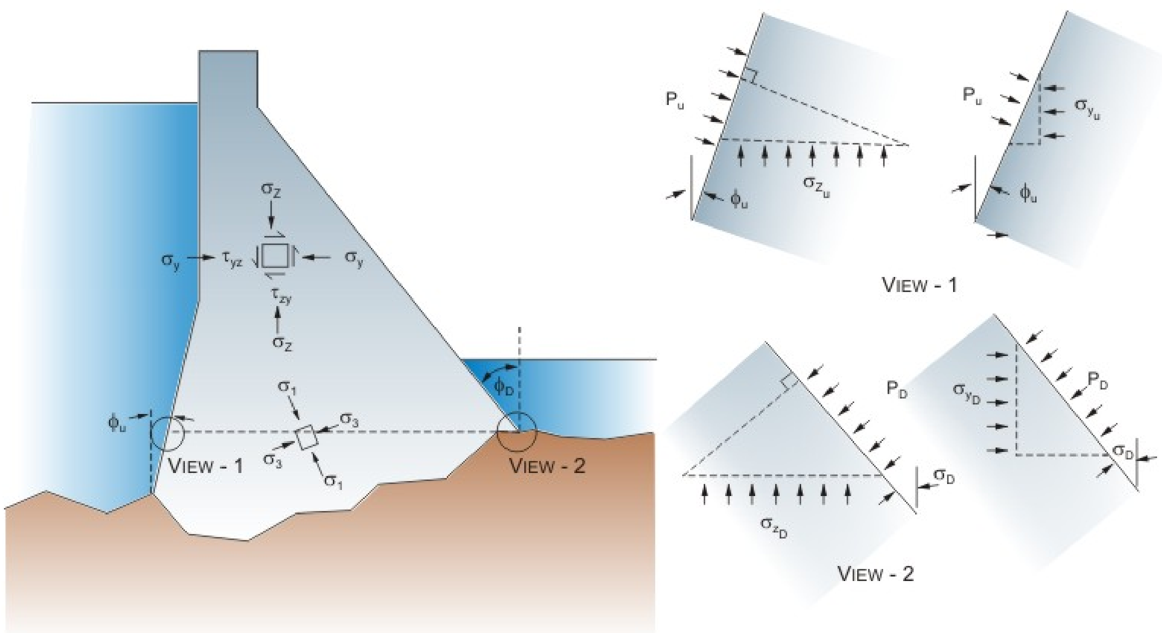


FIGURE.39: State of stress in a concrete gravity dam

Principal stresses

These are the maximum and minimum stresses that may be developed at any point within the dam. Usually, these are denoted as σ_1 and σ_3 respectively, and are oriented at a certain angle to the reference horizontal or vertical lines. The magnitude of σ_1 and σ_3 may be determined from the state of stress σ_z, σ_y and τ_{yz} at any point by the following formula:

$$\sigma_{1,3} = \frac{\sigma_z + \sigma_y}{2} \pm \sqrt{\left(\frac{\sigma_z - \sigma_y}{2}\right)^2 + \tau_{zy}^2} \tag{24}$$

The maximum and minimum shear stress is obtained from the following formula:

$$\tau_{\max} = \sqrt{\left(\frac{\sigma_z - \sigma_y}{2}\right)^2 + \tau_{yz}^2} \tag{25}$$

The upstream and downstream faces are each planes of zero shear, and therefore, are planes of principal stresses. The principal stresses at these faces are given by the following expressions:

$$\sigma_{1_u} = \sigma_{z_u} \sec^2 \phi_u - p_u \tan^2 \phi_u \tag{26}$$

$$\sigma_{3_u} = p_u \tag{27}$$

$$\sigma_{1_D} = \sigma_{z_D} Sec^2 \phi_D - p_D \tan^2 \phi_D \tag{28}$$

$$\sigma_{3_D} = p_D \tag{29}$$

Permissible stresses in concrete

According to IS: 6512-1984, the following have to be followed for allowable compressive and tensile stresses in concrete:

Compressive strength of concrete is determined by testing 150mm cubes. The strength of concrete should satisfy early load and construction requirements and at the age of one year, it should be four times the maximum computed stress in the dam or 14N/mm^2, whichever is more. The allowable working stress in any part of the structure shall also not exceed 7N/mm^2.

No tensile stress is permitted on the upstream face of the dam for load combination B. Nominal tensile stresses are permitted for other load combinations and their permissible values should not exceed the values given in the following table:

Load combination	Permissible tensile stress
C	$0.01f_c$
E	$0.02f_c$
F	$0.02f_c$
G	$0.04f_c$

Where $\mathbf{f_c}$ is the cube compressive strength of concrete.

Small values of tension on the downstream face is permitted since it is improbable that a fully constructed dam is kept empty and downstream cracks which are not extensive and for limited depths from the surface may not be detrimental to the safety of the structure.

4.6.6 Construction of concrete gravity dam

River diversion

Regardless of the type of dam, whether concrete or embankment types, it is necessary to de-water the site for final geological inspection, for foundation improvement and for the construction of the first stage of the dam. In order to carry out the above works the river has to be diverted temporarily. The magnitude, method and cost of river diversion will depend upon the cross- section of the valley, the bed material in the river, the type of dam, the expected hydrological conditions during the time required to complete the dam construction works, and finally upon the consequences of failure of any part of the temporary works.

For concrete dams, it may be necessary to divert the river during the first phase of the construction of the dam. Once this is complete, the river may be allowed to overtop the dam and flow without causing serious damages to the structure or its foundation. For concrete dams, sluice openings are left open in the first stage of concreting and the higher stages constructed. If the second stage outlets are too small for the flood to pass, they would be submerged after the whole works. At some sites, virtually no risk can be afforded. For the Ukai dam on river, Tapi, which is 4927 m long and maximum height of 68.6 m, no risk of overtopping and possible destruction of the control section could be accepted in view of the very large resident population downstream of the dam. Hence, a diversion channel was excavated to carry 49500 m^3/s passed through the blocks of the concrete gravity dam section that was intentionally left low.

The Bureau of Indian standards code IS 10084 (part 2) -1994 “Design of diversion works – criteria” describes the design criteria for diversion channel and open cut or conduit in the body of the dam.

At sites where diversion of flow through tunnels or close conduits is not possible (due to topographical considerations) or proves to be uneconomical, diversion through excavated channels called diversion channels is effected. Diversion channels are often classified according to the type of diversion namely, single stage or multiple stage diversion scheme. In the former which is more suitable for narrow valleys, the same set of diversion channel and coffer dams is utilised throughout the period of construction. In the latter, which is generally suitable for wide valleys, the channels and coffer dams are shifted from place to place in accordance with phasing of the work. A more useful classification, however, is based on the type of the dam to be constructed namely diversion channel for masonry or concrete dams and that for the earth or rockfill dams. The following paragraphs taken from Bureau of Indian Standards code IS: 10084 (Part 2) – 1994 "Design of diversion works – criteria" provides criteria for diversion channels for dam construction.

Diversion Channels

A concrete or masonry dams could be allowed to get overtopped during floods when construction activity is not in progress. The resulting damage is either negligible or could be tolerated without much concern. Therefore, it is customary to adopt diversion flood which is just adequate to be handled during non monsoon season, when construction activity of the dam is continued. Generally the largest observed non-monsoon flood or non-monsoon flood of 100 year return period is adopted as a diversion flood. This is generally a small fraction of the design flood of the spillway and, therefore, diversion channel required to handle this flood is obviously small. Advantage is also taken of passing the floods over partly completed dam or spillway blocks, thereby keeping the diversion channel of relatively smaller size. In such a case a small excavated channel either in the available width of the river or one of the banks of the river proves to be adequate. Construction sluices are located in such excavated channels which allow passage of non-monsoon flows without hindrance to the construction activity. Such sluices are subsequently plugged when the dam has been raised to adequate height. If the pondage is not allowed even when the dam has been raised to sufficient height, the river outlets are often provided in the body of the non overflow or overflow dam to pass the non monsoon flows which later on are kept for permanent use after completion of construction. If the diversion channel is excavated on one of the river banks, it is possible to use the same for locating an irrigation outlet, a power house or a spillway depending upon the magnitude and purpose of the project. Figures 40 and 41 show typical layouts of diversion channel for masonry/concrete dams in wide and narrow rivers respectively.

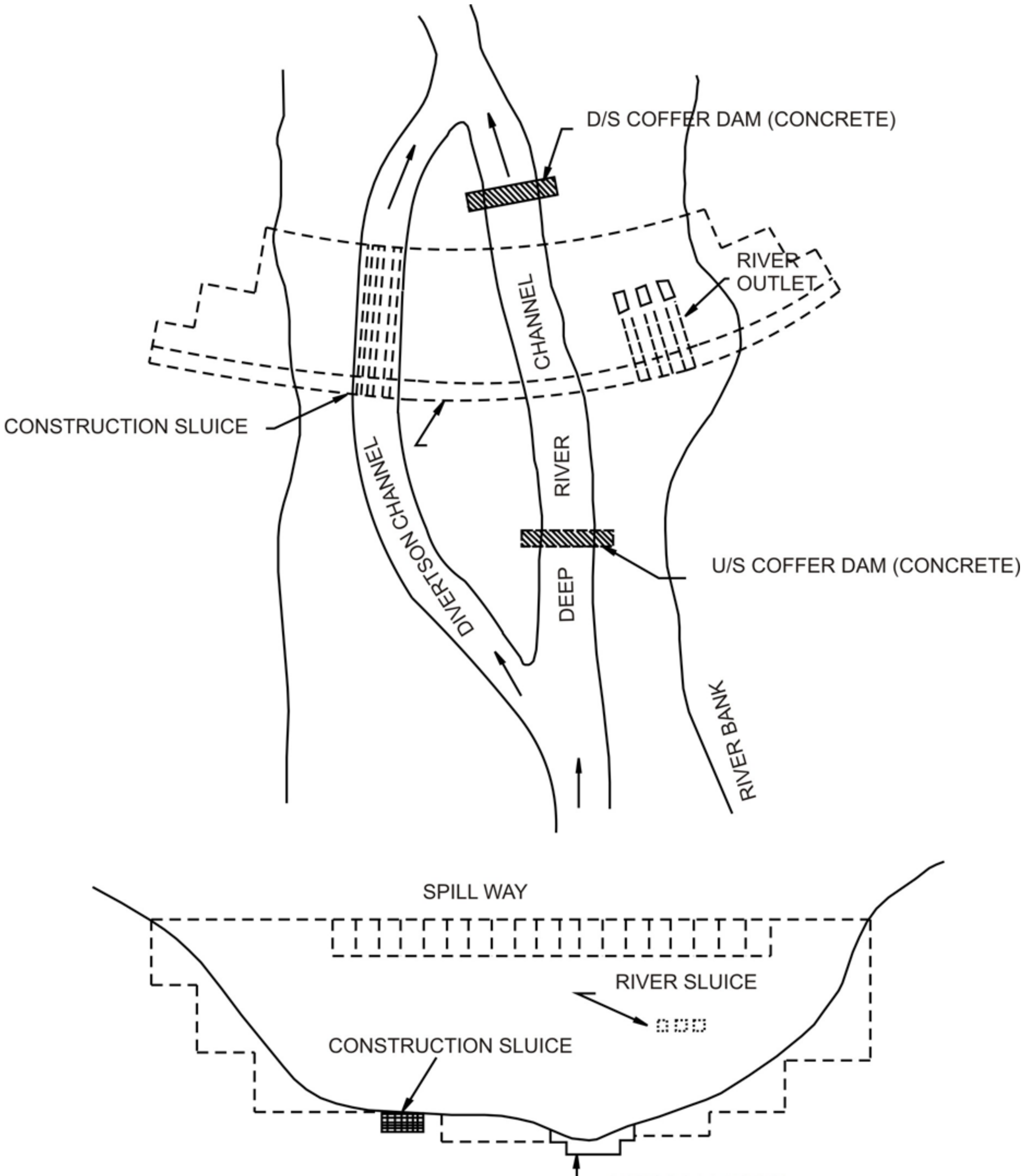


FIGURE 40. DIVERSION CHANNEL FOR CONCRETE DAM IN A WIDE RIVER

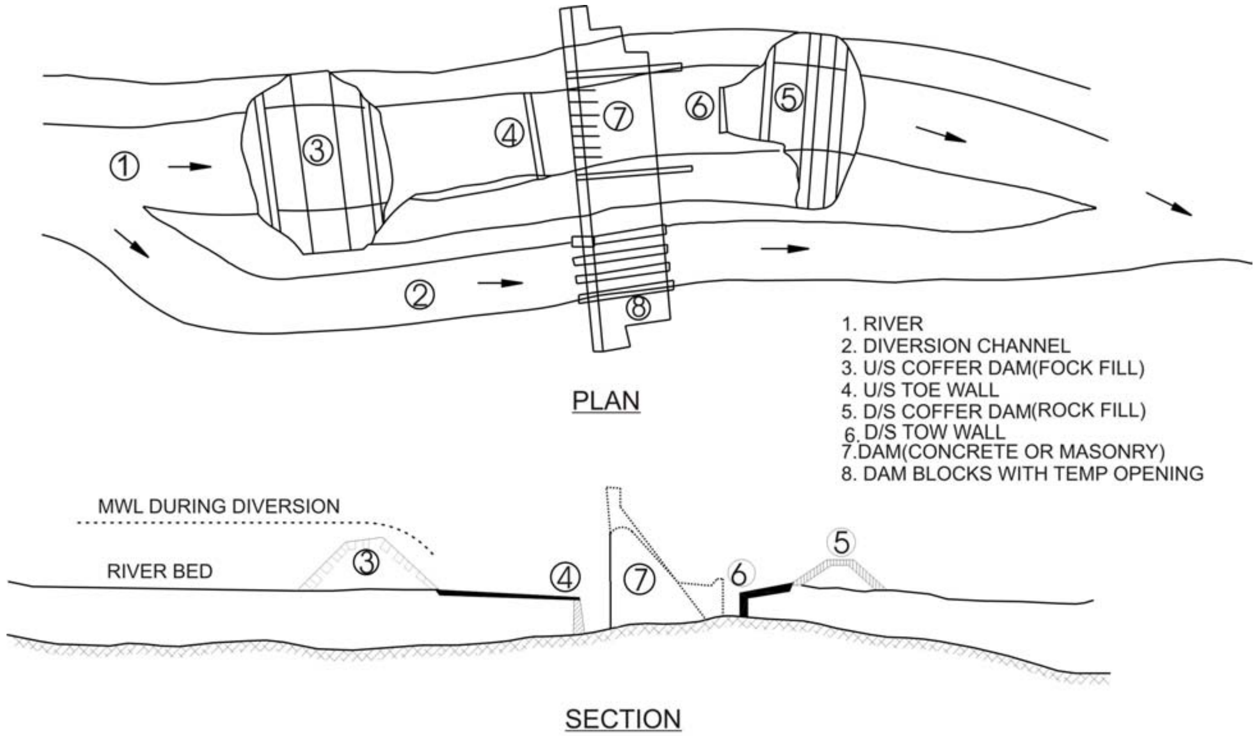


FIGURE 41.DIVERSION CHANNEL FOR CONCRETE DAM IN A NARROW RIVER

4.6.7 Preparation of foundation for dam construction

A concrete gravity dam intended to be constructed across a river valley would usually be laid on the hard rock foundation below the normal river overburden which consists of sand, loose rocks and boulders. however at any foundation level the hard rock foundation, again, may not always be completely satisfactory all along the proposed foundation and abutment area, since locally there may be cracks and joints, some of these (called seams) being filed with poor quality crushed rock. Hence before the concreting takes place the entire foundation area is checked and in most cases strengthened artificially such that it is able to sustain the loads that would be imposed by the dam and the reservoir water, and the effect of water seeping into the foundations under pressure from the reservoir.

Generally the quality of foundations for a gravity dam will improve with depth of excavation. Frequently the course of the river has been determined by geological faults or weaknesses. In a foundation of igneous rock, any fault or seam should be cleaned out and backfilled with concrete. A plug of concrete of depth twice the width of the seam would usually be adequate for structural support of the dam, so that depth of excavation will, on most occasions depend upon the nature of infilling material, the shape of the excavated zone and the depth of cutoff necessary to ensure a acceptable hydraulic gradient after the reservoir is filled. An example of this type of treatment for Bhakra dam is shown in Figure 42.

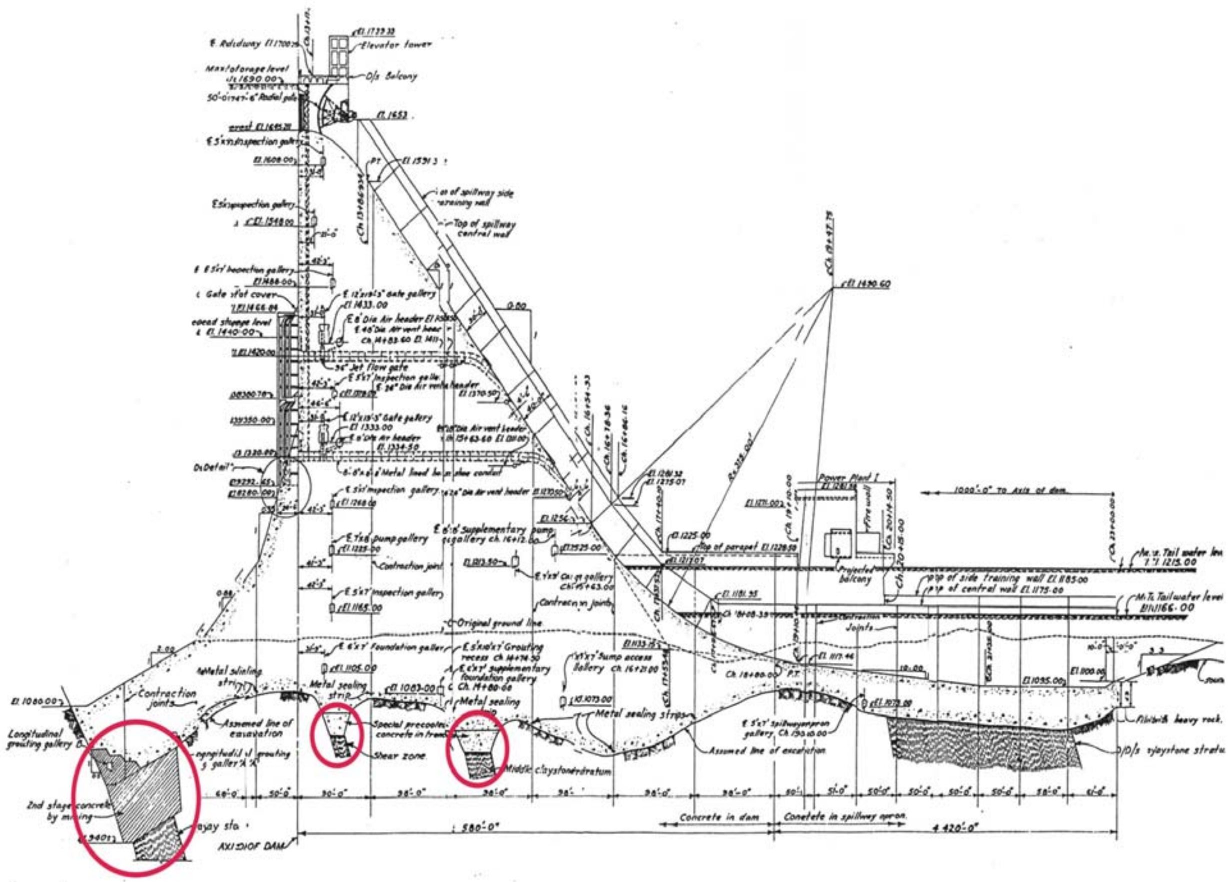


FIGURE 42. FOUNDATION FEATURE FOR BHAKRA DAM

Improvement of the foundation for a dam may be effected by the following major ways:

1. Excavation of seams of decayed or weak rock by tunneling and backfilling with concrete.
2. Excavation of weak rock zones by mining methods from shafts sunk to the zone and backfilling the entire excavated region with concrete.
3. Excavation for and making a subterranean concrete cutoff walls across leakage channels in the dam foundation where the where the water channels are too large or too wet for mining or grouting
4. Grouting the foundation to increase its strength and to render it impervious.

Grouting of the foundation of the dam to consolidate the entire foundation rock and consequently increasing its bearing strength is done by a method that is referred to as consolidation grouting. This is a low pressure grouting for which shallow holes are drilled through the foundation rock in a grid pattern. These holes are drilled to a depth ranging from 3 to 6 m. Prior to the commencement of the grouting operation, the holes are thoroughly washed with alternate use of water and compressed air to remove all loose material and drill cuttings. The grout hole are then tested with water under pressure to obtain an idea of the tightness of the hole which is necessary to decide the consistency of the grout to be used and to locate the seams or other openings in the rock which are to be plugged . The grout is then injected with these holes at relatively low pressure which is usually less than about 390 KN/m². Since this is a low pressure grouting it is accomplished before any concrete for the dam is laid. This grouting results in the consolidation of the foundation into more or less monolithic rock by bonding together the jointed or shattered rocks. Some of the recommendations for grouting under pressure in rock foundations have been taken from Bureau of Indian Standards code IS: 6066 – 1994 “Pressure grouting of rock foundations in river valley projects – recommendations” have been presented in the following paragraphs.

Methods of rock grouting

Rock grouting consists essentially of drilling a series of grout holes in rock and injecting grout under pressure, which eventually sets in the openings and voids in the rock. The drilling and grouting operations can be carried out either to the full depth in one operation or in successive depths either by stage grouting or packet grouting. Grouting in the valley should proceed from river bed to the abutments. There are two broad methods for grouting: Full depth grouting and stage grouting.

In the full depth method each hole is drilled to the full desired depth, washed, pressure tested and grouted in one operation. This method is usually limited to short holes 5m or less in depth or holes up to 10 m that have only small cracks and joints with no risk of surface leakage. In deep bore holes high grouting pressures have to be used for proper penetration of grout at an economical spacing of holes. As full depth grouting involves the risk of disturbance in the upper elevation it is not generally considered for grouting deep holes. For grouting in heterogeneous strata, where the nature of rock discontinuities is subject to large variations in relation to the depth, full depth grouting is not recommended and stage grouting is preferred to packer grouting in such cases.

Stage grouting is done by drilling the holes to a predetermined depth and grouting this initial depth at an appropriate pressure to its final set (within 2 to 4 hours) deepened for the next stage. Alternatively the grout is allowed to harden and re drilling is carried out through the hardened grout and the hole extension to the next stage. In another procedure called the one stage re drilled method, which is sometimes used grout is washed out within a small depth of the top of the stage being grouted and only one stage is re drilled for proceeding to the next stage. In each of procedures the cycle of grouting-drilling-washing-re drilling is repeated until the required depth of the hole is reached.

General criteria for size and depth of grout holes

The pattern and depth of holes is governed primarily by the design requirements and the nature of the rock. When the purpose is consolidation, the holes are arranged in a regular pattern over the entire surface area required to be strengthened and the depth is determined by the extent of broken rock as well as the structural requirements regarding the deformability and strength of the foundation. When the purpose is impermeablisation the grout holes are arranged in a series of lines to form a curtain approximately perpendicular to the direction of the seepage. The depth of holes is dependent on design consideration as also on the depth of pervious rock and configuration of zones of relatively impervious strata.

The size of grout holes is generally less important than the cost of drilling holes and the control of the inclination. For grouting with cement, 38 mm holes are used. The advantage gained by drilling large holes does not often justify the increase in drilling costs. In long holes the diameter at the top of the hole may have to be larger than the final diameter at the bottom of the hole to facilitate telescoping or allow for the wear of the bit.

Patterns of holes for curtain grouting

Single lines grout curtains are effective only in rocks having a fairly regular network of discontinuities with reasonably uniform size of openings. In such cases a curtain of adequate width can be achieved by grouting a single line of holes. In massive rocks, with fine fissures uplift control is primarily obtained by drainage and the grout curtain is only used as a supplementary measure to avoid concentration of seepage which may exceed the capacity of the drainage system. Single line curtain may serve this limited objective in comparatively tight rock formations.

In single line curtains (Figure 43), it is customary to drill a widely spaced system of primary holes, subsequentially followed by secondary and tertiary holes at a progressively small spacing. The usual practice is to split the spacing from primary to the secondary to the tertiary phase. One of the criteria for deciding on the primary spacing is the length of expected intercommunication of grout between holes. The initial spacing usually varies from 6m to 12 m but the choice of spacing should be based on geological conditions and on experience. At every phase of the grouting operation, the results of percolation tests and ground absorption data should be compared with the previous set of holes in order to decide whether a further splitting of the spacing of holes in worthwhile. When no significant improvement is noticed either in terms of decrease of the grout absorption or water percolation, careful review should be made of rock features, the nature of the rock and its relation to the pattern of holes. Sometimes it may be more advantageous to drill another line of holes at a different angle and orientation than to split the spacing further. Spacing below 1 meter are rarely necessary and the requirement of a spacing closer than 1 meter often indicates an unsuitable orientation and inclination of holes. Possibly multiple line curtains may be necessary. If the area is too limited, the setting time of the grout becomes important since it is not desirable to drill close to a freshly grouted hole. Before pressure grouting is started, drilling of all the holes should be completed within a distance of 20 m of the hole to be grouted.

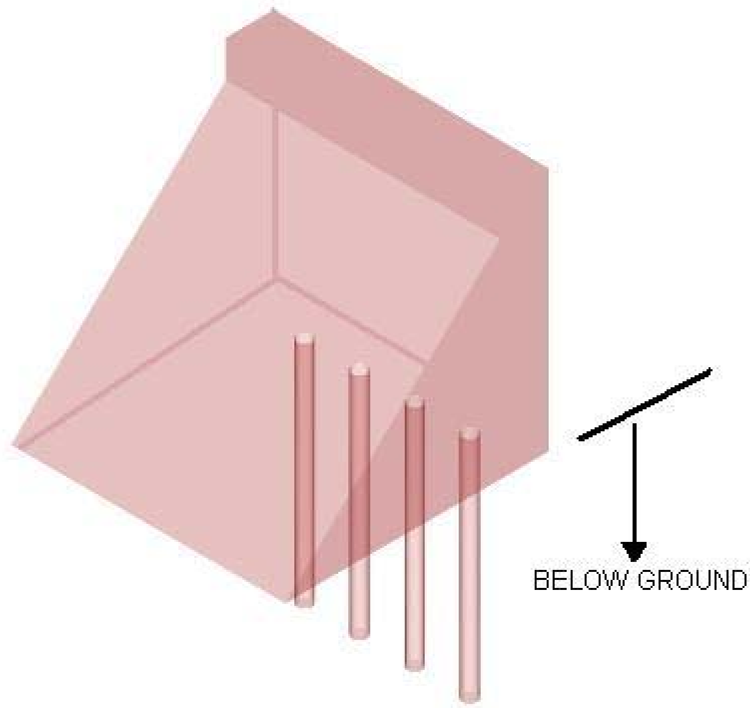


FIGURE 43. Single line of grout holes below a concrete gravity dam

Depending upon initial investigation and strata conditions, the spacing of primary hole treatment should be decided. If the primary holes were spaced more than 6 m apart secondary holes should be drilled and grouted. On completion of primary holes spaced closer than 6m or secondary holes (when the primary holes are spaced more than 6 m), should the percolation tests carried out in a few test holes indicate that further grouting of the area is necessary, secondary, or tertiary treatment as the case may be, should be carried out systematically thereafter in the whole area or in the particular section where the rock conditions are bad. Similarly tertiary holes should be taken over the whole area or the full length of the section which requires the treatment.

In addition to the systematic grouting of primary Secondary or tertiary and subsequent holes it may be necessary to drill and grout additional holes for treatment of peculiar geological feature such as faults, sheared zones and weathered rock seams.

Pattern of holes for consolidation grouting

The choice of pattern of holes, for consolidation grouting depends on whether it is necessary to wash and jet the hole systematically. When washing has to be carried out a hexagonal pattern (Figure 44) would be preferred as this admits for flow reversal. When systematic washing and jetting is carried out to remove all soft material in seams it is generally not necessary to use a primary and secondary system of holes.

When it is desirable to test the efficacy of consolidation grouting by comparing the grout absorption in primary and secondary holes a rectangular or square pattern (Figure 44) of holes would be preferred. This is generally the case when the joints are irregular and relatively free from in-filling or it is not necessary to remove the material filling the joints.

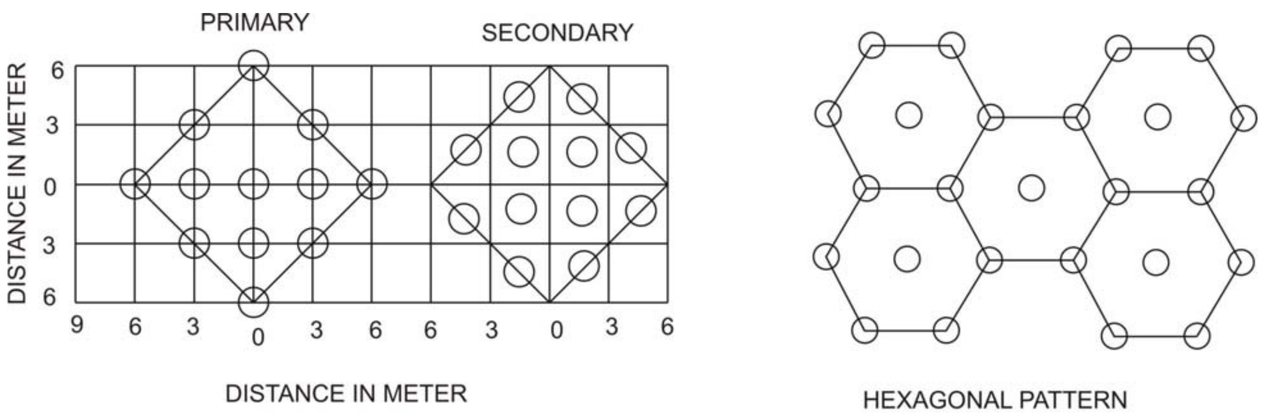


FIGURE 44.TYPICAL PATTERNS FOR CONSOLIDATION GROUTING

Grouting mixture

Rock grouting is normally performed with a mixture of cement and water with or without additives. The cement should be ordinary Portland, Portland Pozzolana, Portland slag, Supersulphated or Sulphate-resisting Portland. The solid materials which may be used as additive to the grout mixture could be Paxxolanas (such as fly ash and calcined shade), fine sand or other fine non-cementeitious materials like clay and silt. While using additives constant field checks and review should be undertaken to achieve the desired results in respect to permeability and strength. Admixtures when added in small quantities to the grout mixture impact certain desirable characteristics like delaying or hastening setting time and increasing the workability.

Drilling equipment

The entire grouting operation is carried out by first drilling and then injecting the grout under pressure. The various types of drilling equipments can be grouped as under

A. Precursive Drilling equipment
   a) Standard drifter or wagon drill
   b) Dom the hole drilling equipment, and
   c) Overburden drilling equipment

B. Rotary drilling equipment with suitable drive, that is hydraulic, electric, diesel or compressed air

Precussive drilling methods are generally more economical in all kind of rocks. For deep rocks it may be advantageous to use overburden drilling equipment. By virtue of the greater rigidity of the casing tube combined with the drill rods, better control on inclination of holes can generally be achieved in the overburden drilling equipment. Down the hole hammer is also capable of maintaining a better control on the inclination. However, the hammer may get clogged when the drill cuttings form slush in form saturated strata and cannot be removed by air flushing.

During precussive drilling in stratified rocks where the resistance of the rock is prone to variation the holes may get curved and control on inclination may be lost. In such cases guide tubes may be used for ensuring verticality of the holes or alternatively rotary drilling may be used. Irrespective of whether air or water is used for flushing the hole during drilling, thorough cleaning by water flushing is essential before starting grouting operations.

Grouting equipment

The major equipment required for carrying out grouting are Grout Mixer and Grout Pump. These are explained below.

Grout mixer: The mixer should have two tank namely mixing tank and agitating tank. Mixers are generally cylindrical in shape, with axis either horizontal or vertical or equipped with a system of power driven paddles for mixing. Grout should be mixed in a mixer operating at 1500 r.p.m. or more. The high speed of mixing serves the purpose of violently separating each cement grain from its neighbour thus permitting thorough wetting of every grain. This proves to be advantageous by chemically activating each grain to through hydration before reaching its final resting state. Further individual grains penetrate finer cracks more readily than flocs. Vertical barrel type mixers have proved satisfactory when small mixers are required for use in confined or limiting working spaces. This type of mixer consists essentially of a vertical barrel having a shaft with blades for mixing, driven by a motor mounted on top of the mixer above the barrel. Centrifugal pump mixers mix the grout by re circulating it through a high speed centrifugal pump. They are sometimes referred to as colloidal type mixers, but they don't achieve a true colloidal grout mix. However they possess considerable merit and produce grout of excellent texture. When mixing sand-cement grouts their action tends to guard against segregation.

Grout pump: A pump suitable for grouting should permit close control of pressures, allow a flexible rate of injection, and be designed to minimize clotting of valves and ports. Grout pimps are of three types namely, piston, screw, and centrifugal.

Washing and testing of holes and surface preparation

Any grouting operation requires major washing of the holes, testing of the holes with water under pressure and surface preparation. The purpose of washing the holes is two fold. First to clean the hole to remove the material deposited on the surface during the drilling operation and second to provoke deliberate inter-connections between adjoining grout holes to remove known seams and layers of erodable material. It should be borne in mind that inter-connections between holes are effective only if he washing operations are carried out systematically to remove all the soft material. Isolated inter-connections don't serve much useful purpose as soft material may still remain in position in a known and irregular pattern. A distinction is therefore made between washing of holes at the end of the drilling operation and systematically washing of group of holes in order to remove the erodable material in the intervening area for which the term jetting is used.

Washing of holes

On completion of a drilling of a stage and before injection, the holes should be washed by allowing drilling water to run until the return from the hoe is reasonably clean. The quantity of water flowing into the hole during the period should be adequate and generally not less than 15 l/min.

When no return of drilling or washing water occurs, the holes should be washed for a reasonable period based on site experience. This is generally for 20 minutes. If an abrupt loss of drill water occurs during drilling and similarly when a strong flow of artesian water is encountered, the drilling should be stopped and the hole grouted even if it has not reached its final depth.

Percolation tests

For routine grouting operations, and simple water test conducted before and after grouting, the test pressure should be limited so as to avoid hydraulic fracture. The value of limiting pressure for various strata and depths should be established by preliminary investigations where cyclic tests should be conducted to evaluate pressure at which fracturing occurs. Additional tests may be carried out in trial grouting plots or in selected primary grouting holes to verify the pressure limits established during preliminary investigations.

Water percolation tests may be used to measure the effectiveness of the grouting treatment. The tests may be simple or cyclic. Cyclic testing is recommended for evaluation stage while before and during grouting operations simple tests should be carried out.

Water tests should be carried out in primary stages before injection to amplify information available from the site investigation. Tests should be carried out in secondary stages before injection to indicate the results of primary injections. Test may be carried out in individual test holes at any time to indicate the results of all treatment carried out before that time. Test holes drilled for this purpose should be sited midway between completed injection holes.

Jetting

Jetting operation are carried out in order to deliberately provoke connection between bore holes and to remove known deposits of erodable material. Jetting should be carried out in group of holes arranged in a square, triangular or hexagonal pattern known as cells.

Surface treatment

For effective treatment of the surface zones, sufficient pressure should be developed to achieve the spread required witH a convenient spacing of holes. Adequate cover should be maintained during grouting to ensure that adequate pressure is applied without causing upheaval or excessive surface leakage.

Injection of grout

As for the method of injection, grout holes should be injected by direct connection to the pump. Each pump should be provided with a packer at the surface or with a short strand pipe threaded at its outer end to accept stand or control fittings, which should be provided with a pressure gauge, bleeder valve and a valve enabling delivery from pump to be cut-off from the hole. Either single line or circulation system may be used, usually circulating system is preferred, however when adequate controls are possible to regulate the pump discharge and pressure by using pumps of suitable design, single line grouting system can be used.

Once the grouting of stage or group of holes has been commenced it should be continued without interruption up to completion. In general a stage may be considered complete when the absorption of grout at the desired limiting pressure is less than 2 l/min averaged over a period of 10 minutes.
As far as practical a continuous flow of grout should be maintained at the desired pressure and the grouting equipment should be operated to ensure continuous and efficient performance throughout the grouting operation. After grouting is completed, the grout holes should be closed by the means of a valve to maintain to grout pressure for a sufficient period to prevent escape of the grout due to back pressure and flow reversal, due to causes like artesian conditions. For this purpose a period of one or two hours is generally sufficient, however this should be verified by trial.

Pressure

The grouting pressure should be adequate to achieve the desired grout and the pressure should be limited so as to avoid disturbances and upheaval of the ground and should take into account reservoir pressure.

For structures on rock foundations, it is a basic requirement that no disturbance should be caused to the surface zones of the foundation by the grouting operation. When grouting is undertaken below an existing structure no upheaval of the foundation can be allowed as it would have very harmful consequences on the structure and/or the equipment. In general the disturbance caused by the grouting is dependent more on the manner in which the pressure is developed and the nature of the rock than on the absolute magnitude of pressure. Relative higher pressures can be sustained without damage to the foundations, when pressure is built up gradually, as resistance to flow is developed by deposition of grout On the other hand when pressures are raised hastily damage can occur even at relatively low pressure. In general, horizontal stratified or low dripping rocks are more vulnerable to disturbance by grouting pressure than fractured igneous or metamorphic rocks or steeply dipped sedimentary rocks. Rocks previously subjected to folding or fracturing or rocks in the process of adjustment after removal of overburden load are also more vulnerable to disturbances.

The most common difficulty experienced in consolidation grouting is surface leakage. It is therefore customary to pipe through the entire height of concrete or masonary and carry out the grouting after the rock has been completely covered. This not only eliminates surface leakage but permits use of higher pressure so that even the smaller seams can be grouted effectively.

4.6.8 Temperature control of mass concrete for dams

When a concrete gravity dam is constructed of mass concrete, it undergoes volumetric changes with time due to the release of heat of hydration by the concrete. A rapid rise in the temperature of mass concrete takes place during the phase when the concrete mass is in plastic stage and undergoes hardening. After hardening, the concrete gradually cools due to effect of atmospheric temperature, which tends to subject the concrete to high tensile stresses. Cracking occurs in the concrete when these tensile stresses exceed the tensile strength of the concrete. This cracking is undesirable as it affects the water tightness, durability and appearance of hydraulic structures. Hence, methods to control the temperature rise during dam construction is absolutely essential. The methods to control temperature in dams is prescribed in the Bureau of Indian Standard code IS: 14591-1999 "Temperature control of mass concrete for dams – guidelines", some of which are given below.

Most commonly used methods are precooking, post cooling and reducing heat of hydration by proper mix design. The ideal condition would be simply to place the concrete at stable temperature of dam and heat of hydration removed, as it is generated, so that temperature of concrete is not allowed to rise above stable temperature. However this is not possible to achieve practically. Therefore, the most practical method is to pre cool concrete so as to restrain the net temperature rise to acceptable levels.

Pre-cooling

One of the most effective and positive temperature control measure is precooling which reduces the placement temperature of concrete. The method, or combination of methods, used to reduce concrete placement temperatures will vary with the degree of cooling required and the equipment available with the project authority or the contractor. In this method usually the fine and coarse aggregates and the water are separately cooled to the requisite temperatures.

Mixing water may be cooled to varying degrees, usually from 0^0 C to 4^0 C. Adding crushed ice or ice flakes to the mix is an effective method of cooling because it takes advantage of the latent heat of fusion of ice. The addition of large amount of ice flakes, however, may not be possible in cases where both coarse aggregate and sand contain appreciable amount of free water, in which case the amount of water to be added to the mix may be so small that substitution of part of the water to be added with ice may not be feasible. From practical considerations, not more than 70 percent of water should be replaced by crushed ice. Although most rock minerals have comparatively low heat capacity, since aggregates comprise the greatest proportion of concrete mix, the temperature of the aggregate has the greatest influence on the temperature of the concrete. Cooling of coarse aggregate to about 1.7°C may be accomplished in several ways. One method is to chill the aggregates in large tanks of refrigerated water for a given period of time or by spraying cold water. Effective cooling of coarse aggregate is also attained by forcing refrigerated air through the aggregate while the aggregate is draining in stock piles, or while it is in a conveyer belt or while it is passing through the bins of the batching plant.

Post-cooling

Post cooling is a means of crack control. Control of concrete temperature may be effectively accomplished by circulating cold water through thin walled pipes embedded in concrete. This will reduce the temperature of newly placed concrete by several degrees, but the primary purpose of the system is to accelerate the subsequent heat removal and accompanying volume decrease, during early ages when the elastic modulus is relatively low. Post cooling is also used where longitudinal contraction joints are provided in order to reduce the temperature of concrete to the desired value prior to grouting of transverse contraction joint. Post cooling will create a flatter temperature gradient between the warm concrete and the cooler exterior atmosphere which, in turn, helps in avoiding temperature cracks. Other methods such as evaporative cooling with a fine water spray, cold water curing and shading may prove beneficial, but the results are variable and do not significantly affect the temperature in the interior of massive placement. The embedded cooling system consist of aluminum or synthetic plastic pipe or tubing generally of 25 mm diameter and 1.50 mm wall thickness placed in grid like coils over the top of each concrete lift. When the expected active cooling period exceeds 3 months, steel tubing should be used. The number of coils in a block depends upon the size of the block and the horizontal spacing of the pipes. For practical reasons, pipe coils are placed and tied to the top of a hardened concrete surface and thus vertical spacing of the pipe corresponds to lift thickness. A horizontal spacing same as the vertical spacing will result in the most uniform cooling pattern but variations may be allowed. Supply and return headers, with manifolds to permit individual connections to each coil are normally placed on the downstream face of the dam. In some case, cooling shafts, galleries and embedded header system may be used to advantage.

4.6.9 Concreting procedures for gravity dams

A concrete gravity dam is normally executed as a mass concrete work, except for some reinforced concreting works as in:

- Piers and bridges over the spillway
- Around galleries and other openings
- Divide wall between adjacent spillways
- Energy dissipators

- Intake to sluices
- Power house if built as a part of the dam

The Bureau of Indian Standards code IS 457-1957 "Code of practice for general construction of plain and reinforced concrete for dams and other massive structures" provides guidelines for practices to be followed in plane and reinforced concreting for mass concrete dams. The main points that have to be taken care are mentioned in the following paragraphs

Aggregate production

Huge quantities of aggregate would be required for the construction of a massive structure like a concrete gravity dam The acceptability of the natural aggregate is to be judged upon the physical and the chemical properties of the material and the accessibility, proximity to site and economic workability of the deposit. A suitable quarry has to be identified in the neighbourhood that can supply continuous source of aggregates.

Structural steel

These may be according to the latest recommendations of the relevant bureau of Indian Standard Codes.

Concrete production and handling

Standard practice is for materials to be batched by weight. The time of mixing is often specified as 2 minutes. The procedure to be adopted for moving concrete from the mixers on to the dam will be governed by site conditions. Having produced a good plaeable concrete, the problem is to transport it to the dam site with the least possible segregation or change in consistency, so that it may be compacted uniformly into the dam without reasonable effort. Nowadays a cableway laid across the dam valley is often used with buckets of capacity 1.5 to 2 m^3. At many construction sites concrete is placed using chutes or even a belt conveyor. It is recommended that concrete shall have to be placed in position within 30 minutes of its removal from the mixer.

Concrete placing, consolidation and curing

For laying concrete over the rock foundations, it has to be ensured that the surface is clean and free from mud, dirt, oil, organic deposits, or other foreign material which may prevent a tight bond between rock and concrete. In case of earth or shale foundations all soft or loose mud and surface debris shall have to be scrapped and removed. Then the surface has to be moistened to a depth of about 15 cm to prevent the subgrade from absorbing water from the fresh concrete. A layer of concrete that is laid is generally kept as 1.5 m, in a view to ease construction and limit excessive temperature rise. These layers of concreting are called lifts and between two successive lifts a horizontal joint would invariably arise. The concrete of subsequent lifts has to be placed after allowing sufficient time for the previously laid concrete to cool and attain its initial set and become hard. Prior to placement of concrete of the next lift, the surface of the previously placed concrete has to be thoroughly cleaned by the use of high velocity jet of water and air as well as by wet-sand blasting. Further immediately before the concrete placing of the next lift begins, a 12.5 mm thick layer of mortar should be applied to permit proper bond between the concrete of the lower lift. Since the area of the concrete block near the foundation would be quite large, joints in the vertical plane, but parallel to the dam axis have to be introduced to ease the concrete placement and to allow safe dissipation of the heat of hydration of concrete. These joints called the longitudinal joints are normally spaced at intervals of 15m to 30 m. Thus during construction a continuous concrete pour is seen to be confined between the transverse joints (defining a block) and the longitudinal joints. Once a lift is cast it is thoroughly compacted with needle vibrators. The longitudinal joints subdivide each block formed by the transverse joints into several smaller sub blocks, but since each block must be a monolithic, these joints are invariantly provided with horizontal keys (or undulations)over the entire surface, which helps to make a good bond with the adjacent lift . It has however, been recognized that the provision of longitudinal joints is basically unsound unless a high degree of perfection is maintained while placing the adjacent pore of concrete and then grouting the gap properly. Hence the present practice is to avoid the longitudinal joints altogether, even in the case of high dams and a better alternative is to attain necessary temperature control by the methods described earlier. Curing of concrete is important but a difficult task for the construction engineer. Primarily it is necessary to maintain satisfactory moisture content in the hardening concrete. This may be achieved either by the application of water (usually from sprinklers or perforated hoses, or occasionally by panding on the top of the lift) or by prevention of loss of water (by application of some membrane to the surface). A second requirement for good curing is favourable temperature. This can be achieved by any of the water methods but not by the membrane methods.

4.6.10 Instrumentation in concrete gravity dams

Normally, instruments are installed in a concrete gravity dam to measure the various parameters that indicate the structural health of the dam and the state of the foundation. These instruments have been classified into two types: obligatory and optional, by the Bureau of Indian Standards code IS 7436(part2)-1997 “Guide for types of measurements for structure in river valley projects and criteria for choice and location of measuring instruments”. These two types of instruments are explained in the following paragraphs.

Obligatory Measurements

The following measurements are obligatory for all dams:

a) Uplift pressure at the base of the dam at a sufficient number of transverse sections
b) Seepage into the dam and appearing downstream there-from;
c) Temperature of the interior of the dam; and

d) Displacement measurements - Except for very small structures (of height 20 m and below not involving any foundation complications). Displacement measurements should include one or more of the following types of -measurements:

1) Those determined by suspended plumb lines;
2) Those determined by geodetic measurements where warranted by the importance of the structure;
3) Those determined by embedded resistance jointmeters at contraction joints where grouting is required to be
done.

Optional Measurements

The following measurements are optional and may be undertaken where warranted by special circumstances of project. These would be beneficial for high dams, for structures of unusual design, for structures where unusual or doubtful foundations exist, for the verification of design criteria and for effecting improvement in future designs:
a) Stress
b) Strain
c) Pore pressure (as distinct from uplift pressure), and
d) Seismicity of the area and dynamic characteristic of the structure.

Due to ever increasing demand of power, emphasis has been laid to construct large size of hydro electric project with very high dam. With the present trend, dam sites are neither geologically seismically suitable as most of the best sites have already been considered for the purpose. Due to this reason those measurements that were considered optional at the time of framing this code may become obligatory now for high dams specially in the Himalayan region.

The various types of parameters that are measured, whether obligatory or optional, are described in the following paragraphs.

Measurement of Uplift Pressure

It is important to determine the magnitude of any hydraulic pressure at the base of a dam. The effect of uplift on a dam is to reduce its effective weight on account of resulting buoyancy.

Measurement of Seepage

Seepage is, undoubtedly, the best indicator of the overall performance of a dam because this reflects the performance of entire dam and not just the condition at discrete instrumented points. Any sudden change in the quantity of seepage without apparent cause, such as a corresponding change in the reservoir level or a heavy rainfall, could indicate a seepage problem. Similarly, when the seepage water becomes cloudy or discoloured, contains increased quantities of sediment, or changes radically in chemical content, a likely serious seepage problem is indicated.

It is customary to provide grout curtain near the upstream face of the dam. Besides, a drainage curtain in the foundation and porous drain in the body of the dam are provided to intercept any seepage that passes through the grout curtain and through the body of the dam respectively. Measurement of seepage water along with uplift measurement at the plane of contact of the dam and its foundation will give direct indication of the effectiveness of the grout curtain and drainage curtain and will indicate whether any remedial measures are necessary. The chemical analysis of the seepage water through the foundation drainage system will help in assessing whether any foundation material is being washed out.

Likewise the quantum of water passing through ungrouted contraction joints or cracks would indicate about the workmanship in general as also any damage that might have been caused to the seals in the contraction joints. The chemical analysis of the water in the case of masonry dam may be indicative of any possible leaching action on the mortar used in the construction of the masonry dam. Corrective measures such as grouting of dam and foundation, besides improving existing drainage or providing additional drainage could thus be planned.

Wet spots or seepage appearing at new or unplanned locations at the abutments or downstream of a dam could also indicate a seepage problem. Measurement of-seepage downstream of the grout curtain provides a direct indication of the adequacy and effectiveness of the grout curtain, drainage curtain and functioning of the drains and holes to decide when and where remedial measures may be required.

Measurement of Temperature during Construction

For concrete gravity dams it is very important to know the thermic variations in the dam during its construction which enables to determine whether the concrete setting process is normal or otherwise. To achieve this purpose, temperature measuring devices are embedded within the dam body and also mounted on the surface according to a predetermined plan for useful observations. Any abnormal setting process indicated by temperature observations may lead to a change in the concrete lift height, and also changes in the treatment of aggregates before concreting and of the mass concrete during curing.

Measurement of Temperature of the Dam interior

It is necessary to measure temperature in the body of concrete and masonry dams in order to ascertain the nature and extent of thermal stresses and the consequent structural behaviour of the dam and also to ascertain when to undertake grouting of contraction joints that may have been provided for the structure.

Measurement of Temperature of Reservoir Water and Air

Measurement of temperature of reservoir water and air is essential for distinguishing the effects of ambient and water temperatures on such measurements as deflection, stresses, strains, joint movements and settlements.

Measurement of Displacement

Measurement of displacement of points either between two monoliths, or between foundation and body of the dam or the displacement of any joint of the dam with respect to the surrounding area will immediately reveal any distress conditions developing in the dam. Measurement of displacement is thus one of the most important factors to be studied while observing the structural behaviour of a dam.

Internal Joint Movement

Concrete and masonry dams are generally built in blocks separated by transverse joints. It is essential to know whether there is any relative movement between two blocks. The movement is likely to be due to differential foundation behaviour. Further, the relative movement of blocks is important from the point of view of growing of transverse contraction joints.

Surface Joint Movement

Measurement of joint movement at the surface of the locations accessible from galleries is made by detachable gauges with a view to assess the amount of joint opening of two blocks of the dam. These gauges may also be advantageously used for observation of opening or of closing of surface cracks at any location.

Foundation displacement

Measurement of vertical or horizontal displacement of foundation provides information for taking preventive measures for inclination, distortion etc. of structures. The data can also be used for studying the elastic and inelastic properties of dam and foundation. Measurement of foundation displacement involves vertical and horizontal displacement of part of foundation with respect to dam.

Displacement of One Part of the Dam Relative to other Parts of the Dam

Measurement of relative displacement of two points in a dam is a direct indication of structural behaviour of the dam. The deflection characteristics of a dam observed for the first few years will reveal any dangerous tilt or movement of the dam. These observations are made by regular and inverted plumblines. The plumbline data together with other supporting data may be used to study the elastic behaviour of the dam.

Displacement of Dam with Reference to Surrounding Area

This measurement gives the absolute displacement of the dam with respect to surrounding area, and is a direct indication of structural behaviour of the dam. Provisions would be made for periodic deflection measurements. Where topography permits, this can be done by theodolite from fixed bases, using either line-of-sight over the top of the dam or by turning angles to targets on the downstream face and at the crest. At concrete dams, the deflections should be consistent with changes in reservoir water surface level and in temperature and should not change appreciably from year to year.

Measurement of Tilt

Tilt is measurement of rotation in vertical plane. It is normally measured with the help of tiltmeter system consisting of tiltmeter sensor, tilt plates and indicator. Tilt plates are bonded to the surface of mass of structure under observation. The sensor is oriented on three pegs of tilt plate and senses change in tilt of tilt plate. The portable indicator gives the degree of rotation.

Measurement of Stress

Direct measurement of stress developed inside the mass of concrete or masonry helps in watching the structural behaviour of dams and their foundations. Any adverse change in stress will indicate distress conditions and remedial measures can be taken. The observation of stress also helps in studying the assumed stresses and actual stresses in dams and this can be used in improving upon the design procedure.

Measurement of Strain

Factors like temperature, chemical action, moisture change and stress result in volume changes which case strain in the structure. The measurement of strain, therefore, becomes necessary. As the design of structures is based on stress it is essential to measure the stresses developed in the structures during its life time. Moreover, the instruments available for measurement of stresses can measure only compressive stress and not the
tensile stress. Further, the stress measuring instruments are more expensive and delicate than strain meters and hence, it is a common practice to measure the strain and to calculate from it the developed stress.

Measurement of Pore Pressure

Since large concrete and masonry dams are provided with internal formed drains located near the upstream face, a record of pore pressure development and its variations would indicate the effectiveness and adequacy of these drains. Any sudden unusual increase in the pore pressures will be indicative of choking up of these internal drains and any unusual reduction from the normal would indicate possibility of formation of cracks or establishment of flow channels in the body of the dam. Measurement of uplift in the foundation is mandatory for all gravity dams and is generally accomplished by uplift pressure pipes which provide a direct indication of the prevailing magnitude of uplift resulting from the operating reservoir heads and consequently the effectiveness of the grout curtain close to the upstream face of the dam and effectiveness of the drainage curtain provided in the foundation apart from checking of design assumptions for its stability.

Seismicity of the Area and Dynamic Characteristic of the Structure

Surveillance of seismic environment of the project site needs special attention in case of large dams to know about the seismicity of the region before taking up construction. Creation of a reservoir generates additional load on the surrounding area and underlying geological strata. Thus, it becomes essential to know the change in the seismicity pattern, if any, due to creation of large reservoir. The behaviour of dam during an earthquake also needs to be assessed. For these purposes, a seismological laboratory may be established near the project site.

Measurement of Water Level on Upstream and Downstream Side

This measurement is useful for calculating the water pressure on the upstream face and downstream face of the dam.
A typical set of piezometer installations for an embankment dam is shown in Figure 45.

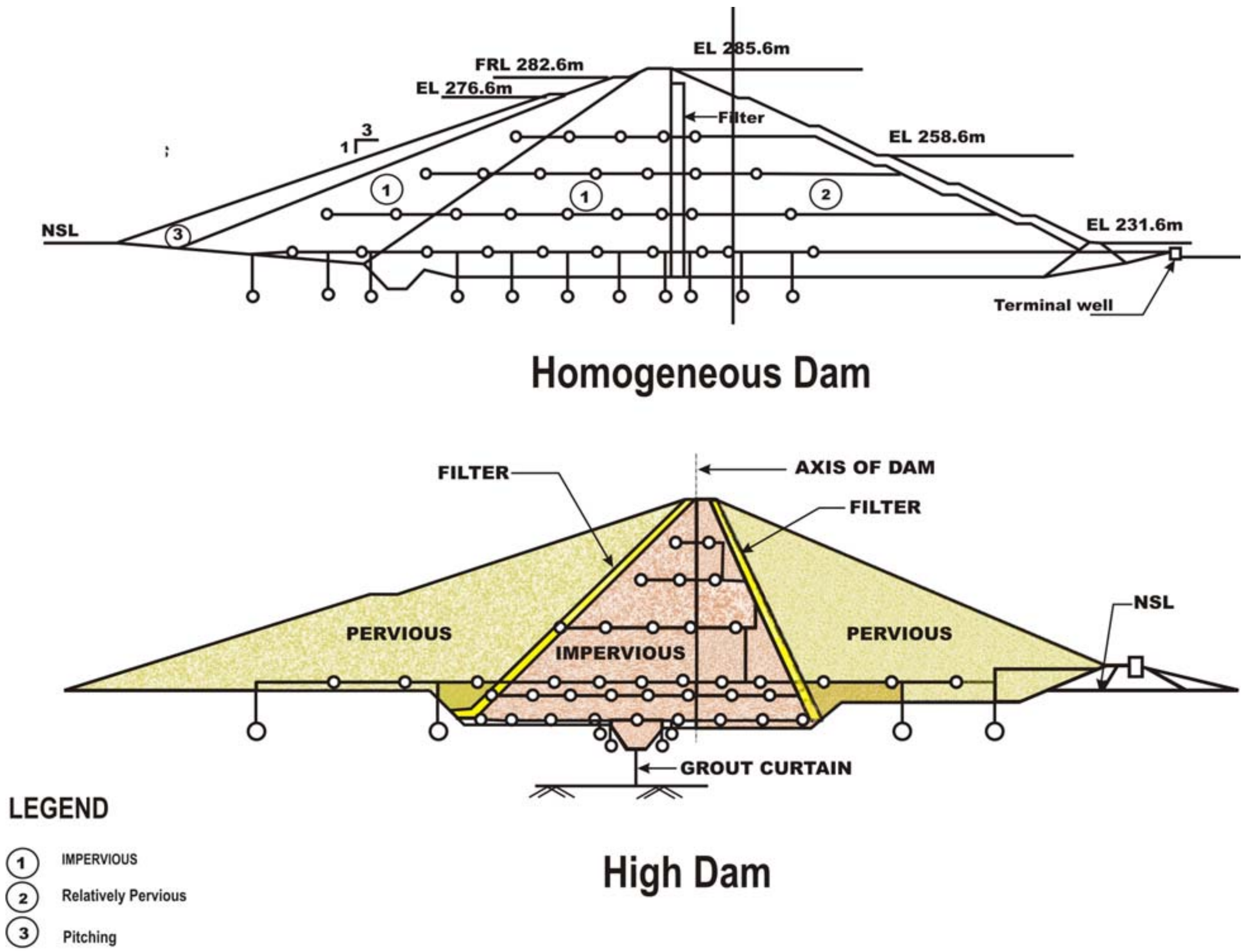


FIGURE 45. Typical installation of piezometers in embankment dams

4.6.11 Structural design of special components

Although maximum value of concrete in gravity dam construction goes into mass concreting, reinforcements are required at some places for resisting tensile stresses. Of these two components are quite important: Galleries and spillway piles. The Bureau of Indian Standards has brought out two codes on the structural design of these components and they are as follows;

1. IS: 12966(Part 2)-1990 “Code of practice for galleries and other openings in dams” (Part 2: Structural design)
2. IS: 13551-1992 “Structural design of spillway piers and crest–criteria”

Though the details may be found in the above code it may be mentioned that galleries are openings in the body of the dam that introduce stress concentration in its surroundings. This concentration would be minimum if the openings are circular or nearly so. But for ease of operation, the galleries are mostly rectangular thus accentuating the stress concentration at the corners. Hence reinforcement has to be provided all round the openings, and special care taken where two openings meet, say at the junction of the foundation gallery and an inspection adit. Is 12966(part 2) -1990 gives guidelines for both evaluating the stresses and design of reinforcements. It also illustrates typical layout of placing reinforcements.

As for the spillway piers, they are erected over the crest profile and are provided to divide the spillway into a number of bays so as to control the flow over the spillway by installing gates between two piers. Piers are also used to support the bridge over the spillway for the movement of the gantry crane and normal traffic. IS 13551-1992 helps to identify the forces acting on a pier and method to compute the induced moments and stresses. It also illustrates typical layout of reinforcement in a pier and its junction with the spillway crest.

It may be noted that all reinforced concrete works in concrete dams have to conform to IS 456-2000 “code of practice for plain and reinforced concrete”. The following two Bureau of Indian Standard codes may also be referred to for further details related to training walls, divide walls and spillway anchorages.

1. IS 12720-1992 “Criteria for structural design of spillway training and divide walls”
2. IS 95-1993 “Design aid for anchorages for spillway structures”

Module 4

Hydraulic Structures for Flow Diversion and Storage

Lesson
7
Design and Construction of Concrete Gravity Dams

Instructional objectives

On completion of this lesson, the student shall learn:

1. The different types of embankment dams
2. Causes of failure of embankment dams
3. Design procedure for earthen embankment dams
4. Seepage control measures for embankment dams and their foundations
5. Computation of seepage through embankment dams
6. Stability calculations for embankment dams
7. Construction process for embankment dams
8. Instrumentation in embankment dams

4.7.0 Introduction

An embankment dam, as defined earlier, is one that is built of natural materials. In its simplest and oldest form, the embankment dam was constructed with low-permeability soils to a nominally homogeneous profile. The section featured neither internal drainage nor a cutoff to restrict seepage flow through the foundation. Dams of this type proved vulnerable associated with uncontrolled seepage, but there was little progress in design prior to the nineteenth century. It was then increasingly recognized that, in principle, larger embankment dams required two component elements.

1. An impervious water-retaining element or core of very low permeability of soil, for example, soft clay or a heavily remoulded 'puddle' clay, and
2. Supporting shoulders of coarser earthfill(or of rockfill), to provide structural stability

As a further enhancement to the design, the shoulders were frequently subject to a degree of simple zoning, with finer more cohesive soils placed adjacent to the core element and coarser fill material towards either face. Present embankment dam design practice retains both principles. Compacted fine grained silty or clayey earthfills, or in some instances manufactured materials, like asphalt or concrete, are employed for the impervious core element. Subject to their availability, coarser fills of different types ranging up to coarse rockfill are compacted into designated zones within either shoulder, where the characteristics of each can best be deployed within an effective and stable profile.

Although the loads acting on the concrete gravity dam (discussed in lesson 4.6) is the same acting on the embankment dam, the method of design and analysis of the two differ considerably. This is mostly because the gravity dam acts as one monolithic structure, and it has to resist the destabilizing forces with its own self weight mainly. Failure to do so may lead to its topping, sliding or crushing of some of the highly stressed regions. An embankment dam, on the other hand, cannot be considered monolithic. It is actually a conglomerate of particles and on the action of the various modes, which are much different from those of a gravity dam. Hence, the design of an embankment dam is done in a different way than that of a gravity dam. In fact, the design procedures are targeted towards resisting the failure of an embankment dam under different modes, which are explained in the next section.

4.7.1 Embankment dam and appurtenant structures-basic types and typical layouts

An embankment dam, whether made of earth completely or of rock in-filled with earth core, has a trapezoidal shape with the shoulder slopes decided from the point of stability against the various possible modes of failure, discussed in section 4.7.2. The top crest is kept wide so as to accommodate roadway (Figure1). In order to check the seepage through the body of the dam, a number of variations are provided. For earthen embankment dams, these range from the following types:

1. Homogeneous dam with toe drain (Figure 2)
2. Homogeneous dam with horizontal blanket (Figure 3)
3. Homogeneous dam with chimney drain and horizontal blanket (Figure 4)
4. Zoned dam with central vertical core and toe drain (Figure 5)
5. Zoned dam with central vertical core, chimney filter and horizontal blanket (Figure 6)
6. Zoned dam with inclined core, chimney filter and horizontal blanket (Figure 7)

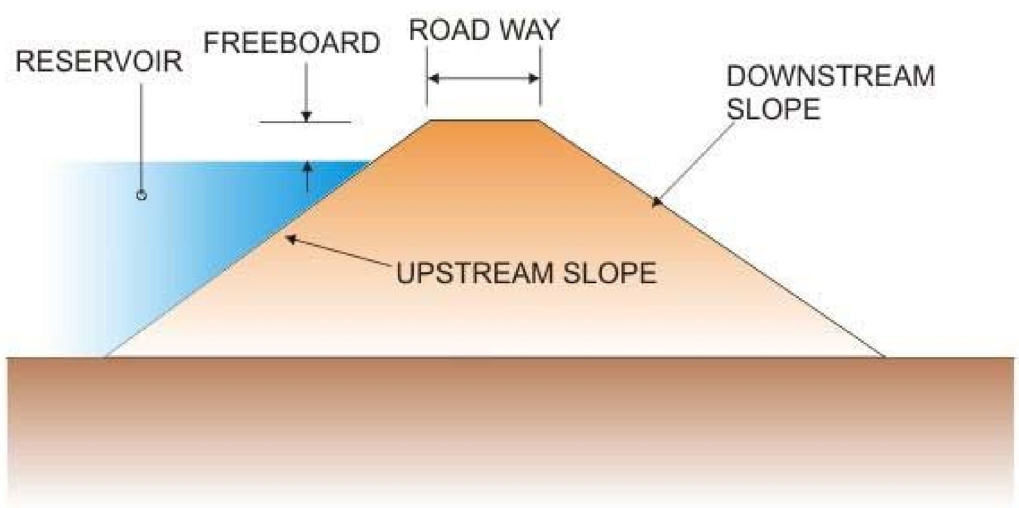


FIGURE 1. General shape of an embankment dam

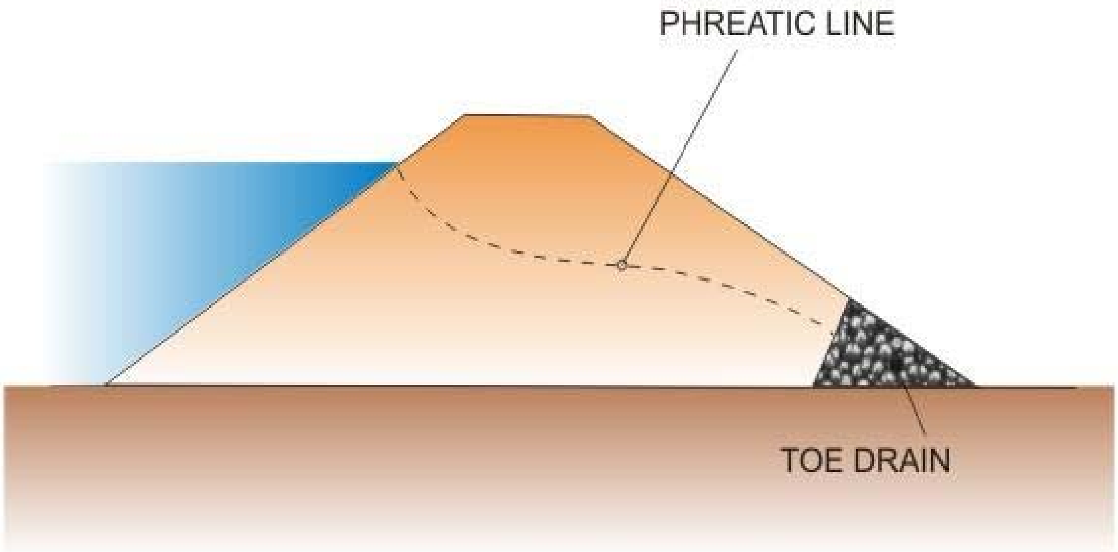


FIGURE 2. Homogeneons earthen embankment dam with toe drain

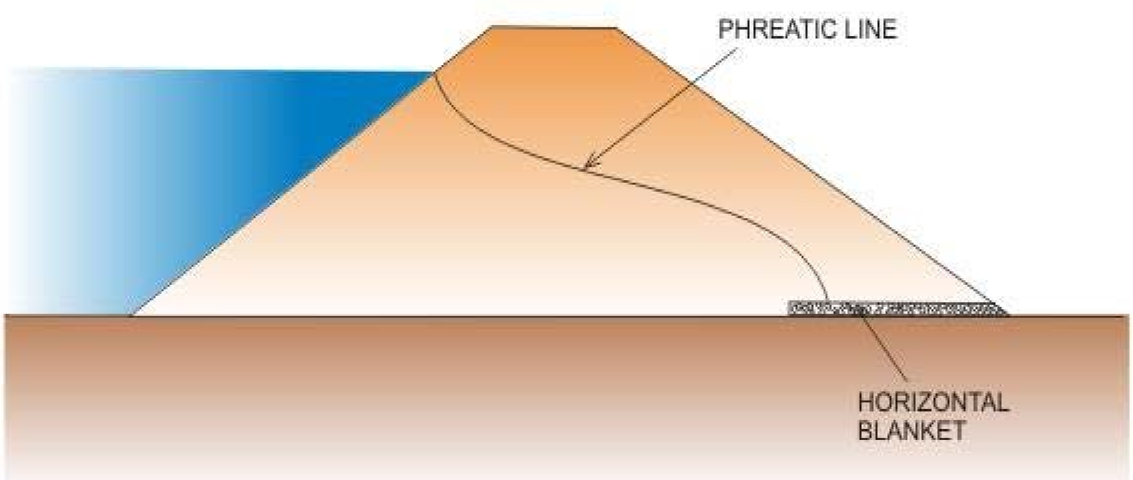


FIGURE 3.Homogeneous dam with horizontal blanket

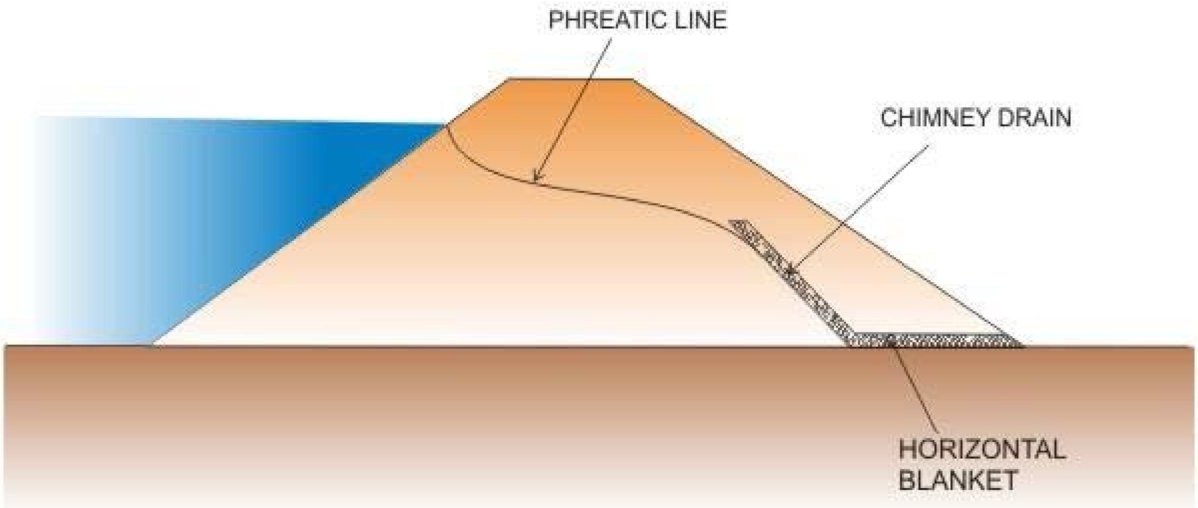


FIGURE 4. Homogeneous dam with inclined chimney drain connected to horizontal blanket

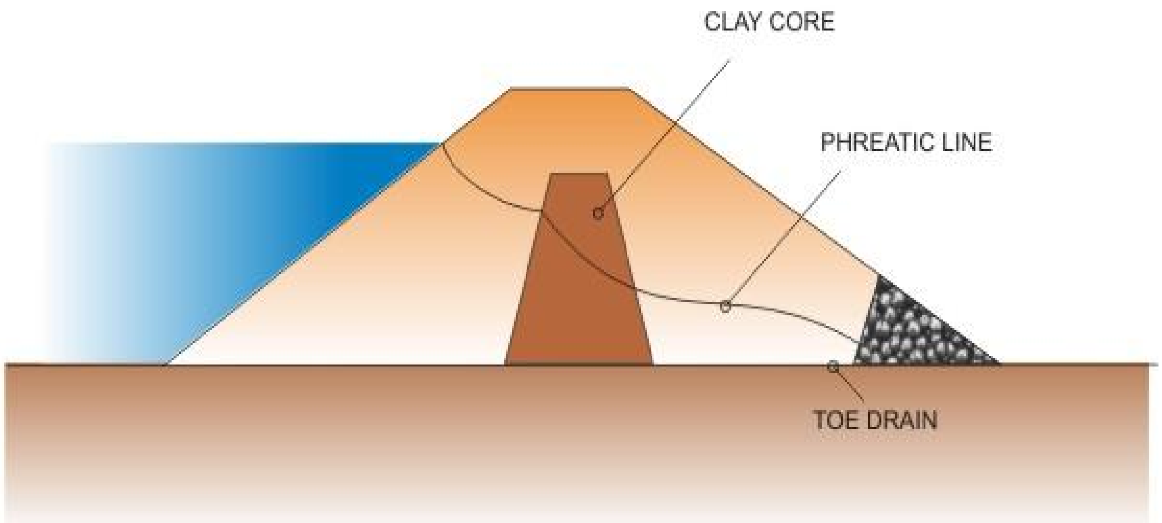


FIGURE 5. Zoned dam with central vertical clay core and toe drain

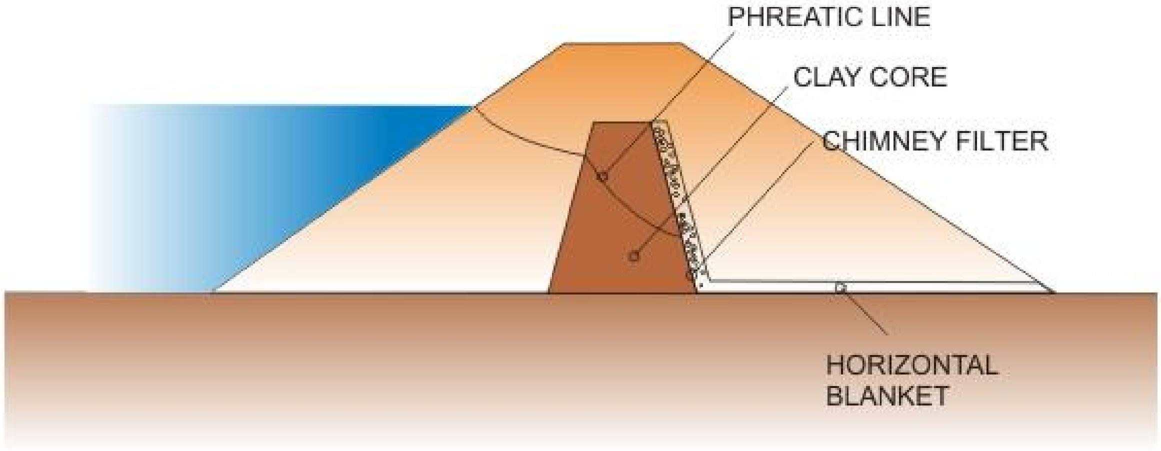


Figure 6. Clay core zoned dam with central vertical core and chimney filter with horizontal blanket

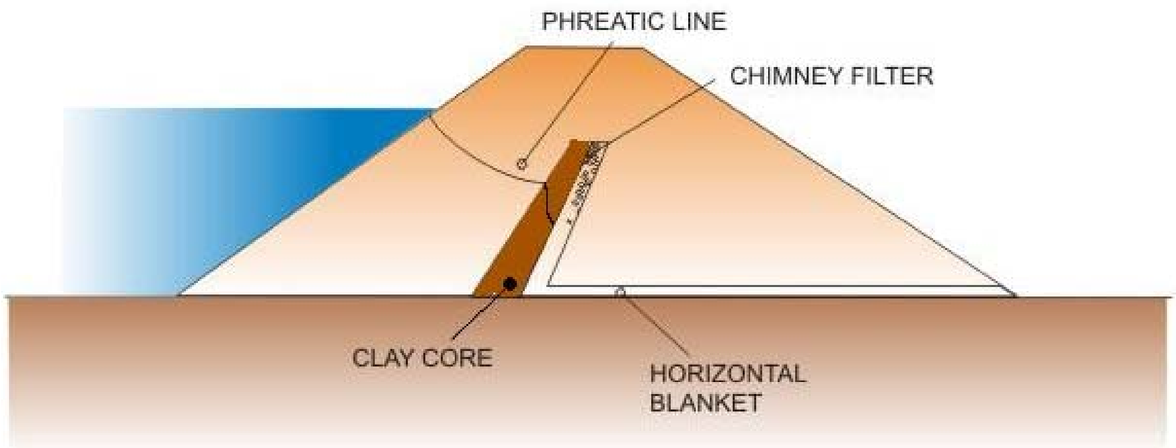


FIGURE 7. Inclined Clay core zoned dam with chimney filter and horizontal blanket.

An embankment dam is not as impervious as a concrete dam and water continuously seeps through the dam body. In Figures 2 to 7, the position of the phreatic line (that is, the line corresponding to the phreatic surface lying above the saturated zone when seen in a vertical plane) has been marked in the respective figures. It may be noticed how the phreatic line is forced to remain within the dam body by providing the clay core (with relatively less permeability) to reduce the amount of seepage water and the chimney filter with the horizontal blanket, composed of materials with very high permeability, that is used to drain out the seepage water safely through the body of the dam.

For the rockfill embankment dams, the following variants are common:

1. Central vertical clay core (Figure 8)
2. Inclined clay core with drains (Figure 9)

3. Decked with asphalt or concrete membrane on upstream face with drains (Figure 10)

All are provided with a chimney filter connected to a horizontal blanket.

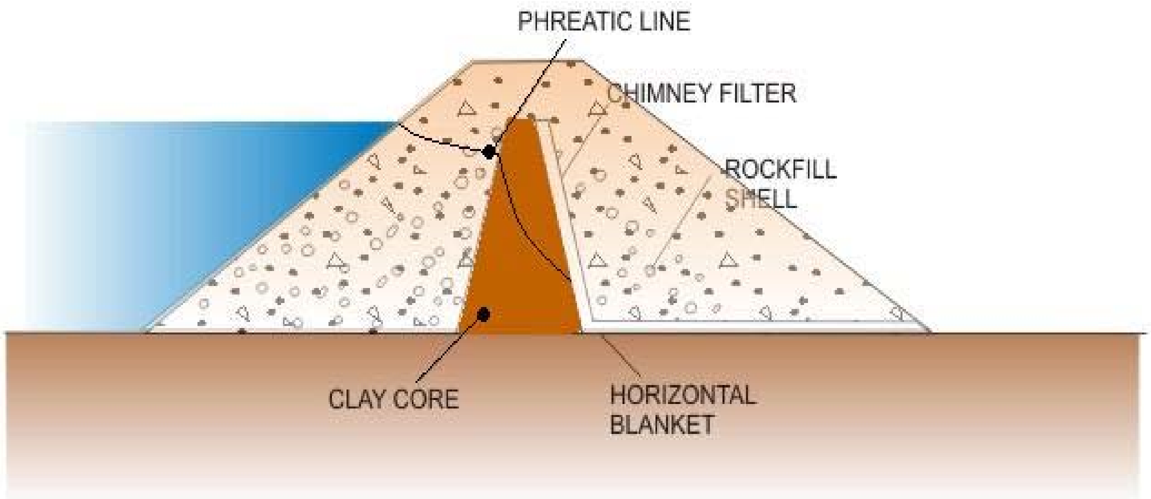


FIGURE 8. Rockfill dam with vertical clay core, chimney filter and horizontal blanket.

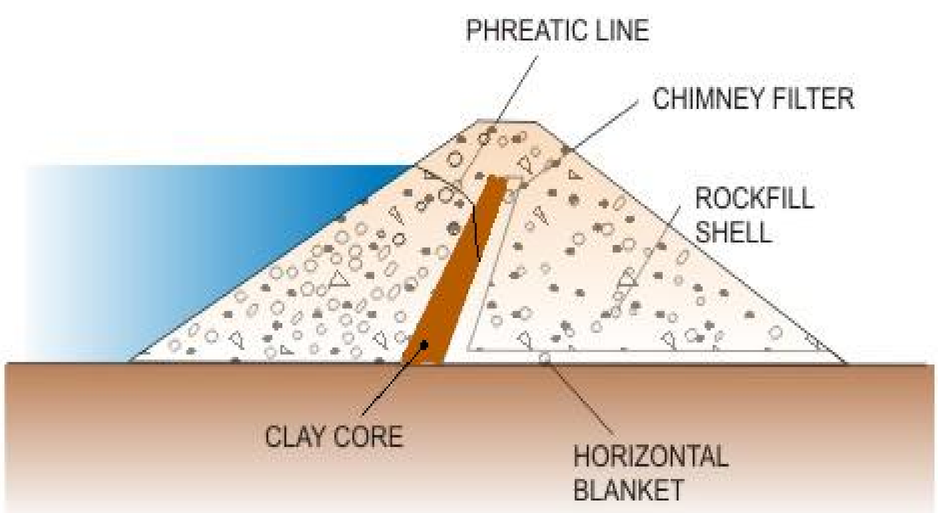


FIGURE 9 . Rock fill dam with inclined clay core, chimney drain and horizontal blanket

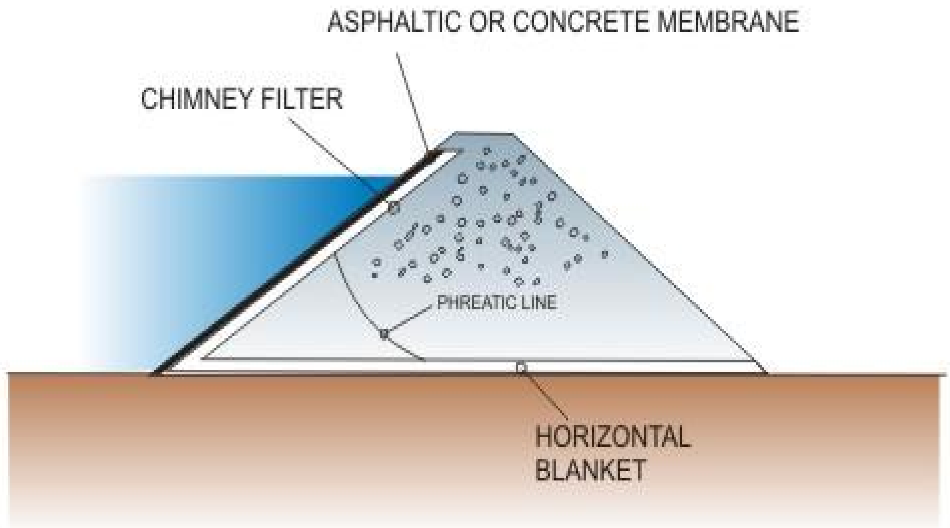


FIGURE 10. Decked rock fill dam with upstream asphaltic or concrete membrane with chimney drain and horizontal blanket. The phreatic live is for the small amount of water that leaks through the cracks of the upstream membrane.

Here, too it may be observed that the clay core with relatively very low permeability or the asphaltic and concrete membranes serve to reduce the quantity of seepage water. The rockfill shell only serves as a support to the core or membrane. The phreatic line through the rock fill is very gently inclined due to the materials of high permeability.

Since both the upstream and downstream faces of an embankment are inclined, usually varying in the ratio of 1V:1.5H to 1V:2.0H, the plan view of such a dam in a river valley would look as shown in Figure 11.

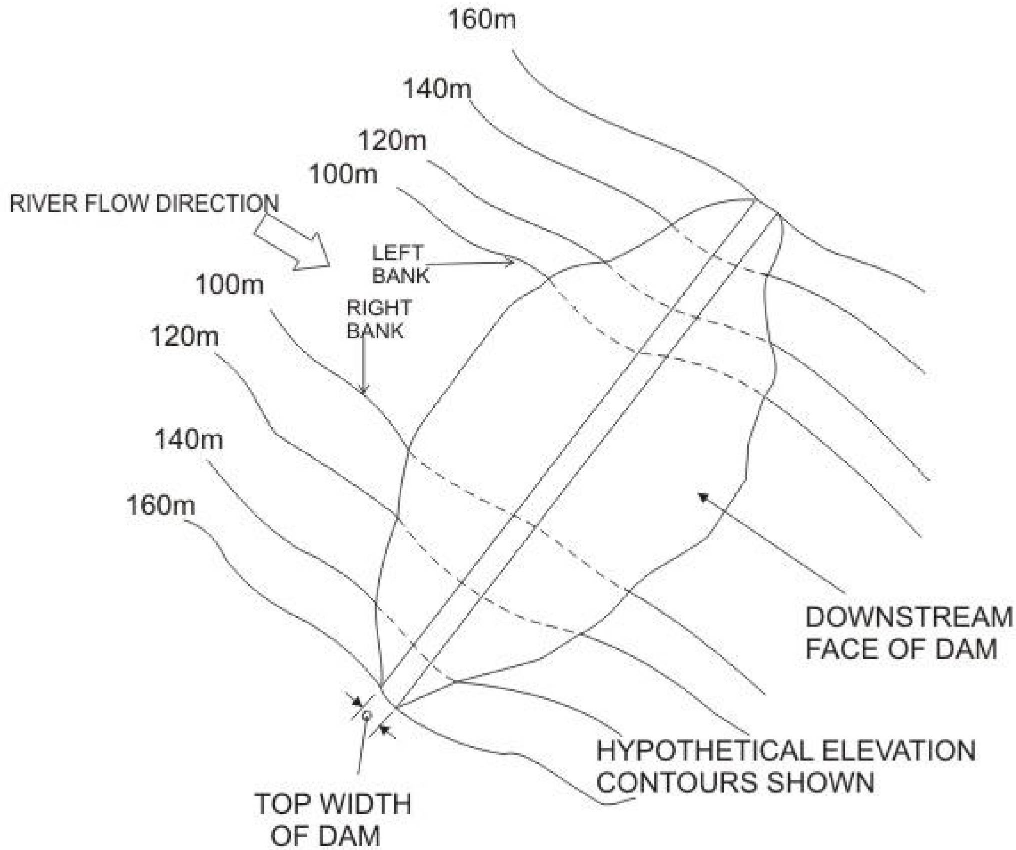


FIGURE 11. Layout of an embankment dam within a river valley

Though this is entirely satisfactory, the problem remains in providing a spillway to let out flood flows. This cannot be done through an embankment dam, since that would lead to its washout. Instead the spillway portion is usually made as a concrete gravity dam section, as shown in Figure 12 for the layout of Konar Dam on Konar river, in Jharkhand.

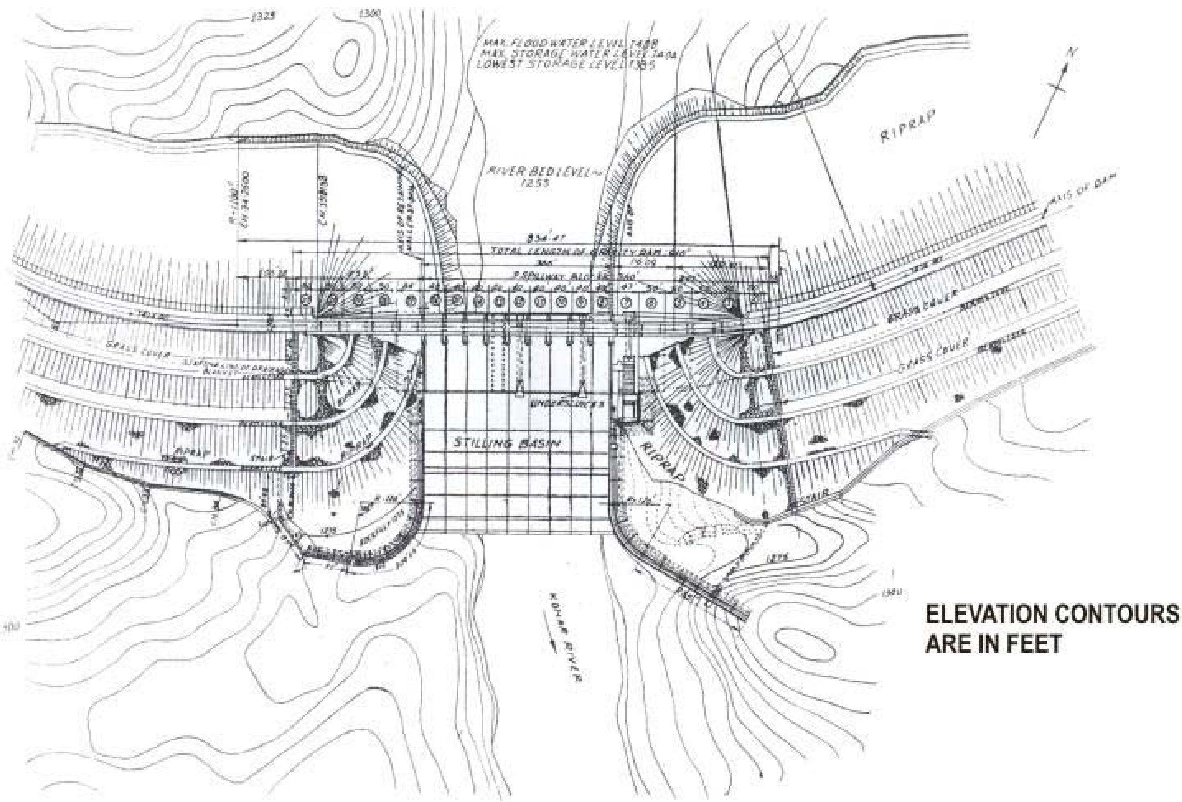


FIGURE 12. Layout of Konar Dam on River Konar, and located in Jharkhand
(Drawing courtesy: CBIP Publication 138, volume II)

Typical cross section through the embankment portion and the spillway portion are shown in Figure13.

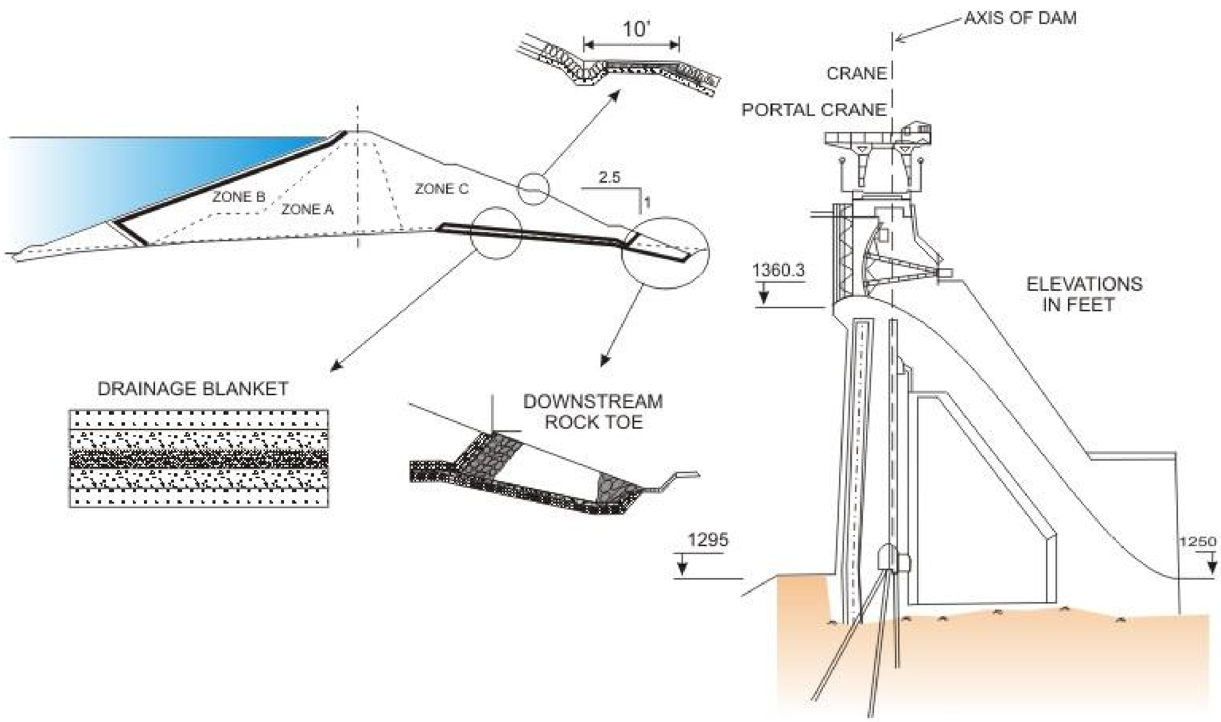
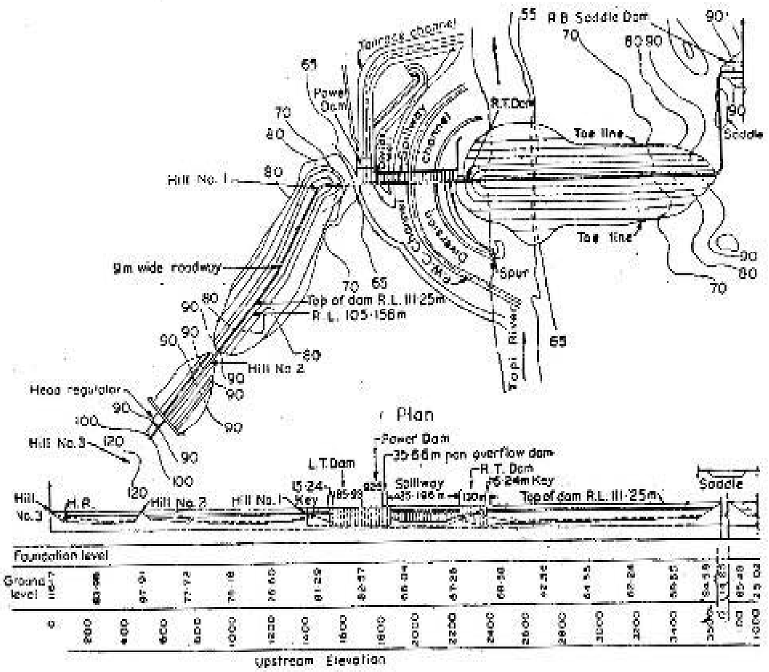


FIGURE 13. Cross sections of embankment dam and concrete gravity dam spillway of Konar dam shown in FIGURE 12

Another layout of the combination of an earthen embankment dam and a concrete spillway is shown in Figure14, for Ukai dam on Tapi river, in Gujarat.

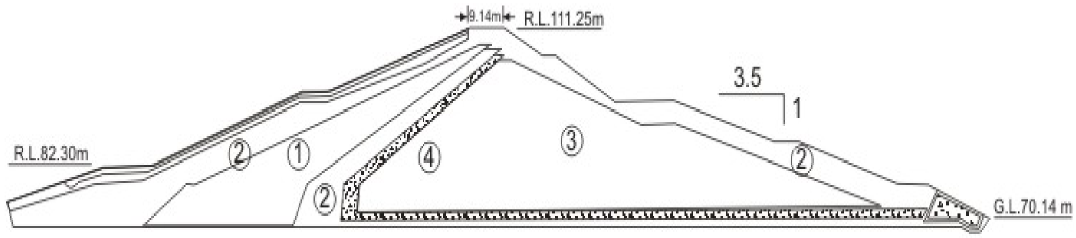


ELEVATIONS
IN METRES

FIGURE 14 . Layout of Ukai dam- showing concrete spillway and embankment dam

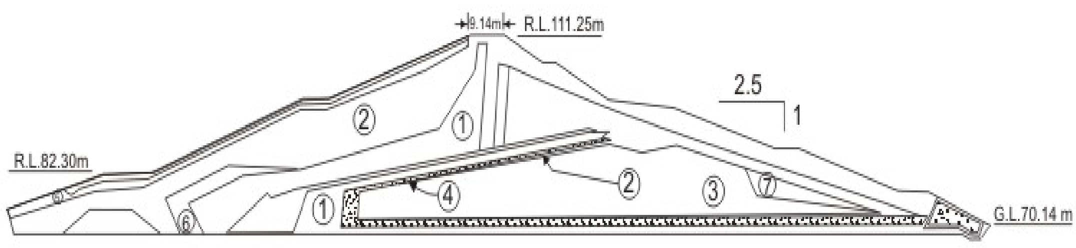
(Drawing courtesy: CBIP Publication 138, Volume I)

Typical cross sections of the two types of embankment dam section used in the project is shown in Figure 15.



Legend

1. Impervious material
2. Semi-previous material
3. Random fill (material available from excavation)
4. Filter



Legend

1. Impervious zone
2. Semi-previous zone
3. Random zone
4. Sand Filter
5. Rock zone
6. Silt zone
7. Sand zone

FIGURE 15. Typical section of embankment dams constructed in Ukai project

Both the spillway section and the power dam section conveying penstock for power houses are made of concrete and are shown in Figures 16 and 17 respectively.

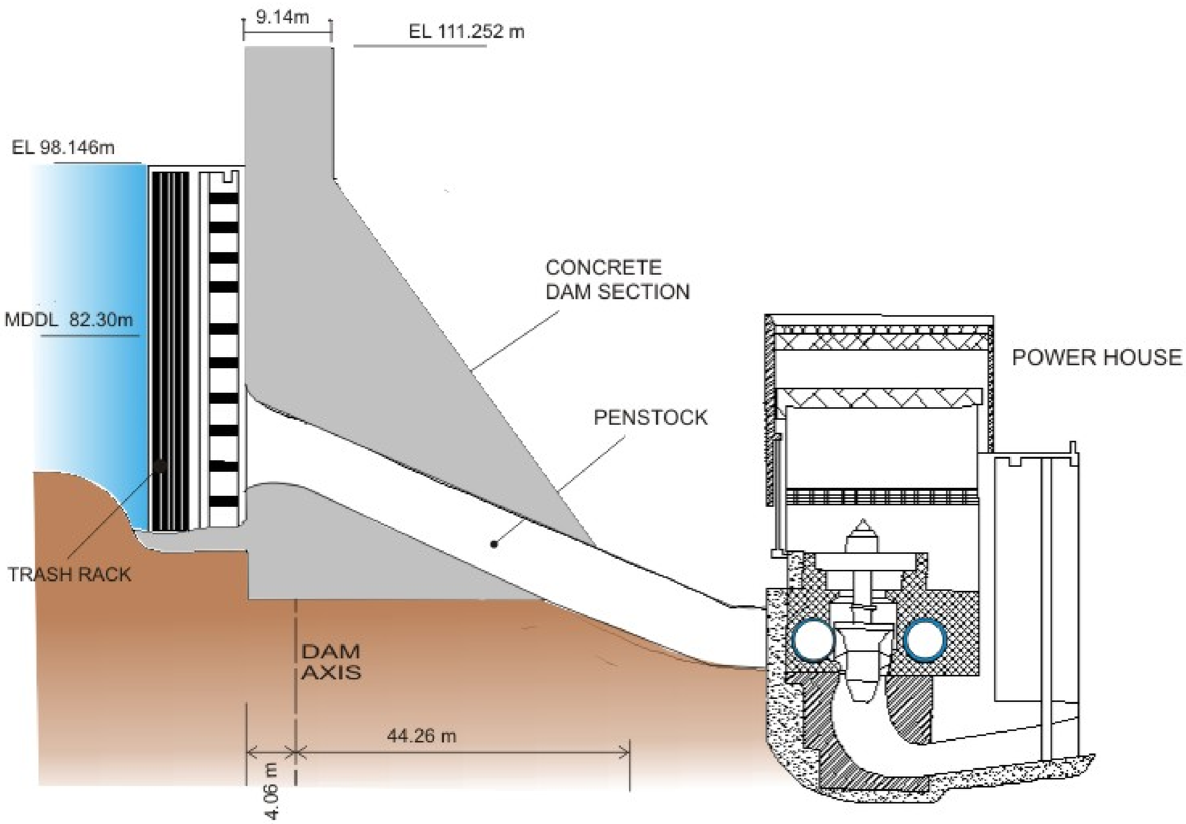


FIGURE 16. Typical power-dam section Ukai project

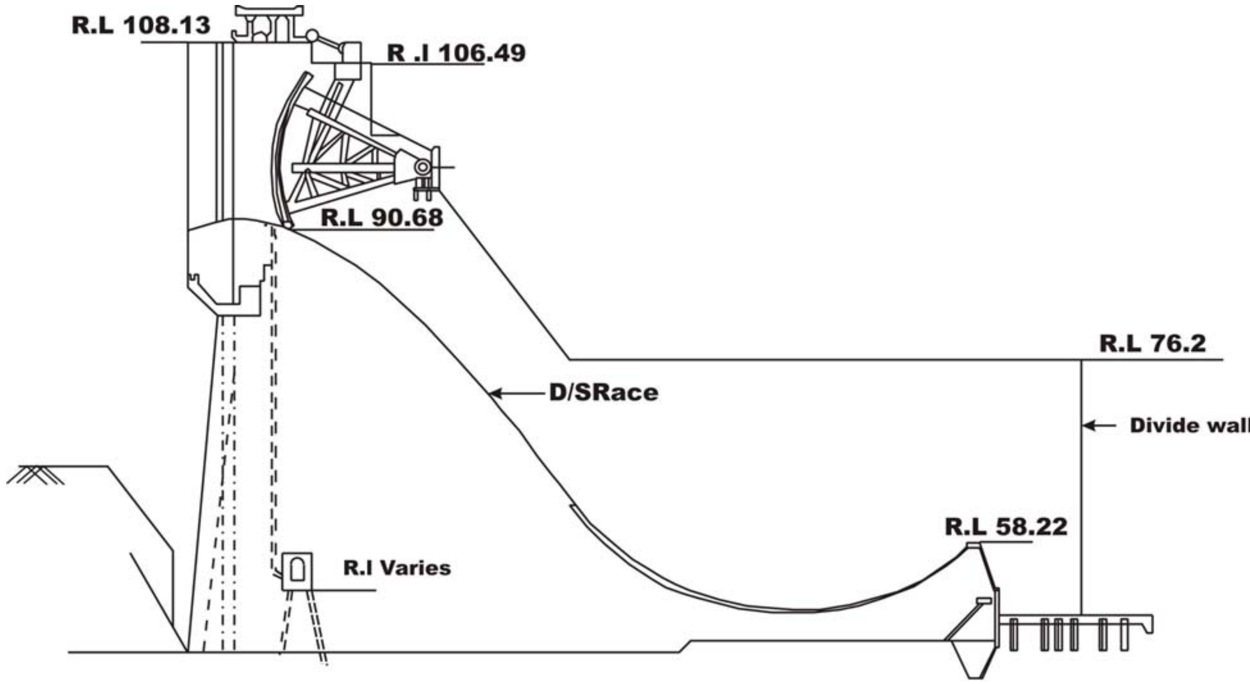


FIGURE 17. CROSS-SECTION OF SPILLWAY SECTION OF UKAI DAM PROJECT

The example of Beas Dam at Pong (Figure 18) may be cited to show a typical layout for outlet work through the embanked section (Figure19).

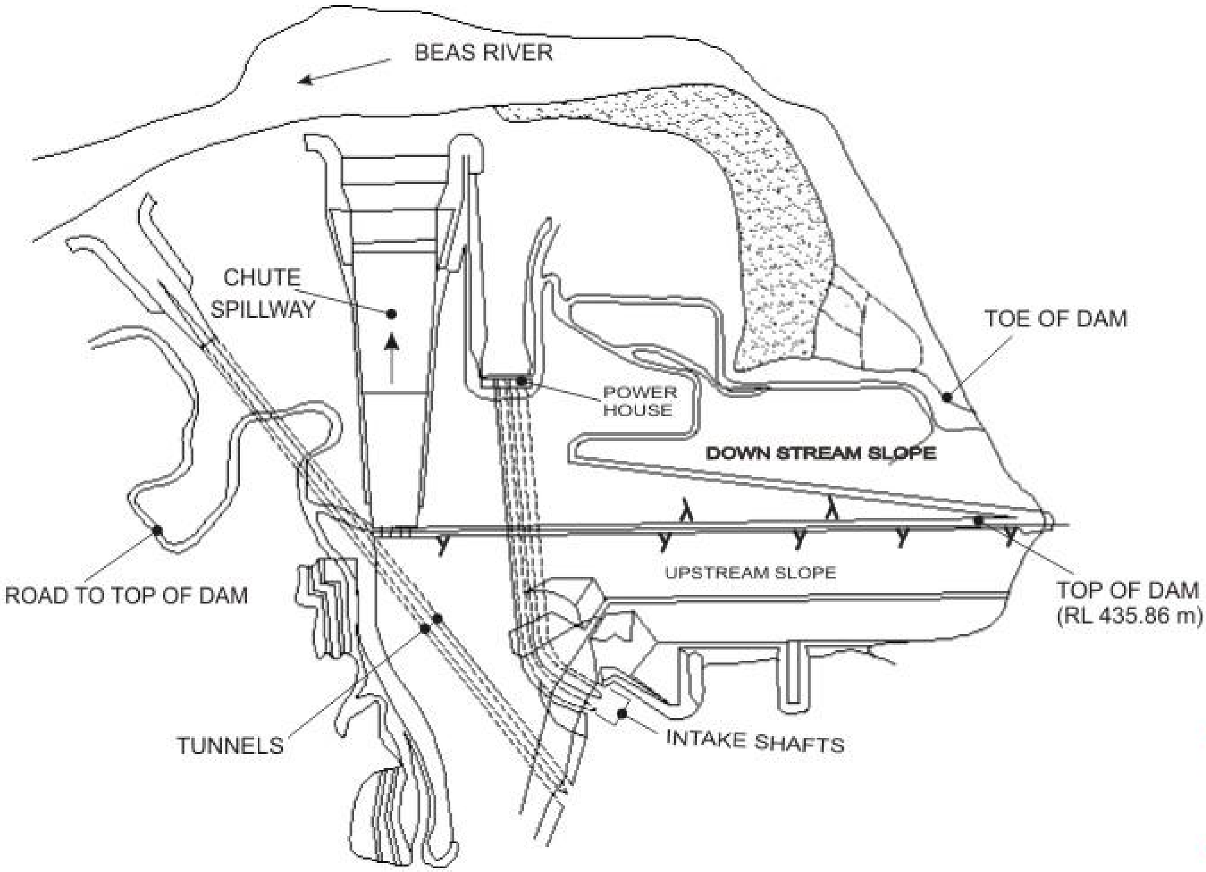
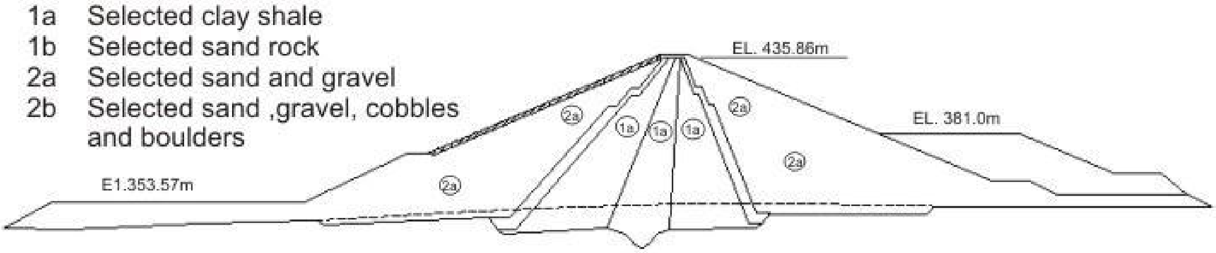
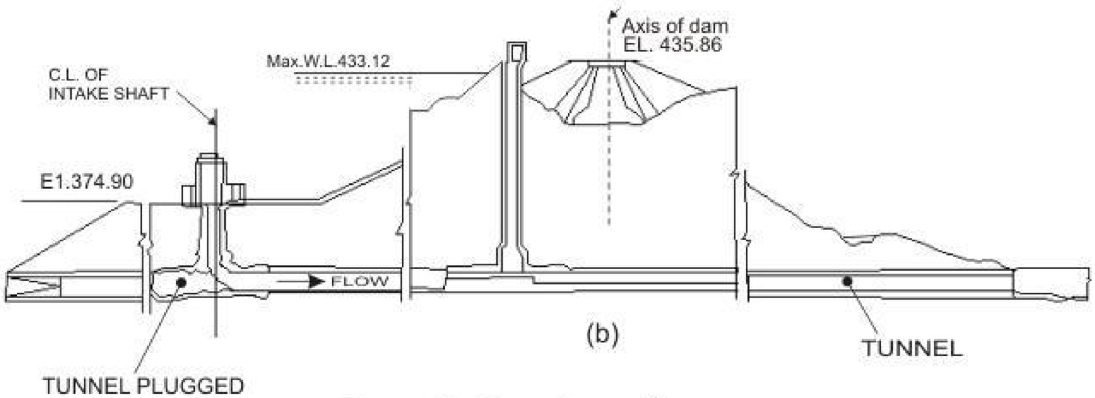


FIGURE 18 . Layout of Beas dam at Pong



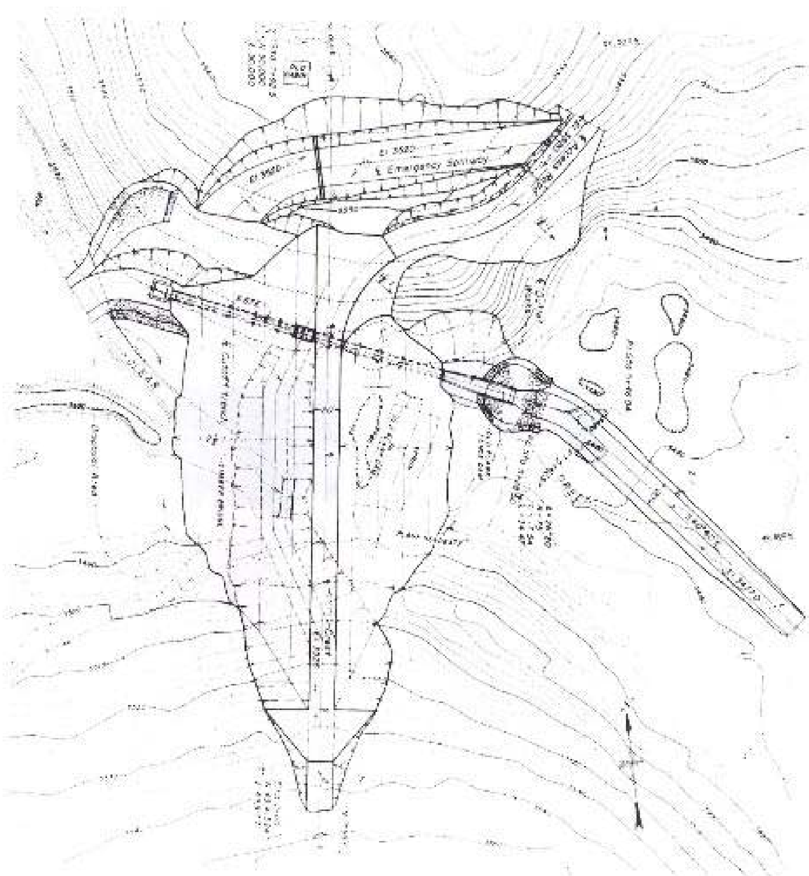
(a)



(b)

Figure 19. Beas dam at Pong
(a) Maximum section of earth dam
(b) Outlet work for power house through earth dam section

Another example is given in Figure 20 of the Wasco Dam, on Clear Creek, U.S.A, which shows an emergency spillway excavated out of left abutment hills and outlet work using rectangular duct through the bottom of the embankment dam (Figure 21).



ELEVATION CONTOURS IN FEET

FIGURE 20. General layout of Wasco Dam, Clear Creek,USA
Image courtesy: "Design of Small Dams", published by USBR

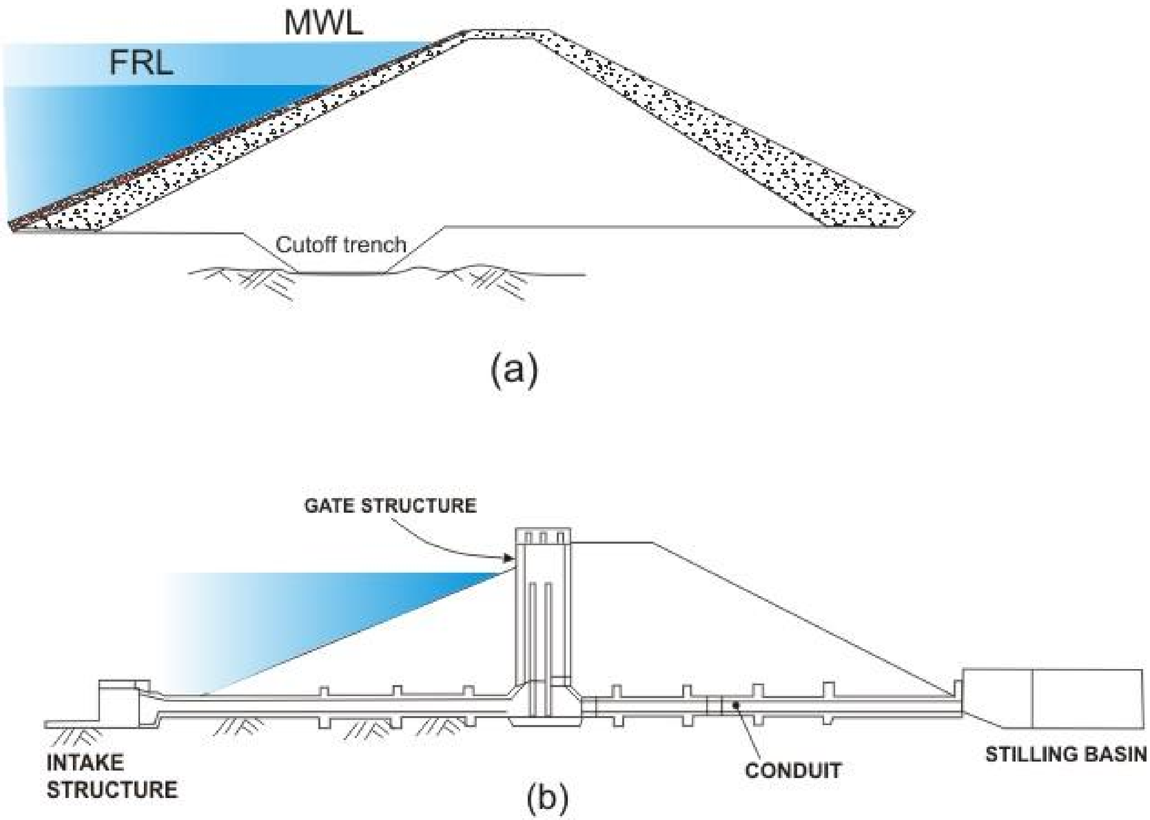


FIGURE 21. Cross sectional details of embankment dam shown in FIGURE 19.
(a) Through maximum section
(b) Through outlet work

4.7.2 Causes of failure of earth dams

The various modes of failures of earth dams may be grouped under three categories:

1. Hydraulic failures
2. Seepage failures, and
3. Structural failures

These modes of failures are explained in some detail in the following paragraphs.

Hydraulic failures

This type of failure occurs by the surface erosion of the dam by water. This may happen due to the following reasons:

1. Overtopping of the dam which might have been caused by a flood that exceeded the design flood for the spillway. Sometimes faulty operation of the spillway gates may also lead to overtopping since the flood could not be let out in time through the

spillway. Overtopping may also be caused insufficient freeboard (the difference between the maximum reservoir level and the minimum crest level of the dam) has been provided. Since earth dams cannot withstand the erosive action of water spilling over the embankment and flowing over the dam's downstream face, either complete or partial failure is inevitable (Figure 22).

2. Erosion of upstream face and shoulder by the action of continuous wave action may cause erosion of the surface unless it is adequately protected by stone riprap and filter beneath (Figure 23).
3. Erosion of downstream slope by rain wash. Though the downstream face of an embankment is not affected by the reservoir water, it may get eroded by heavy rain water flowing down the face, which may lead to the formation of gullies and finally collapse of the whole dam (Figure 24).
4. Erosion of downstream toe of dam by tail water. This may happen if the river water on the downstream side of the dam (which may have come from the releases of a power house during normal operation or out of a spillway or sluice during flood flows) causes severe erosion of the dam base. (Figure 25).

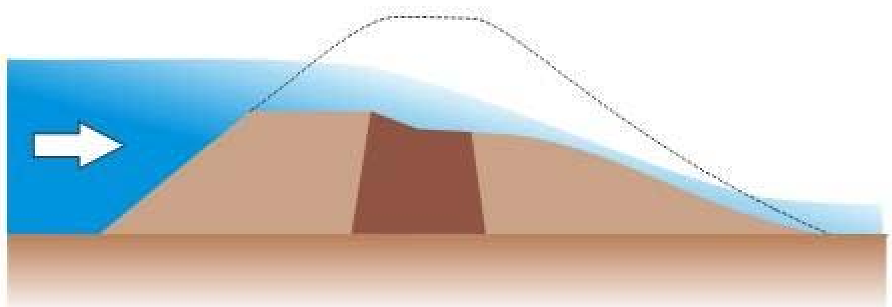


FIGURE 22. Overtopping of dam resulting in washout

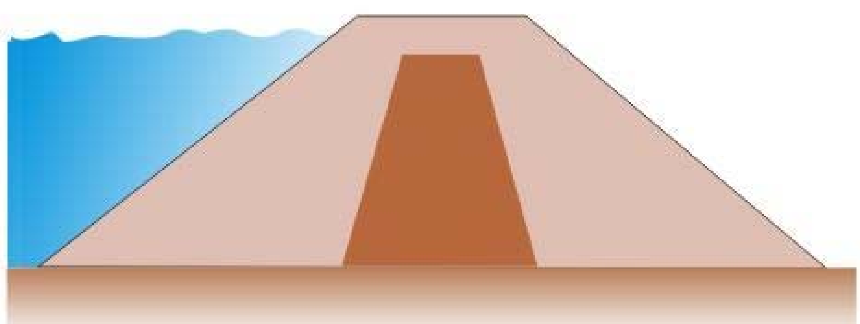


FIGURE 23. Erosion of upstream face by waves breaking on the surface

Figure 24. Scour of downstream face by impact of rain and resulting sheet flow

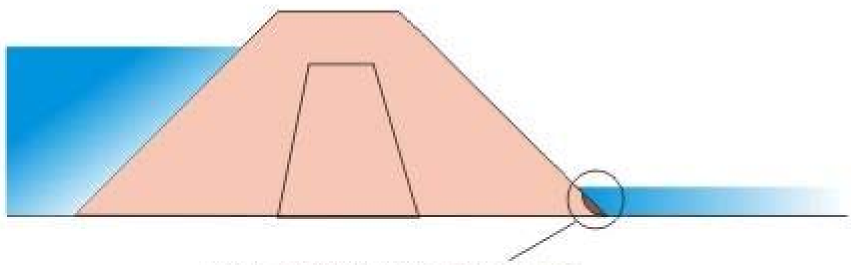


Figure 25. Scour on the downstream toe by tailwater

Seepage failures

The water on the reservoir side continuously seeps through an embankment dam and its foundation to the downstream side. Unless a proper design is made to prevent excessive seepage, it may drive down fine particles along with its flow causing gaps to form within the dam body leading to its collapse. Seepage failures may be caused in the following ways:

1. Piping through dam and its foundation: This is the progressive backward erosion which may be caused through the dam or within its foundation by the water seeping from upstream to the downstream (Figure 26)

2. Conduit leakage: This is caused due to seepage taking place by the surface of a conduit enclosed within an embankment dam (Figure 27). The seepage of water may be from the reservoir to the downstream or due to the water leaking out of the conduit through cracks that might have developed due to unequal settlement of dam or by overloading from the dam. Further, the cracking of a conduit may also be caused when the soil mass lying below it settles and the conduit is not sufficiently strong to support the soil mass lying above.
3. Sloughing of downstream face: This phenomena take place due to the dam becoming saturated either due to the presence of highly pervious layer in the body of the dam. This causes the soil mass to get softened and a slide of the downstream face takes place (Figure 28)

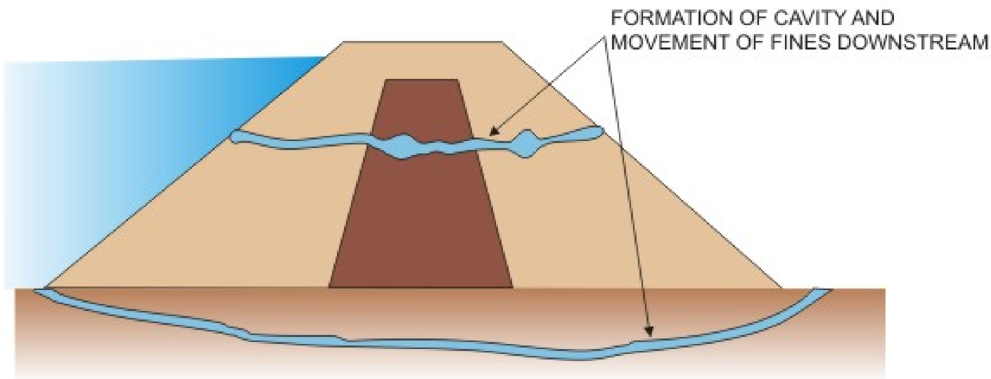


FIGURE 26. Internal erosion and piping through dam body and foundation

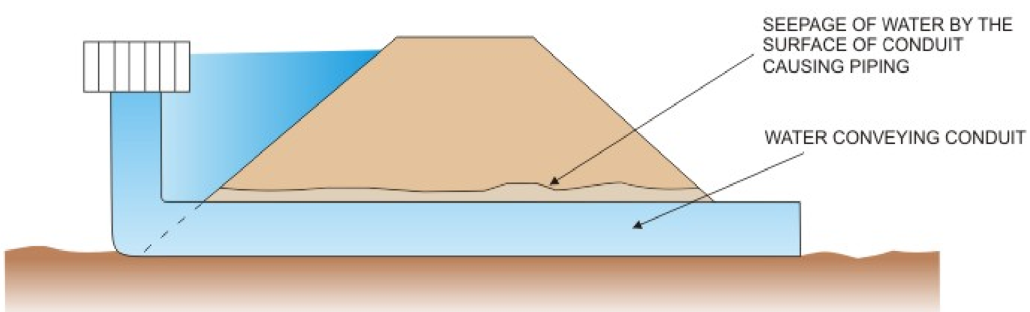


FIGURE 27. Seepage by the outer surface of conduit; may lead to progressive piping

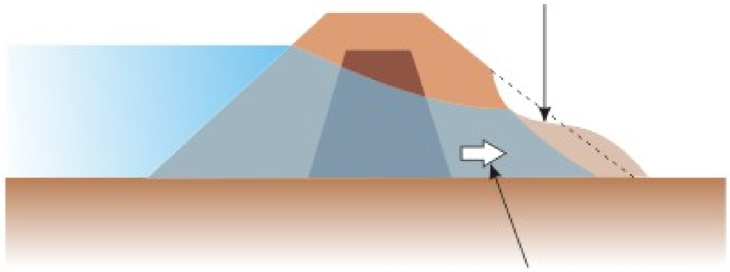


FIGURE 28. Sloughing of downstream face due to high pore water pressure

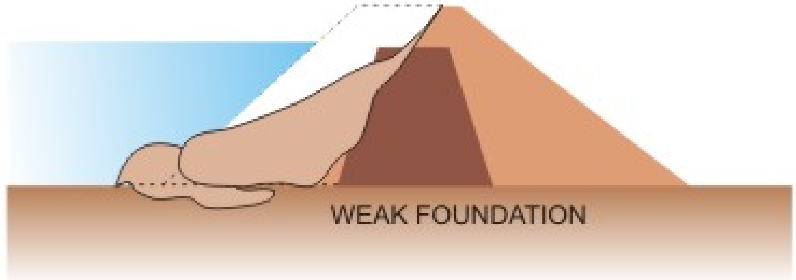
Structural failures

These failures are related to the instability of the dam and its foundation, caused by reasons other than surface flow (hydraulic failures) or sub-surface flow (seepage-failures). These failures can be grouped in the following categories:

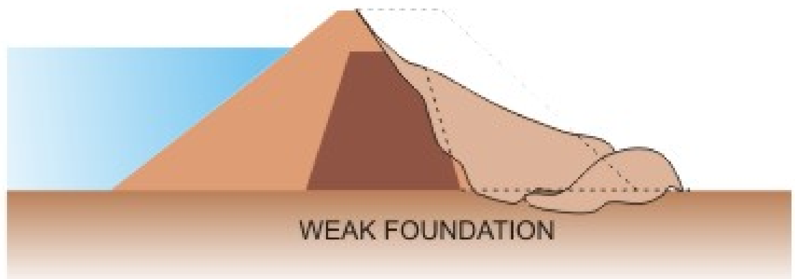
1. Sliding due to weak foundation: Due to the presence of faults and seams of weathered rocks, shales, soft clay strata, the foundation may not be able to withstand the pressure of the embankment dam. The lower slope moves outwards along with a part of the foundation and the top of the embankment subsides (Figure 29) causing large mud waves to form beyond the toe.
2. Sliding of upstream face due to sudden drawdown: An embankment dam, under filled up condition develops pore water pressure within the body of the dam. If the reservoir water is suddenly depleted, say due to the need of emptying the reservoir in expectation of an incoming flood, then the pore pressure cannot get released, which causes the upstream face of the dam to slump (Figure 30).
3. Sliding of the downstream face due to slopes being too steep: Instability may be caused to the downstream slope of an embankment dam due to the slope being too high and / or too steep in relation to the shear strength of the shoulder material. This causes a sliding failure of the downstream face of the dam (Figure 31).
4. Flow slides due to liquefaction: Triggered by a shock or a movement, as during an earthquake, some portion of the dam or foundation may destabilize due to the phenomena called liquefaction. Here, even cohesionless soil cannot drain quickly enough as the movements are so sudden that the rate of extra loading on the soil becomes greater than the rate of drainage of the seepage water out of the soil. This causes excess pore water pressure to develop, where both the effective stress and the strength decrease. Under circumstances when the effective stress

drops to zero, which means the soil loses all its shear strength, it behaves like a dense liquid and slides down, and the dam slumps.

5. Damage caused by burrowing animals or water soluble materials: some embankment dams get damaged by the burrows of animals which causes the seepage water to flow out more quickly, carrying fine material along with. This phenomena consequently leads to piping failure within the body of the dam, finally leading to a complete collapse. Similarly, water soluble materials within the body of the dam gets leached out along with the seepage flow causing piping and consequent failure.
6. Embankment and foundation settlement: Excess settlement of the embankment and/or the foundation causes loss of free board (Figure 32). The settlement may be more in the deeper portion of the valley, where the embankment height is more.



FAILURE OF UPSTREAM SLOPE



FAILURE OF UPSTREAM SLOPE

FIGURE 29. Instability of upstream or downstream slopes caused by failure of weak foundations

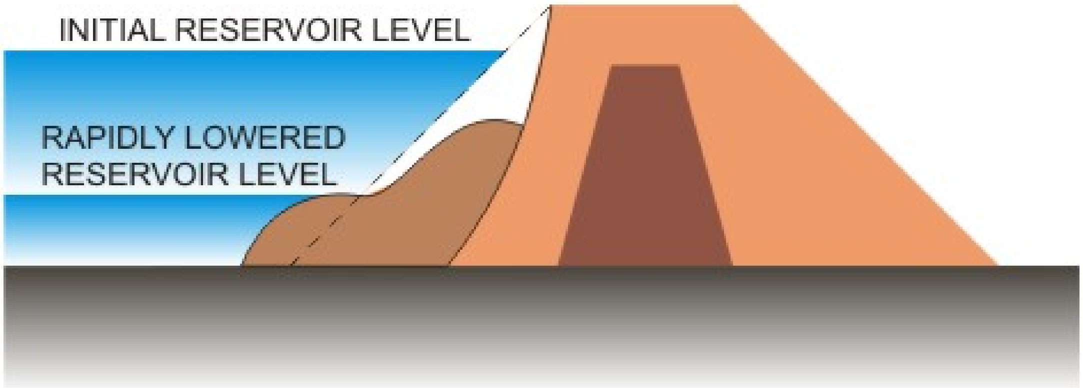


FIGURE 30. Upstream slope failure due to rapid drawdown

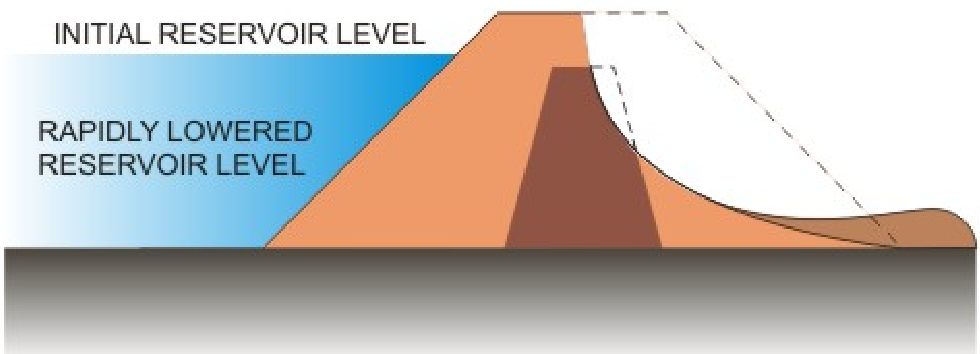


FIGURE 31. Downstream face too steep unable to be resisted by soil shear strength

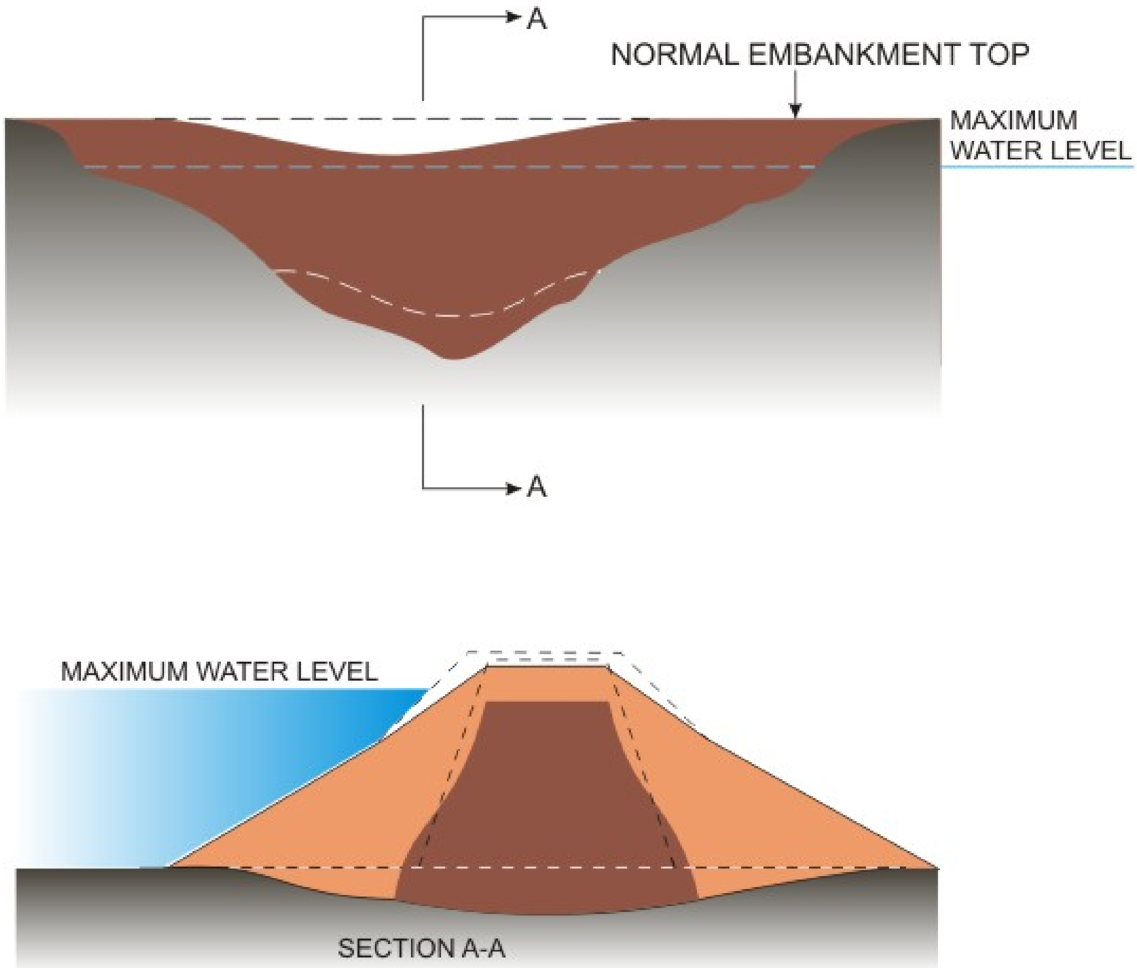


FIGURE 32. Excessive settlement of dam and foundation

4.7.3 Design of earth dams

As may be observed from the modes of failures of earth dams, there are three main types, which have to be prevented in a safe design of embankment dam. These are:

1. Safety against hydraulic failures due to overtopping , rain cuts, wave action or tail water.
2. Safety against seepage failures due to internal erosion and development of pore pressure due to insufficient drainage.
3. Safety against structural instability.
4. Special design requirements.

The features in design that must be incorporated in order to take care of the above criteria are elaborated in the Bureau of Indian Standard codes IS: 12169-1987 " Criteria for design of small embankment dams" and IS: 8826-1978 "Guidelines for design of large earth and rockfill dams" from which the following have been summarized:

1. Safety against hydraulic failures

i) Sufficient spillway capacity should be provided to prevent overtopping of embankment during and after construction.

ii) The freeboard should be sufficient enough to prevent overtopping by waves. Bureau of Indian Standards code IS: 10635-1993" Free board requirement in embankment dams-guidelines" provide methods to determine freeboard. However, for simplicity, the methods to determine free board for concrete dams may be used.

iii) Sufficient height of dam has to be provided to take care of any future settlement and consequent loss of free board

iv) Upstream slope shall be protected by riprap, which is a layer of rock fragments, against wave action. The riprap shall have to be provided from an elevation 1.5m or half of maximum wave height at Minimum Draw Down Level (MDDL), whichever is more below MDDL to the top of the dam. Figure 32 illustrates the recommended methods of terminating the ripraps at the lowest level. As may be observed from Figure 32, the riprap or pitching should be underlain with two layer of filters to prevent the water from eroding washing out of the underlying embankment material.

v) The downstream slope of the embankment should be protected by turfing, that is growing of grass on the surface against erosion by rain wash. A system of open passed drains (chutes) along the sloping surface terminating in longitudinal collecting drains at the junction of berm and slope shall have to be provided at 90m centre-to-centre to drain the rain water. Drains may be formed of stone pitching or with pre-cast concrete sections (Figure 33)

vi) Downstream slope of embankment, if affected, upon by tail water, should be protected by placing riprap of 300mm thickness over proper filter layers up to 1m above the maximum tail water level.

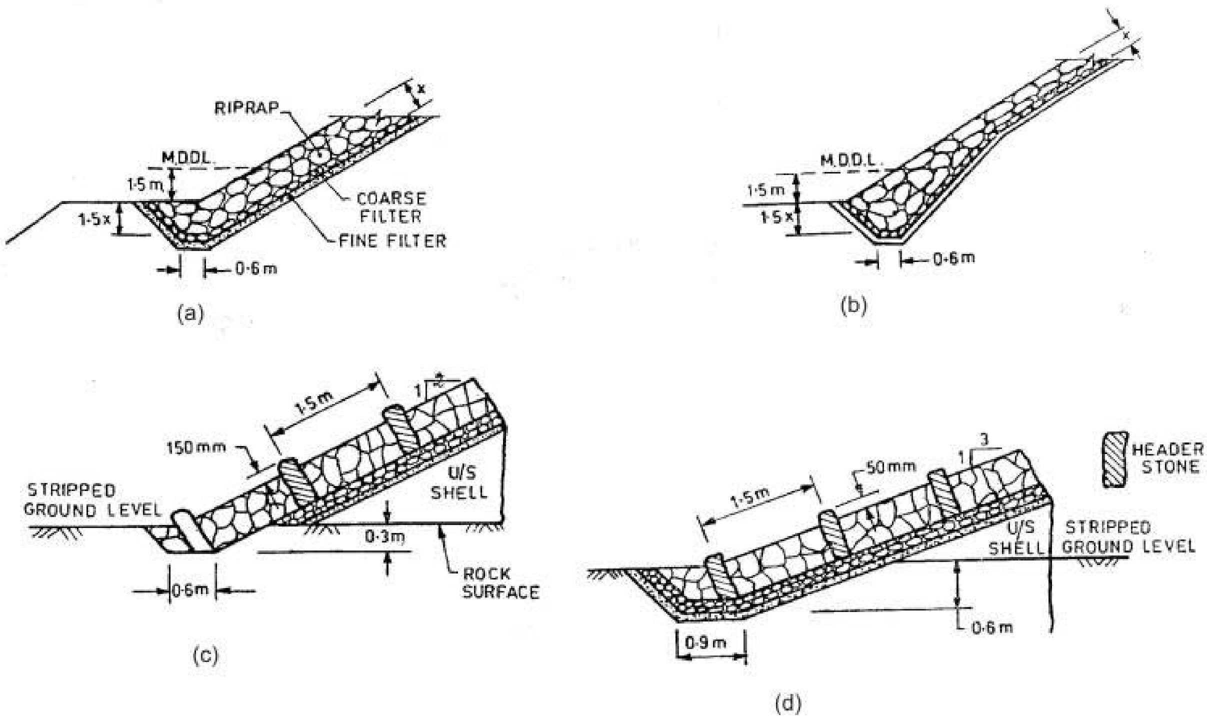


FIGURE 33. Pitching of rip rap (a) With berm below MDDL ; (b) Without berm below MDDL ; (c) Terminating at rock surface ; (d) Terminating at stripped ground level

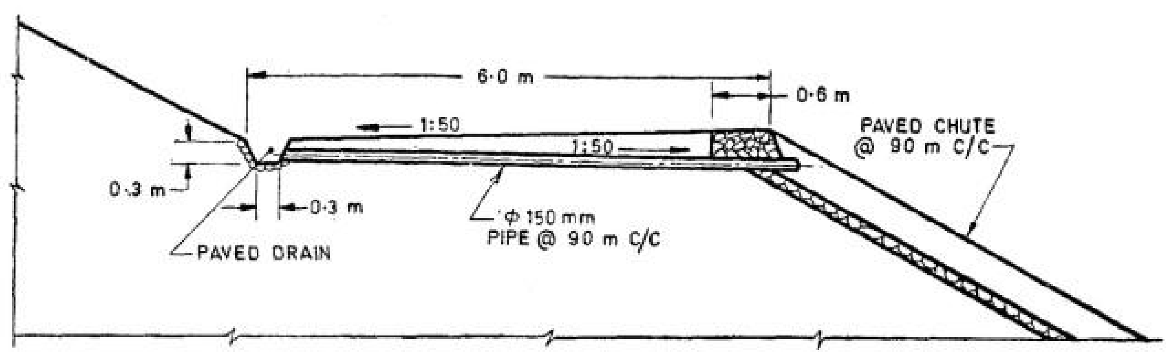


FIGURE 34. Typical downstream face surface drainage arrangement

2. Safety against seepage failures

i) Amount of seepage water passing through the embankment and foundation should be limited so as to control piping, erosion, sloughing and excessive loss of water. Seepage control measures are required to control seepage through the dam have to be made according to the recommendations provided in the Bureau of Indian Standards code IS: 9429-1999 "Drainage systems for earth and rockfill dams-code of practice", which has been discussed in section 4.7.4. Design for control of seepage through foundation have to be made as per IS: 8414-1977 "Guidelines for design of under-seepage control measures for earth and rockfill dams". Salient features of these are also summarized in section 4.7.4.

ii) The phreatic line should be well within the downstream face of the dam to prevent sloughing. Methods to estimate the location of the phreatic line is discussed in section 4.7.5

iii) Seepage water through the dam or foundation should not be so high that may cause removal of fine materials from the body of the dam leading to piping failures.

iv) These should not be any leakage of water from the upstream to the downstream face, which may occur through conduits, at joints between earth and concrete dam sections, or through holes made by burrowing animals.

3. Safety against structural instability

i) The slopes of the embankment on the upstream and downstream should be stable under all loading conditions. Embankment slopes have to be designed in accordance with Bureau of Indian Standards Code IS: 7894-1975 "Code of practice for stability analysis of earth dams", which has been discussed separately in section 4.7.6.

ii) The embankment slopes should also be flat enough so as not to impose excessive stresses on the foundation, and as much, be within the permissible limits of the shear strength of the material

4. Special design requirements

According to IS: 8826-1978, an embankment dam should, in addition to the basic design requirements detailed above, has to satisfy the criteria of control of cracking, stability in earthquake regions, and stability at junctions. These are explained in the following paragraphs:

i) Control of cracking

Cracking of the impervious core results into a failure of an earth dam by erosion, piping, breaching, etc. Cracking occurs due to foundation settlement and/or differential movements within the embankment. Differential moments may occur due to unsuitable or poorly compacted fill materials, different compressibility and stress-strain characteristics of the various fill materials; and variation in thickness of fill over irregularly shaped or steeply inclined abutments. In order to prevent cracking of the core material, the following measures may be adopted:

a) Use of plastic clay core and rolling the core material at slightly more than optimum moisture content. In case of less plastic clay, 2 to 5 percent bentonite of 200 to 300 liquid limit may be mixed to increase the plasticity.

b) Use of wider core to reduce the possibility of transverse or horizontal cracks extending through it.

c) Careful selection of fill materials to reduce the differential movement. To restrict the rockfill in lightly loaded outer casings and to use well graded materials in the inner casings on either side of the core.

d) Wide transition zones of properly graded filters of adequate width for handling drainage, if cracks develop.

e) Special treatment, such as preloading, pre-saturation, removal of weak material, etc, to the foundation and abutment, if warranted.

f) Delaying placement of core material in the crack region till most of the settlement takes place.

g) Arching the dam horizontally between steep abutments.

h) Flattening the downstream slope to increase slope stability in the event of saturation from crack leakage.

i) Cutting back of steep abutment slopes.

ii) Stability in earthquake regions

Embankment dams which are located in earthquake affected regions are likely to be subjected to additional stresses and deformation on account of the sudden vibrations generated by seismic forces. The following factors need to be considered while designing embankment dams in earthquake prone regions:

a) Stability of the slopes should be checked under the additional seismic forces. This has to be done according to IS: 7894-1985 “Code of practice for stability analysis of earth dams”.

b) The settlement of loose or poorly compacted fill or foundation material may lead to loss of free board. Hence, an additional free board may have to be provided to take care of this situation.

c) Cracking of the impervious core leading to possible failure of the dam. In order to take care of this situation, the measures recommended for the control of cracking has to be adopted. In addition, provisions shall have to be made for discharging the maximum anticipated leakage rapidly. For this purpose, downstream zones of large quarried rock or screened gravels and cabbles have to be used. The impervious core may have to be made thicker for resisting the piping action. The top of the dam should be made thicker by increasing the crest width or by using flatter slopes at the top than would be required for dams in non-seismic regions, so as to increase the path of seepage through cracks.

d) Liquefaction of deposits of loose sand in the foundation of the dam, causing cracking, sliding, or actual horizontal movement of the dam. In order to take care of this situation, the foundation should be made as compact as possible, or re-compacted.

iii) Stability at junctions

Junctions of embankment dams with foundation abutments, concrete overflow and non-overflow dam sections, and outlets need special attention with reference to one or all of the following criteria:

a) Good bond between embankment dam and foundation
b) Adequate creep length at the contact plane
c) Protection of embankment dam slope against scouring action, and
d) Easy movement of traffic.

Earth dams may be founded on soil overburden on rock. For foundations on soils or non-rocky strata, vegetation like bushes, grass roots, trees, etc. should be completely removed. The soil containing, organic material or dissoluble salt, should also be completely removed. After removal of these materials, the foundation surface should be moistured to the required extent and adequately rolled before placing embankment material. For rocky foundation, the surface should be cleaned of all loose fragments including semi-detatched and over-hanging surface blocks of rocks. Proper bond should be established between the embankment and the rock surface so prepared.

For junction of earth dams with concrete dam block, the concrete block has to be inclined at 1V:0.5H to meet the impervious core of the earth dam. A wider impervious core and thicker transitions have to be provided at the abutment contacts to increase the length of path of seepage and to protect against erosion.

At the junction of embankment dam with outlet works, proper bond have to be provided. Staunching rings should be added to the outside of the outlet conduits in the impervious zone, at intervals, so as to increase the path of percolation along the contact. Backfilling of the trench for the outlet conduit should be done with concrete up to the top of the rock surface and the portion of the trench above the rock level should be re-filled with impervious materials compacted with moisture content about 2 percent more than optimum.

4.7.4 Seepage control measures in embankment dam and foundation

One of the basic requirements for the design of an earth or rockfill dam is to ensure safety against internal erosion, piping and excessive pore pressure in the dam. A suitably designed drainage system is therefore essential to satisfy these requirements. The seepage of reservoir water through the body of the dam or at the interfaces of the dam with the foundation or abutment creates two main problems, apart from causing excessive water loss and thereby reducing usable storage of reservoir:

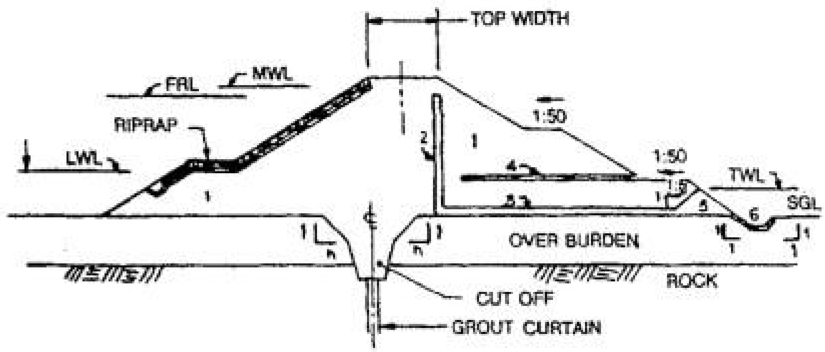
1. Seepage force causing excessive water loss
2. Piping

Inspite of taking all measures in design as well as construction to minimize seepage, it does take place either through the body of the dam or at interfaces. Whatever may be quantum of seepage, if it is not safely drained away from the toe of embankment dam into nearby drainage, valley, etc, it may lead to failure or heavy damage to the embankment, by way of slips of slopes and/or development of internal erosion leading to formation of sink holes, boiling, settlement, etc, besides creating unfriendly environment on downstream faces and areas of embankment dams.

The drainage system should be so devised that it tackles the problems mentioned in section 4.7.4. The design is mostly governed by type and permeability of base materials as well as filter materials, water depth in reservoir, topographical features of dam site, etc. The conventional types of seepage control and drainage features generally adopted for the embankment dam are:

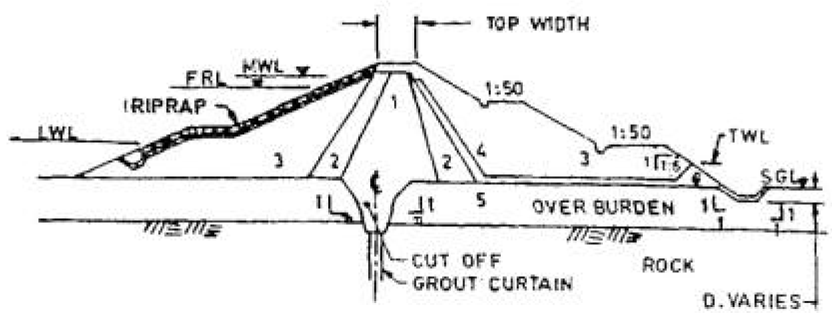
a) Impervious core,
b) Inclined/vertical filter with horizontal filter,
c) Network of inner longitudinal drain and cross drains,
d) Horizontal filter,
e) Transition zones/transition filters,
f) Intermediate filters,
g) Rock toe, and
h) Toe drain.

The drainage system may comprise of either one or a combination of more than one of these drainage features, and typical sections are shown for homogeneous dams, in Figure 35 and for zoned dams, in Figure 36. The functions of each of the components are described in the following paragraphs.



Zones:
1. Impervious fill material
2. Vertical filter
3. Horizontal filter at base
4. Intermediate filter
5. Rock toe
6. Toe drain

FIGURE 35. Section of homogenous dam showing seepage control features



Zones:
1. Central impervious core
2. Transition zone / Transition filter
3. Shell / Rockfill
4. Inclined filter
5. Horizontal filter
6. Rock toe
7. Toe drain

FIGURE 36. Section of zoned dam showing seepage control features

Inclined/Vertical Filter

Inclined or vertical filter abutting downstream face of either impervious core or downstream transition zone is provided to collect seepage emerging out of core/transition zone and thereby keeping the downstream shell relatively dry. In the eventuality of hydraulic fracturing of the impervious core, it prevents the failure of dam by piping.

Horizontal Filter

It collects the seepage from the inclined/vertical filter or from the body of the dam, in the absence of inclined/vertical filter, and carries it to toe drain. It also collects seepage from the foundation and minimizes possibility of piping along the dam seat.

Inner Longitudinal and Inner Cross Drains

When the filter material is not available in the required quantity at reasonable cost, a network of inner longitudinal and inner cross drains is preferred to inclined/vertical filters and horizontal filters. This type of drainage feature is generally adopted for small dams, where the quantity of seepage to be drained away is comparatively small. A typical arrangement of longitudinal and cross drains is shown in Figure 37.

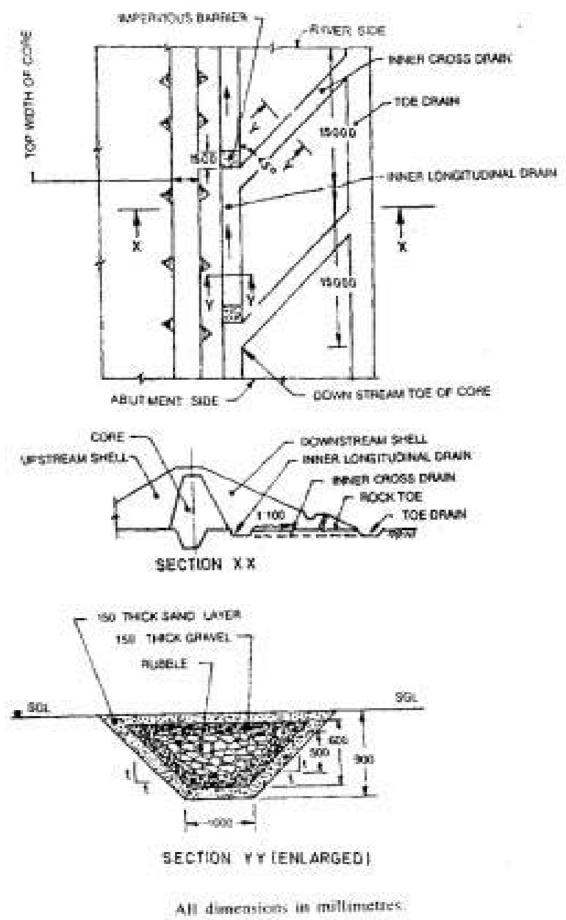


FIGURE 37. Typical arrangement of inner longitudinal and inner cross drains

Transition Zones and Transition Filters

Transition zones/filters in earth and rockfill dams in the upstream and downstream shells are necessary, when the specified gradation criterion is not satisfied between two adjacent zones. When such zones/filters are placed on either side of the impervious core, they help to minimize failure by internal piping, cracking, etc, that may develop in the core or by migration of fines from the core material.

The filter material used for drainage system shall satisfy the following criteria:

a) Filter materials shall be more pervious than the base materials;
b) Filter materials shall be of such gradation that particles of base material do not totally migrate through to clog the voids in filter material; and
c) Filter material should help in formation of natural graded layers in the zone of base soil adjacent to the filter by readjustment of particles.

Horizontal Filters at Intermediate Levels

Horizontal filter layers at intermediate levels are sometimes provided in upstream and downstream shells, to reduce pore pressures during construction and sudden drawdown condition and also after prolonged rainfall (see Figure 38).

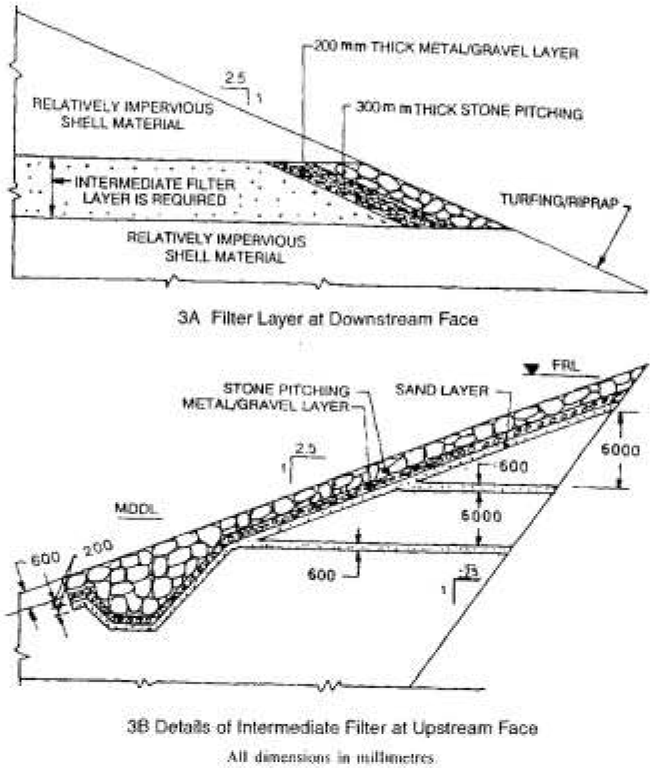


FIGURE 38. Horizontal filter at intermediate levels;
(a) On downstream, and (b) On upstream

The filter layers should be extended upto the outer slopes of the embankment so as to drain out the collected water. These filter layers should not be connected with inclined or vertical filters. A minimum space of 2.0 m or more, should be kept between the face of inclined/vertical filter and downstream intermediate filter. The material of the filter layers should be protected at exposed faces as shown in Figure 38. Details are shown in Figure 39.

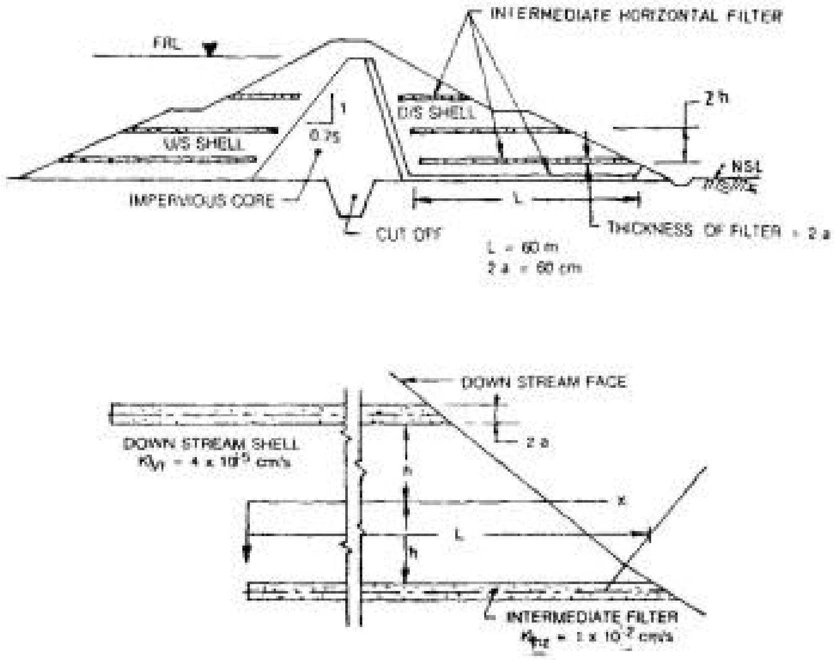


FIGURE 39. Thickness of horizontal and inverted filters

Rock Toe

The principal function of the rock toe is to provide drainage. It also protects the lower part of the downstream slope of an earth dam from tail water erosion. Rock available from compulsory excavation may be used in construction of the rock toe. Where this is not possible and transportation of rock is prohibitively costly, conventional pitching should be used for protecting the downstream toe of the dam. The top level of the rock toe/pitching should be kept above the maximum tail water level (TWL). In the reach where the ground level at the dam toe is above the maximum tail water level, only conventional pitching should be adopted. The top of such pitching should be kept 1.0 m above the top of horizontal filter, or stripped level, whichever is higher. A zone of coarse filter should be introduced between the rockfill/ pitching and the fine filter. A combination of partial rock toe and pitching may also be considered to effect economy.

Details of rock toe/pitching protection and toe drains are illustrated for various combination of Tail Water Level (TWL) and stripped Ground Level (SGL).

1. Rock toe when TWL is higher than SGL (Figure 40)
2. Pitching when TWL is higher than SGL (Figure 41)
3. Rock toe + pitching when TWL is higher than rock toe (Figure 42)
4. Pitching when SGL is above TWL (Figure 43)
5. Pitching and lined toe drain (Figure 44)

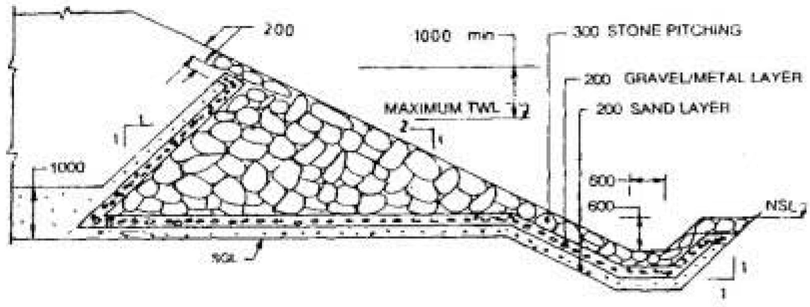
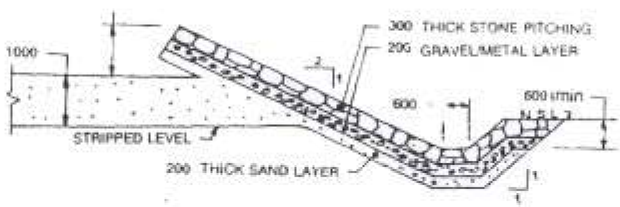
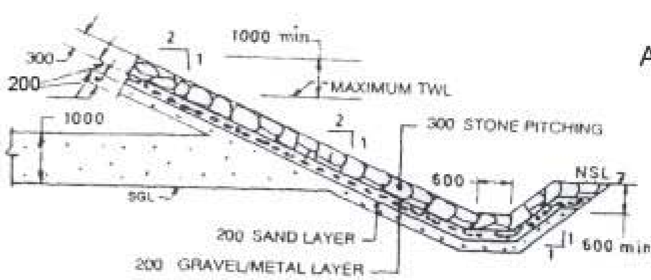


FIGURE 40. Details of rock toe protection with toe drain where TWL is higher than SGL



All dimensions in mm

FIGURE 41. Details of pitching with toe drain where TWL is higher than SGL



All dimensions in mm

FIGURE 42. Details of rock toe and pitching with toe drain where TWL is higher than rock toe

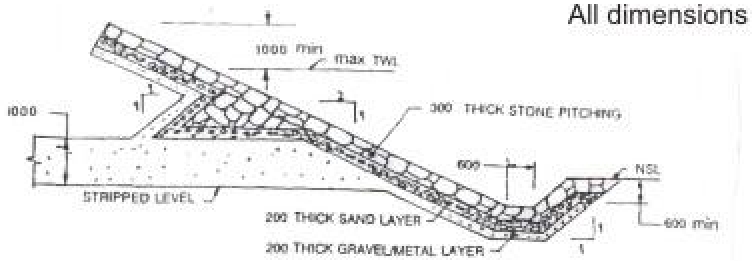


FIGURE 43. Details pitching with toe drain where SGL is above TWL

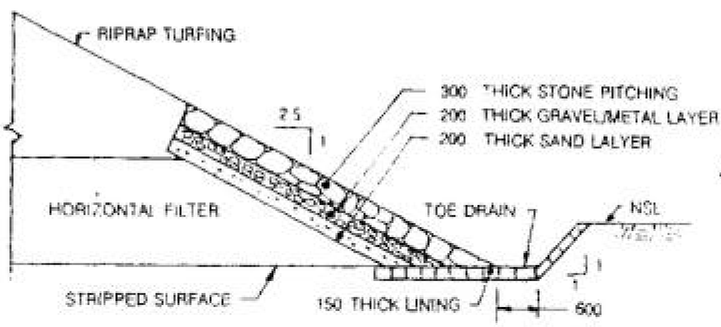


FIGURE 44. Details of pitching and lined toe drain

Toe Drain

Toe drain is provided at the downstream toe of the earth/rockfill dam to collect seepage from the horizontal filter or inner cross drains, through the foundation as well as the rain water falling on the face of the dam, by suitable means according to site conditions. Additional longitudinal drain and cross drains connected with the toe drain are sometimes provided where outfall conditions are poor. It is preferable to provide the toe drain outside the toe of rock toe, to facilitate visual inspection. The section of the toe drain should be adequate for carrying total seepage from the dam, the foundation and the expected rain water.

Details of the above measures required for seepage control within the body of an embankment dam may be had from the Burean of Indian Standards Code IS: 9429-1999 “Drainage system for earth and rockfill dams-code of practice”.

For the control of seepage below the dam, through the foundation, the Bureau of Indian Standards Code IS: 8414-1977 “Guidelines for design of under-seepage control measures for earth and rockfill dams mention a number techniques. The provision of seepage control in the foundations, as in the body of the dam, is required to control the loss of water to an amount compatible with the purpose of the project, and the elimination of the possibility of a failure of the structure by piping. Many dams have been in successful service for decades in spite of losses of water. Therefore, the first step in rational design of seepage control measures is to estimate the largest quantity of water that may escape if no attempt is made to intercept percolation through the foundation. In many instances, it would be found that interception of the most conspicuously pervious zones would be sufficient. Sometimes, the reservoir bottom may have to be made impervious to reduce the amount of water seeping into ground. In addition, relief wells may be used at downstream to release the building up of excess pore pressure. These methods are described in the following paragraphs.

Positive Cutoff Trench

The positive cutoff trench (Figure 45) consists of an impervious fill placed in a trench formed by open excavation into an impervious stratum. Grouting of the contact zone of the fill and the underlying strata constitutes an integral part of the positive cutoff. Pockets of such size that compaction equipment cannot be operated and pot holes with overhangs should be filled with concrete.

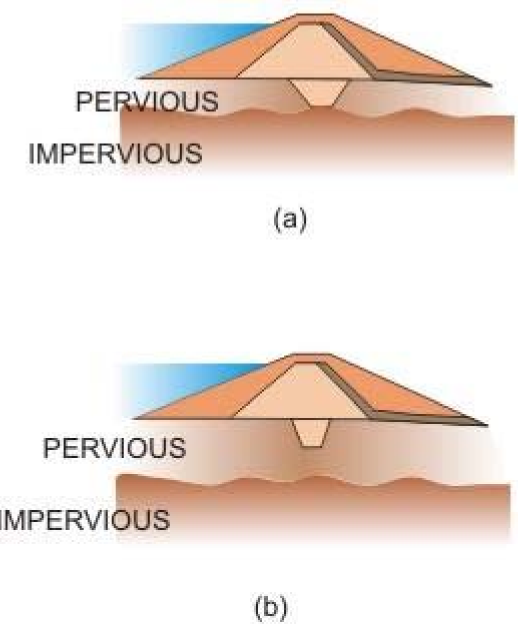
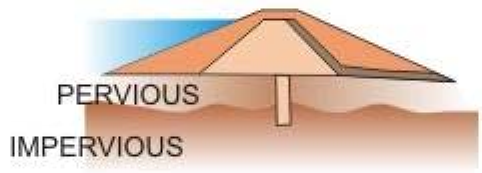


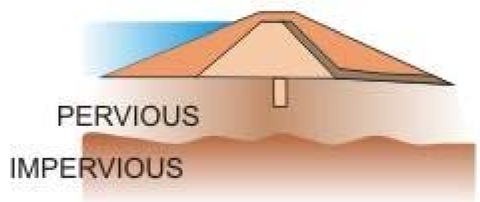
FIGURE 45. Cut off trenches
(a) Positive cut-off
(b) Partial cut-off

Concrete Diaphragm

A single diaphragm or a double diaphragm may also be used for seepage control (Figure 46). Concrete cutoff walls placed in slurry trench are not subject to visual inspection during construction, therefore require special knowledge, equipment and skilled workmen to achieve a satisfactory construction.



(a)



(b)

FIGURE 46. Concrete diaphragm
(a) Complete
(b) Partial

Grout Curtain

Grout Curtain in Pervious Soils: Grouted cutoffs are produced by injection, within the zone assigned to the cutoff, of the voids of the sediments with cement, clay, chemicals, or a combination of these materials. An essential feature of all grouting procedures is successive injection, of progressively finer pockets in the deposit. Inasmuch as grout cannot be made to penetrate the finer materials as long as more pervious pockets are available, the coarser materials are treated first, usually with the less expensive and thicker grouts, whereupon the finer portions are penetrated with less viscous fluids.

Grout Curtain in Rock: Grout curtain in rock admit of routinized treatment if the purpose is only to block the most pervious zones. These can be treated by cement grout with suitable admixtures. Concentrated seepage would generally develop at the base of the positive cutoff. This zone is particularly vulnerable when a narrow base width is used for the cutoff trench in relation to the height of the dam. The depth of the grouted zone would be dependent on the nature of the substrata and their vulnerability to subsurface erosion.
Details about the method of grouting may be had from Bureau of Indian Standards code IS: 11293(Part1)-1985 “Guidelines for the design of grout curtails: Earth and rockfill dams”. An indicative illustration of grout curtain is shown in Figure 47.

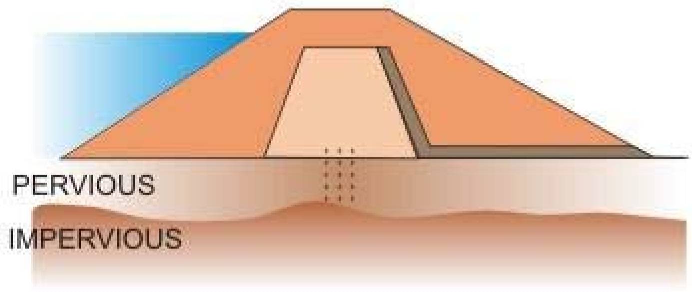


FIGURE 47. Grout curtain

Slurry Trench Cutoff Walls

A backhoe or dragline excavates a trench through the pervious deposits down to suitable impervious materials. A bentonite slurry, retained in the trench above the existing ground-water level, prevents the trench walls from caving. After a sufficient length of trench has been excavated and the bottom suitably prepared, back filling begins.

The physical characteristics of the backfill are specially controlled; in general, the backfill should be well-graded, impermeable in place, and sufficiently coarse to minimize post construction settlements. A selected amount of bentonite slurry may be blended with the backfill to improve its properties. The embankment should be suitably designed to resist cracking by differential settlement due to the slurry trench.

Steel Sheet Piles

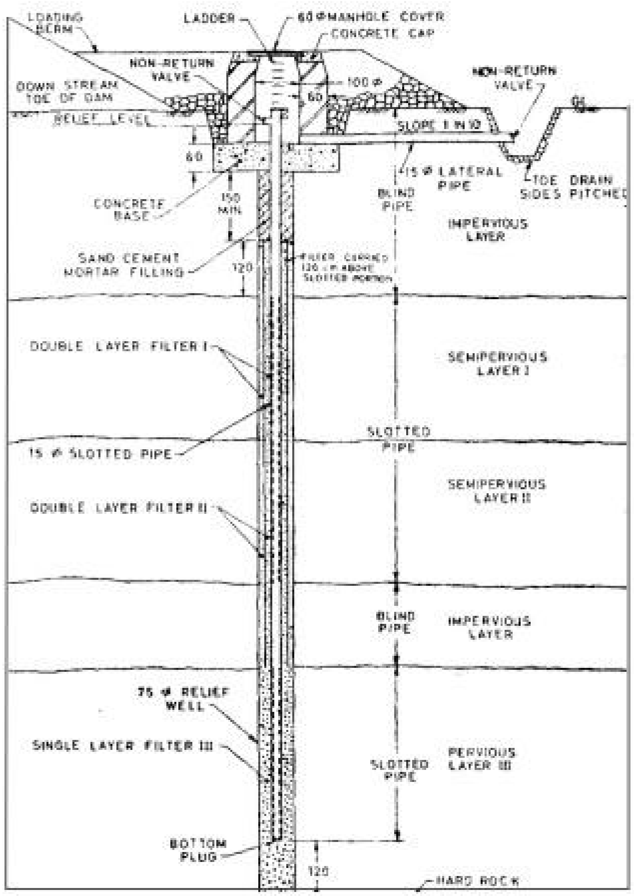
Sheet piles are useful as barrier to arrest internal erosion. But they have proved to be rather ineffective as a positive means of controlling seepage through pervious deposits. Even if sheet pile cutoffs are intact they are not water-tight because of leakage across the interlocks. In addition the locks may break because of defects in the steel or when a pile hits an obstacle. Once the lock is split, the width of the gap increases rapidly with increasing depth and may assume dimensions of a few meters.

Upstream Impervious Blanket

If a positive cutoff is not required, or is too costly, an upstream impervious blanket combined with relief wells in the downstream section may be used. Filter trenches supplement relief wells in heterogeneous deposits and in zones of seepage concentrations. An upstream blanket may result in major project economies, particularly if the only alternative consists of deep grout curtains or concrete cutoff walls. Since alluvial deposits in river valleys are often overlain by a surface layer of relatively impervious soils, it is advantageous if this natural impervious blanket can be incorporated into the overall scheme of seepage control.

Relief Wells

Relief wells are an important adjunct to most of the preceding basic schemes for seepage control. They are used not only in nearly all cases with upstream impervious blankets, but also along with other schemes, to provide additional assurance that excess hydrostatic pressures do not develop in the downstream portion of the dam, which could lead to piping. They also reduce the quantity of uncontrolled seepage flowing downstream of the dam and, hence, they control to some extent the occurrence and/or discharge of springs. Relief wells should be extended deep enough into the foundation so that the effects of minor geological details on performance are minimized. It is necessary to note the importance of continuous observation and maintenance of relief wells, if they are essential to the overall system of seepage control Details of relief wells may be had from Bureau of Indian Standards code IS: 5050-1968 “Code of Practice for design, construction and maintenance of relief wells”. A view of the relief well is shown in Figure 48.



All dimensions in mm

FIGURE 48. A typical relief well

4.7.5 Seepage calculations in embankment dams

Two important seepage calculations are required in embankment dams, which are as follows:

1. Location of the phreatic line
2. Quantity of seepage discharge

Basic computations to arrive at these two parameters are discussed in the following paragraphs

Location of phreatic line

Phreatic line, also variously also called as saturation line, top flow line, seepage line, etc. is defined as the line within a dam in a vertical plane section below which the soil is saturated and there is positive hydraulic pressure. On the line itself, the hydrostatic pressure is equal to atmospheric pressure, that is, zero gauge pressure. Above the phreatic line, there will be a capillary zone in which the hydrostatic pressure is negative (Figure 49). Since the flow through the capillary zone is insignificant, it is usually neglected and hence the seepage line is taken as the deciding line between the saturated soil below and dry or moist soil above in a dam section.

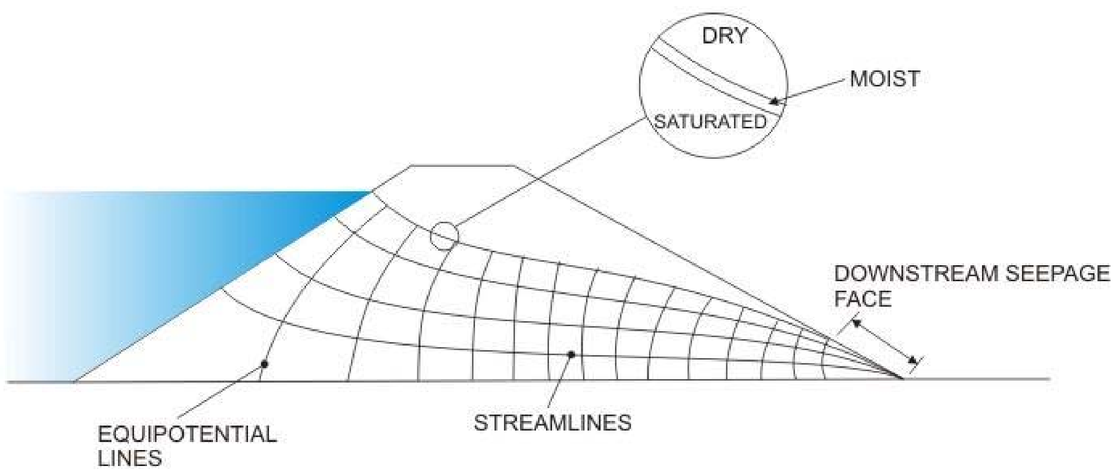


FIGURE 49. Flow net showing equipotential and streamlines for saturated flow through a homogeneous embankment dam

The flow of the seepage water below the phreatic line can be approximated by the Laplace Equation:

$$\frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial z^2} = 0 \qquad (1)$$

where ϕ is the potential head, and x and z are the coordinates in the horizontal and vertical dimensions, respectively

As may be observed from Figure 48, the streamlines are perpendicular to the equipotential lines, that is, lines joining points with equal potential head ϕ. The upstream face of the embankment dam, conforms to one equipotential.

Surface, since for any point along the face, the sum of hydrostatic head measured below the water surface and the potential head measured below the water surface and the potential head measured above any horizontal datum is the same. The embankment section shown in Figure 48 is homogeneous, which is used rarely in practical dam construction except for small bunds and levees. In the following paragraphs, methods to locate the phreatic line for two practical dam sections would be treated.

Homogeneous dam with horizontal drainage filter

This embankment section is shown in Figure 50. The point Q is located at the upstream face corresponding to the top surface of horizontal drainage of blanket. The point P is the point on the upstream face where it meets the reservoir water. The horizontal distance between P and Q is termed as L, as shown in the figure. Another point P_0 is located on the reservoir surface that is 0.3L from the point P.

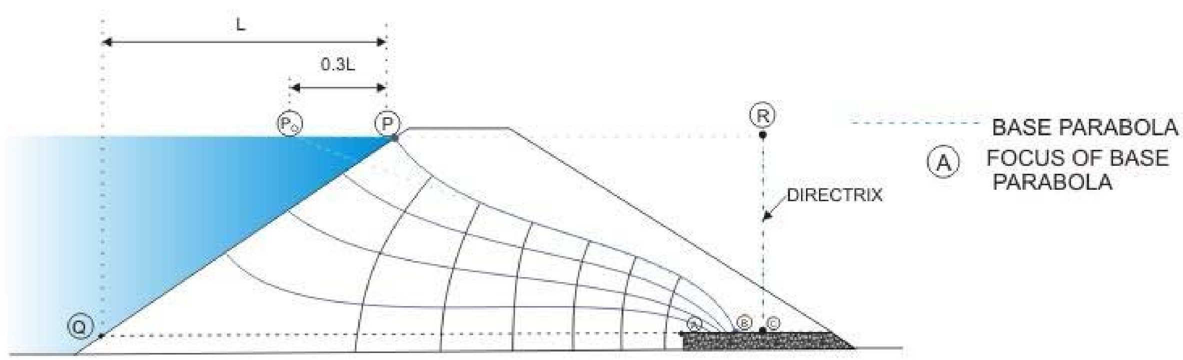


FIGURE 50. Determination of phreatic line for homogenous dam with horizontal drainage blanket

It is assumed that the phreatic line which emanates at P, meets the horizontal drainage blanket at B and is, for most of its downstream part, a parabola (first proposed by A. Casagrande). This curve is termed as the Base Parabola and is assumed to have its focus at A, the upstream edge of the horizontal drainage blanket. The Base Parabola, on its upstream part is assumed to meet the reservoir water surface at a point P_0 that is 0.3L upstream of P, as shown in Figure 49. In order to obtain the Base Parabola, one has to consider P_0 as the centre, and draw an arc A-R, with the radius equal to P_0-A. The point R is on a horizontal line at the same elevation of the reservoir surface. From point R, a perpendicular is dropped on to the top surface of the horizontal drainage blanket to meet it at a point C.

Knowing the focus, the directrix and the point P_0, a parabola can be drawn, which gives the Base Parabola shape. It may be recalled that point B is mid way of points A and C. At its upstream point, however, the parabola has to be modified such that it takes a curve upwards and meets the point P with the gradient of the phreatic line being perpendicular to the dams upstream face.

Homogeneous dam with rock toe

An embankment having a rock toe is shown in Figure 51. The Base Parabola may be drawn in much the same way by considering point A as the focus. The upstream face of the rock toe may be at any angle, say α, to the horizontal. Accordingly, an appropriate value of $\left(\frac{\Delta a}{a+\Delta a}\right)$ has to be read from the curve shown in Figure 52, which is used to determine the point of attachment of the actual seepage line with upstream face of rock toe at point D, as shown in the figure. For α less than 30^0, the above method has to be determined by a slightly modified technique. However, since rock toes with such angles are not practically provided, they are not considered here.

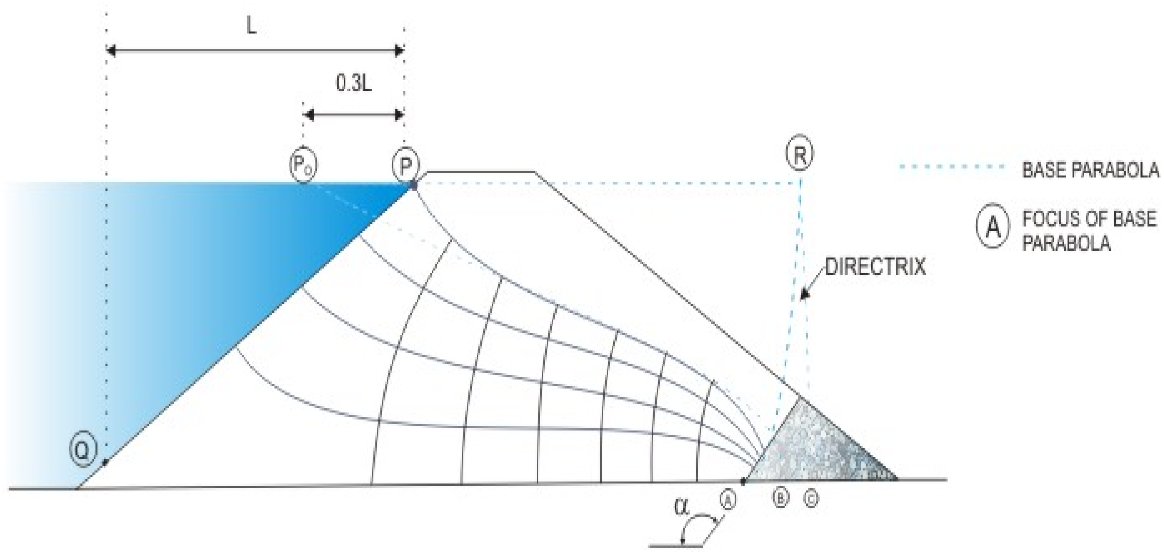


FIGURE 51. Determination of phreatic line for homogenous dam with rock toe

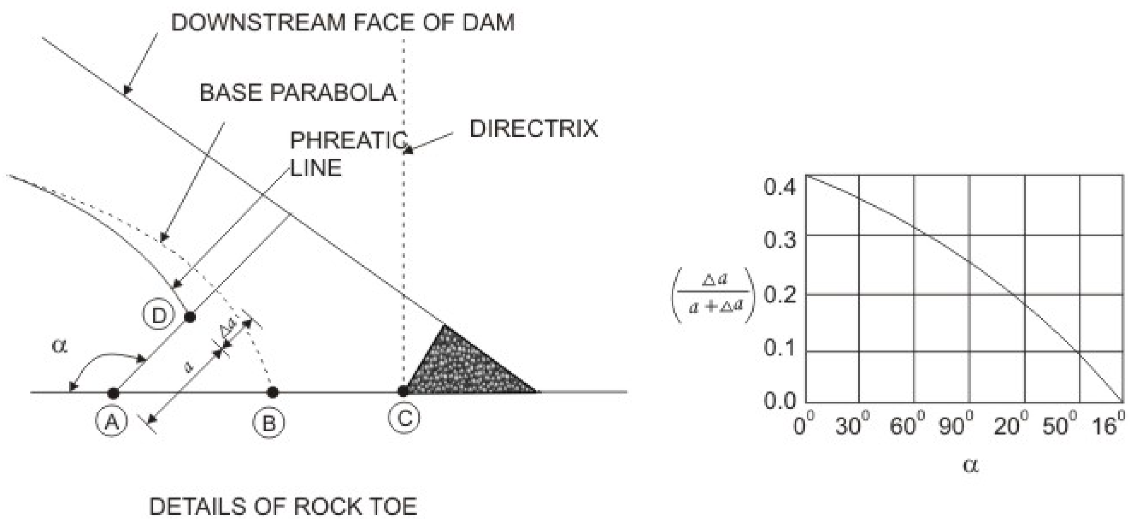


FIGURE 52. Determination of point of attachments (D) of phreatic line and upstream face of rock toe

Quantity of seepage flow

An embankment dam should be so designed that the quantity of water seeping through it is not excessive, otherwise the reservoir water would be drained off within some time. Hence, it is essential for the engineer to calculate the quantity of water seeping out through the dam. One method of calculating this is by drawing flownets, as shown in Figures 48, 49 and 50. Flownets can be drawn by trial and error, with the equipotential lines being approximately at right angles to the stream lines. The saturated zone through the embankment dam is then seen to be divided into a number of flow net boxes, bounded by two streamlines and two equipotential lines. If **h** is the total hydraulic head and $\mathbf{N_d}$ is the number of potential drops, then the drop in potential head (**Δh**) per drop is given as:

$$\Delta h = \frac{h}{\mathrm{N}_d} \tag{2}$$

If along the flow path, the length of the side of a flow net box between one potential and the other is ***l***, then the hydraulic gradient across the square is ***Δh/l***

The discharge passing through two streamlines of the field (Δq) is given as

$$\Delta q = \mathrm{K} \cdot \frac{\Delta h}{l} \cdot b \tag{3}$$

where **K** is the coefficient of permeability and **b** is the width of one flow channel, that is, the distance between two stream lines. If $\mathbf{N_f}$ is the total number of flow channels, then the seepage per unit width of the embankment (q) is given as

$$q = \sum \Delta q = \mathrm{K} \cdot \frac{h}{\mathrm{N}_d}\left(\frac{b}{l}\right) \cdot \mathrm{N}_f = \mathrm{K} \cdot h \cdot \left(\frac{b}{l}\right) \cdot \left(\frac{\mathrm{N}_f}{\mathrm{N}_d}\right) \qquad (4)$$

The above calculations would give the quantity of water seeping through the body of the dam. Similar calculations have to be done for the quantity of seepage taking place through the foundation.

4.7.6 Stability calculation

For an embankment dam the most important cause of failure is sliding. It may occur slowly or suddenly and with or without any prior warning. Such a failure causes a pportion of the earth or rockfill to slide downwards and outwards with respect to the remaining part generally along a well-defined slide surface (Figure 53). As may be observed, the profile of the slide surface may be nearly approximated by circular arcs (Figure 53 a, b) or by wedges (Figure 53 c, d). The upstream failure surfaces in all cases may be possible, among others, during sudden drawdown of the reservoir level from elevation I to II, shown in the figures. At the time of the failure, the average shearing resistance all along the sliding surface. It is, therefore, necessary that the designers take special care to estimate the possibility of such a failure.

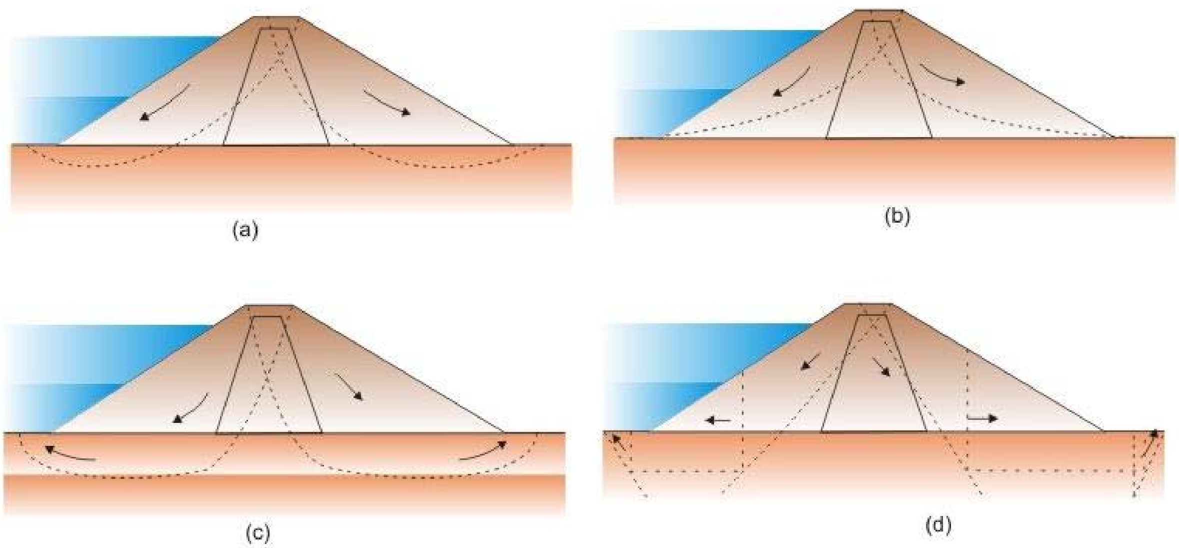


FIGURE 53. Possible modes of failure of embankment dams
(a) Homogeneous foundation
(b) Hard foundation
(c) Layered foundation
(d) Approximation of failure surfaces of(c)

The slope stability methods generally employed to analyse the failure modes are two, depending upon the profile of the assumed failure surface. These are:

a) Circular Arc method, and
b) Sliding Wedge method

In the Circular Arc method of analysis, the surface of rupture is assumed as cylindrical or in the cross section by an arc of a circle. This method, also known as the Swedish Slip Circle method, is generally applicable for analysing slopes of homogeneous earth dams and dams resting upon thick deposits of fine grained materials.

In circular arc method of analysis, the sliding mass is divided in to a convenient number of slices (Figure 54a). Each slice is assumed to act independently of its adjoining slices and the forces acting on the sides of a slice have no influence on the shear resistance which may develop on the bottom of the slice.

The Sliding Wedge method of analysis is generally applicable in the circumstances where it appears that the failure surface may be best approximated by a series of planes rather than a smooth continuous curve. This method is generally applicable under the following two circumstances:

a) Where one or more horizontal layers of weak soil exists in the upper part of the foundation, and
b) Where the foundation consists of hard stratum through which failure is not anticipated and the dam resting on it has a core of fine grained soil with relatively large shells of dense granular material.

In sliding wedge method of analysis, the trial sliding mass is divided into two or three segments (Figure 54b). The top segment is called the active wedge and the bottom segment is called the passive wedge. The middle wedge in case of a three wedge system is called the central block. The resultant of the forces acting on the active wedge and the passive wedge are first determined. These resultants acting on the central block along with other forces on the block shall give a closed polygon of forces for stability.

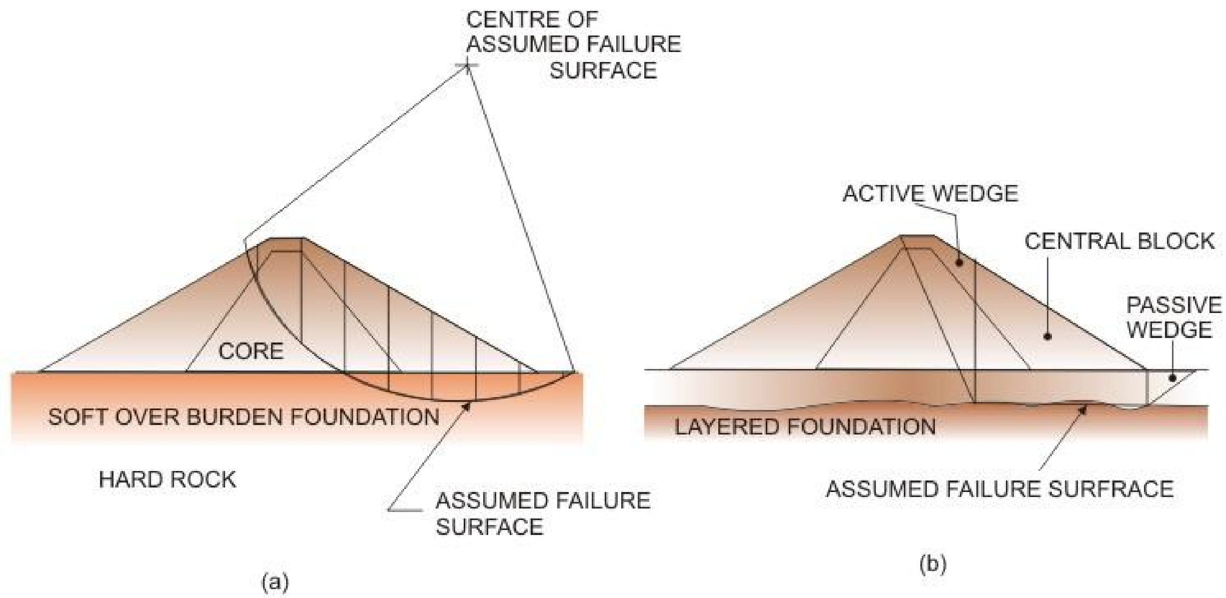


FIGURE 54. Stability analysis techniques for earth dams ; (a) Swedish ship circle method; (b) Sliding wedge method

The details about the two methods of stability analysis for earthen embankment dams is given in the Bureau of Indian Standards Code IS: 894-1975 “Code of practice for stability analysis of earth dams”.

An earth dam has to be safe and stable during all phases of its construction and operation of the reservoir. Hence, the analyses have to be carried out for the most critical combination of external forces which are likely to occur in practice. The following conditions are usually considered critical for the stability of an earthen embankment dam:

- Case I - Construction condition with or without partial pool: Check stability of upstream and downstream slopes
- Case II - Reservoir partial pool: Check stability of upstream slope
- Case III - Sudden drawdown: Check stability of downstream slope
- Case IV - Steady seepage: Check stability of downstream slope
- Case V - Steady seepage with sustained rainfall: Check stability of upstream and downstream slopes

The above critical conditions are explained below. For further details, one may refer to IS: 7894-1975 “Code of practice for stability analysis of earth dams”.

Case I - Construction Condition With or Without Partial Pool (for Upstream and Downstream Slopes)

This represents a situation when the structure is just constructed. In this condition the pore pressures developed as a result of dam material compression due to the overlying fill are not dissipated or are only partly dissipated. If the rate of raising of dam is less than about 15 meters per year, this condition may not become critical as the residual pore pressure in the dam and foundation are expected to be negligible except in highly clayey foundation with high water table, for example, marshy areas.

Construction pore pressures may exceed the pore pressures likely to be developed due to the seepage from the reservoir and consequently may control the design of dam. The magnitude and distribution of these pore pressures depend primarily on the construction water content in embankment and natural water content in the foundation, the properties of the soil, rate of raising, the height of dam and the rate at which dissipation may occur due to internal drainage.

Case II - Reservoir Partial Pool (for Upstream Slope)

This condition corresponds to the initial partial pool filling in which it is assumed that a condition of steady seepage has developed at the intermediate stages. The stability of upstream slope shall be investigated for various reservoir levels on upstream, usually levels corresponding to one-third to two-thirds height of head of water to be stored at full reservoir level and minimum value of factor of safety worked out. The analysis should account for reduction, if any, in the effective normal stresses where the pore pressures developed during construction are not dissipated before a partial pool condition can develop.

All the zones above phreatic line drawn for upstream water level under consideration should be considered as moist for working out resisting and driving forces and zones below it should be taken with their submerged weights for working out both resisting and driving forces.

Partial pool condition may not prove to be critical for all earth dams and hence analysis for this condition needs to be carried out only in cases where it is considered necessary. This condition is likely to be critical in cases of high dams where the range of drawdown is small as compared to the height of dam.

Case III - Sudden Drawdown (for Upstream Slope)

Earth dams may get saturated due to prolonged higher reservoir levels. Sudden drawdown condition corresponds to the subsequent lowering of reservoir level rate faster than pore water can dissipate. This induces unbalanced seepage forces and excess pore water pressures. This condition becomes critical if the materials of the upstream portion of the dam are not freely draining.

Depending upon the value of the coefficient of permeability of the upstream shell material, the pore pressures in the shell material in the drawdown range shall be allowed arbitrarily in the analysis as follows:

a) Full pore pressures shall be considered if the coefficient of permeability is less than 10^{-4} cm/s.

b) No pore pressures shall be considered if the coefficient of permeability is greater than 10^{-2} cm/s.

c) A linear variation from full pore pressures to zero pore pressures shall be considered for the coefficients of permeability lying between 10^{-4} cm/s to 10^{-2} cm/s.

The above values of pore pressures are based on a drawdown rate of 3 m/month.

For the core material which is generally impervious full pore pressures shall be allowed for the core zone lying in the drawdown range. If a zone of random material with the properties intermediate between core and the shell material is provided in between upstream shell and core of the dam, the pore pressures for sudden drawdown condition shall be allowed for in the same way as for the core.

Case IV - Steady Seepage (for Downstream Slope)

The condition of steady seepage is developed when the water level is maintained at a constant level for sufficiently long time and the seepage lines arc established in the earth dam section. This condition is likely to be critical for the downstream slope. In the analysis, existence of tail water and drawdown effects, if any, shall also be taken into account. The stability of downstream slope shall be examined by effective stress method. Steady seepage from level in the reservoir which is sustained for a period of one month should be taken as critical.

The stability analysis of the earth dam shall be done assuming that the dam is fully saturated below phreatic line. Allowance for pore pressure in the analysis shall be made in terms of the buoyancy of the material or by drawing flow nets. The core material lying below the phreatic line (and above the tail water level, if any) shall be considered as saturated for calculating the driving forces and buoyant for resisting forces. All the zones of the dam and foundation lying below the tail water level, if any, shall be considered as buoyant for calculating the driving and resisting forces. A part of upstream pervious shell material below the phreatic line, if any, included in trial sliding, mass shall be considered as saturated for calculating the driving and buoyant for resisting forces. All zones lying above the phreatic line shall be considered as moist for calculating both the driving and resisting forces.

Case V - Steady Seepage with Sustained Rainfall (for Downstream Slope)

Where there is a possibility of sustained rainfall, the stability of the downstream slope shall be analysed on arbitrary assumption that a partial saturation of shell material due to rainfall takes place. Accordingly for this condition of analysis, the shell and other material lying above the phreatic line shall be considered as moist for calculating driving forces and buoyant for resisting forces.

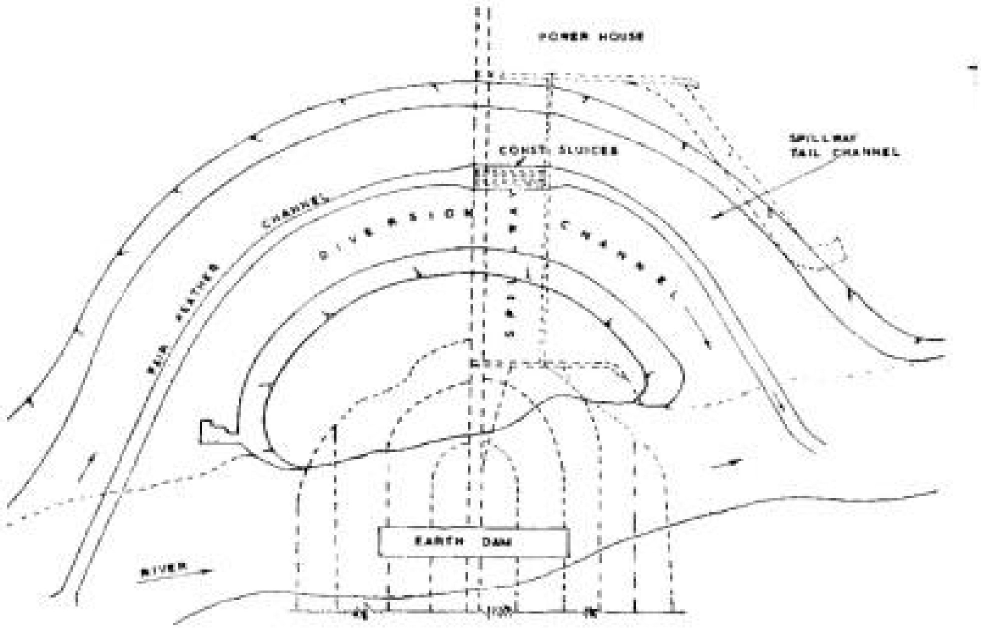
Earthquake Condition

In the regions of seismic activity, stability calculations of the slope of an embankment dam has to include earthquake forces also because they reduce the margin of safety or may even bring about the collapse of the structure. General design approach for earthquake forces is given in the Bureau of Indian Standards Code IS: 1893-1975/2002 “Criteria for earthquake resistant design of structures”. Where the analysis is carried out by the circular arc or sliding wedge method, the total weight of the sliding mass considered for working out horizontal seismic forces has to be based on saturated unit weights of the zones below the phreatic line and moist weights above it. If the zone above the phreatic line is freely draining, drained weights shall have to be considered for that zone.

4.7.7 Construction of embankment dams

River diversion

As was discussed for concrete dams in Lesson 4.6, arrangements have to be made to divert the river while constructing an embankment dam. This temporary exclusion of river flow is necessary to provide dry or semi-dry area for the work to continue. However, for concrete dams, after the initial levels in touch with the foundation have been built, the river water during floods may be allowed to overtop the partially completed structure during monsoons. In case of embankment dam, it is rarely allowed as overtopping would generally lead to a washout of the downstream face first and the rest follows the collapse. Hence, for embankment dams, the diversion measures require the detouring of the whole flood water through bye-pass. In only special cases, has an embankment been constructed that has been allowed to overtop only when partly constructed. The downstream face has to be sufficiently protected in that case with concrete blocks or gabions. Details of river diversion for dams may be found in the Bureau of Indian Standards Code IS:10084-1984 (Part 2) “Design of diversion works-Criteria” from where the following has been called. Figure 55 shows a typical example of a diversion channel for earth/rockfill dam project in a narrow river. The layout and principal dimensions, specifically the cross-section of the diversion channel is governed by several considerations such as topography, volume of flood to be handled, water levels during passage of monsoon and non-monsoon floods in consonance with raising of the dam and requirement of excavated material from the diversion channel for use in constructing the earth dam, etc. The coffer dams in such a case form an integral part of the earth and rockfill dam in the finally completed stage, and are also not allowed to be overtopped. Because of the considerable expenditure and time involved in the construction of diversion channel for earth dams, these channels are designed to be useful for other purpose also such as spillway tail channel or power house tail channel. Although, initially such channels may be without protective lining on the sides, they are protected at a subsequent stage when utilized for spillway or power house tail race channel.



In a wide river channel, provided the height of the earth dam is small enough, diversion could be managed by a temporary channel revolving a gap through the earthfill dam while the remainder of the embankment is being constructed (Figure 56). Before the stream is diverted, the foundation required for the dam should be completed in the area where the temporary opening will be left through the embankment. This preparation would include excavation and refilling of a cut-off trench, if one is to be constructed. The stream is then channelised through this area after which the foundation work in the remainder of the streambed is completed.

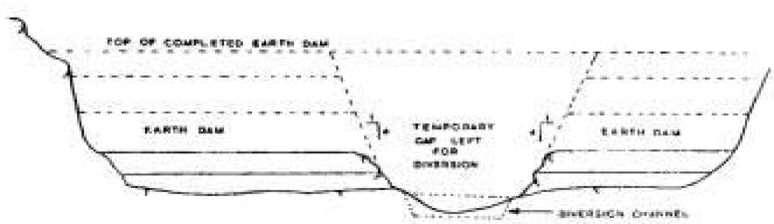


FIGURE 56. Diversion through a gap in a partially constructed earth dam in a wide river

In some rivers, the floods may be so large that provision of diversion channels even for average floods may be highly expensive. The only alternative then is to have the discharge passed through a conduit excavated through one or either abutment. Coffer dams, nevertheless, have to be constructed on the upstream and downstream of the working area to divert the stream flow into the diversion tunnel and to prevent the water on the downstream side of the river from flooding the work space.

Foundation preparation

The principles of foundation treatment for concrete gravity dams have been outlined in Lesson 4.6, Similar methods have to be adopted for embankment dams, too. However, the Bureau of Indian Standards code IS: 11973-1986 “Code of practice for treatment of rock foundations, core and abutment contacts with rock, for embankment dams” deals specifically with the requirement of an embankment dam. Some important points from this standard are explained below.

Basically, the surface under the entire core and under a portion of the upstream filter and downstream transition zone shall be completely excavated to such rock as will offer adequate resistance to erosion of fines in the core.

All loose or semi-detached blocks of rock should be removed. The quality of rock shall be judged characteristic of core material. Rock of ‘Lugeon’ values in percolation test within 10 (Ten) will generally be free of cracks larger than 0.025 mm. Erosion of fines from core materials commonly used would not occur through such cracks. Grouting may be necessary to bring down ‘Lugeon’ values to above allowable limits in the contact zone.

The amount of care required in treating the rock surface is controlled by the character of the core material. If the core material is resistant to piping, especially if it contains considerable coarse material with adequate proportion of sand, surface treatment is less demanding than if the core material is susceptible to piping; for example, a fine silty sand and very lean clays. In the latter case, extreme care should be taken and the core material should be placed only after very careful inspection of the treated surface. For dispersive clays, special precautions, such as protection by filter fabric or plastic concrete may be required.

Small ribs and similar irregularities should be filled with plastic concrete to produce slopes not steeper than about 1:1 where the difference in elevation is a few centimeters to a meter or so. Surface treatment in this fashion should extend upstream to approximately the mid-point of the upstream filter and downstream at least 0.6 to 0.9 m beyond the downstream edge of the fine filter. In particularly adverse situations, such as where there are joints wider than the coarser particles of the filter, surface treatment as described may be necessary under the entire transition zone.

The final rock surface should have smooth contours against which soil can be compacted by heavy equipment. Hand compaction is generally unsatisfactory and it is advisable to place plastic concrete in core contact areas of conduit trenches and other irregularities transverse to the dam axis for a width at least 0.5 H or preferably 1.0 H.

Surface treatment as described may be difficult to accomplish on steeply sloping abutments. In this case, gunite may be used for filling depressions after the cracks and joints have been cleaned and sealed. If there is extensive jointing, especially if the joints slope upward away from the face, adequate sealing of the joints may require constructing a concrete slab, which is dowelled to the rock, and then grouting through the slab.

The depth of excavation necessary in weathered rock is difficult to establish during initial design. The depth of weathering is usually very irregular, being controlled by minor variations in joint spacing and rock type. Abrupt changes in elevation of the surface of 'groutable rock' probably will be found. Overhangs, some of large size, should be anticipated.

Usual practice is to select material, preferably a plastic soil, for the first lift over the rock surface. If plastic soils are limited, the most plastic soil available should be used. Gravel or stone exceeding about 50 mm in size should be removed or excluded from the material placed in this first layer over the rock to improve compaction at the contact. The surface on which the core material is placed should be moist but free of standing water, and the material when placed should be wet of optimum. In dry climates or during dry weather, difficulty may be experienced with this first lift becoming excessively dry where it feathers out on a gentle to moderate slope. In such a case the edge of the fill should be sloped slightly downward toward the contact with the rock. Against steep rock faces or adjacent to concrete structures, sloping the fill slightly upward near the contact is desirable to provide better clearance and better compaction at the contact.

Treatment of rock defects and discontinuities

In evaluating and planning for excavation and seepage control measures, special attention shall be given to discontinuities such as faults and relief (sheet) joints, which may extend for long distance as nearly plane surfaces. Relief joints may exist naturally or may open during excavation. They are most likely to occur in deep, steep-walled valleys, especially in brittle rocks, or where high modulus rock is underlain by low modulus rock. Since they are roughly parallel to the valley wall, they may cause slides during construction. Openings of several centimeters have been observed. Control of seepage through such joints becomes a major problem. Installation of concrete cutoffs across particularly bad joints may be warranted or extensive grouting may be necessary. Drainage from such joints shall be provided.

When seams are filled with silt, clay, etc, or in faults with gauge, it is essential to excavate and backfill the seam and gauge zones in the entire core contact zone. It is advisable to excavate and backfill a further length on the upstream for a distance equal to the reservoir head and backfill it with concrete. On the downstream side the seams should be excavated and backfilled with a well designed and adequate filter again for a distance equal to the reservoir head.

Grouting

There are three main objectives in the grouting programme (see also IS : 6066-1984). These are as follows:

- To reduce the seepage flow through the dam foundation;
- To prevent possible piping or washing of fines from the core into cracks and fissures in the foundation; and

To reduce the hydrostatic pressure in the downstream foundation of the dam. The latter is generally a problem only for dams on fairly weak foundations and critical abutment configurations. This is usually accomplished in conjunction with an abutment drainage system.

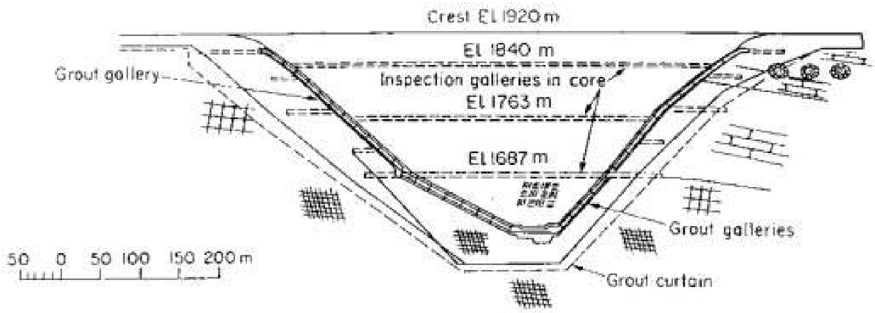
To prevent possible piping of the fine core material through the foundation, blanket grouting is accomplished as determined by the rock conditions. If the core foundation of the dam consists of closely fractured and jointed rock, a blanket grout pattern is used with holes spaced at 3 m to 5 m with depths of 6 m to 10 m. If the foundation rock is massive, no blanket grouting is done. Localised area consisting of faults, fissures, or cracks are generally grouted upstream of the cutoff and sometimes downstream.

Quality Control

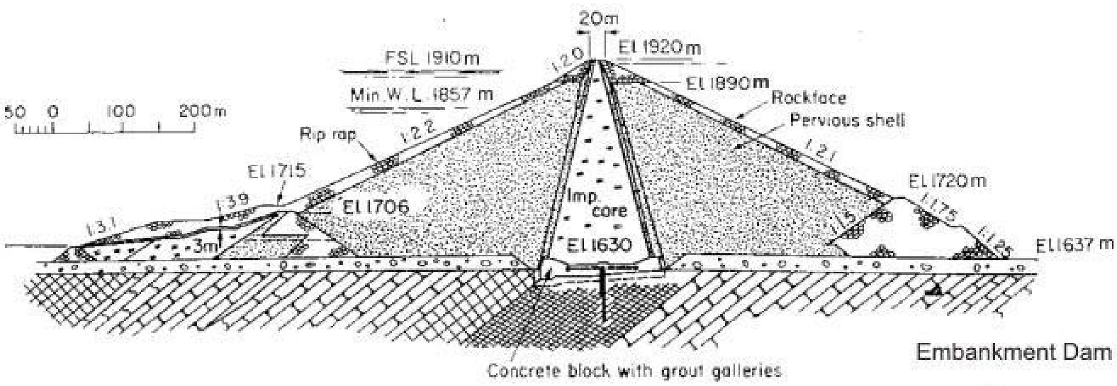
The performance of an earth or rockfill dam depends upon the control exercised during construction, supervision and inspection. An entirely safe design may be ruined by careless and shoddy execution. Proper quality control during construction is as important as the design. The skill, experience and judgment required of the engineer in charge of construction, is in no way lesser than that of the design engineer. Hence, the Bureau of Indian Standards has published the following publication which provides guidance for construction of embankment dams with regards to quality control IS: 14690-1999 “Quality control during construction of earth and rockfill dams-recommendations”.

4.7.8 Instrumentation

Numerous embankment dams constructed in India and abroad and the height achieved ever increasing, like the Nurek Dam of Russia (Figure 57). From the point of view of safety as well as to garner knowledge about the physical behaviour of these dams, instrumentation has been recommended all medium and large sized dams. The data obtained from these measurements also help the commonly made either explicity or implicity in an embankment dam design. In fact, it is very important to monitor the behavior of the dams under earthquake loadings and those constructed in regions of high seismic activity need to be instrumented carefully.



Embankment Dam Elevation



Embankment Dam Section

FIGURE 57. Elevation and cross section of Nurek embankment Dam,Russia

Even for the continued maintenance of any embankment dam, vertical and timely observations of the measurements taken will provide means of evaluating the behaviour of the structure and, if need be, allow the engineers to take appropriate remedial measures on the basis of the observed data. Hence, the importance of providing instruments in an embankment dam, or any dam for that matter, cannot be over emphasized. The Bureau of Indian Standards code IS: 7436 (part1)-1993 “Guide for types of measurements for structures in river valley projects and criteria for choice and location of measuring instruments (Earth and Rockfill dams)” provides guidelines for various types of instrumentation to be carried out in embankment dam. The following paragraphs highlights the salient features of the recommended measurements to be taken and the corresponding instruments that have to be installed.

Pore Pressure

The measurement of pore pressure is probably the most important and usual measurement to be made in the embankments. Their measurement enables the seepage pattern set up after impounding of reservoir to be known, the danger of erosion to be estimated, at least partially, and the danger of slides in the dam and abutments to be estimated if the reliable shear strength is known. Valuable information about behaviour during construction and drawdown is obtained.

Movements

Measurement of movements is as important as the measurement of pore pressures. Movements conforming to normal expectations are basic requirements of a stable dam. An accurate measurement of internal and external movements is of value in controlling construction stability. The measurement of the plastic deformation of the upstream and downstream slopes under the cycles of reservoir operation may indicate the likely development of shear failure at weak points.

Seepage

Measurement of seepage through and past a dam, may indicate erosion or blocking of downstream drains and relief wells, by increase or decrease of seepage, respectively at constant reservoir conditions. Seepage and erosion along the lines of poor compaction and through cracks in foundations and fills may specially be indicated by such measurements.

Strains and Stresses

Design analysis of earth and rockfill dams is based on radical simplifications of the stress pattern and the shape of the rupture planes. Stress measurements, therefore, require considerable judgement in interpretation. Accurate measurement of stress is difficult and distribution of stress in earth and rockfill dams is complex. Strains may be calculated from displacements or measured directly.

Dynamic Loads (Earthquakes)

Earthquake causes sudden dynamic loading and measurement of vibrations in dams located in areas subjected to seismicity is important for evolving design criteria for such conditions. The instruments for recording the measurements can be divided into two types: Vertical Movement Gauges and Horizontal Movement Gauges. These are explained below.

Vertical Movement Gauges

Surface Markers: Surface marker points consist of steel bars which are driven vertically into the embankment or the ground and embedded in concrete. A reference base line is established on a firm ground outside the area of movement due to reservoir and embankment load. Position of surface stakes or markers fixed on the embankment are determined by survey with reference to this line. It measures horizontal movements also. Surface markers may be established on lines parallel to the centre line of the dam at 50 to 100m centres. The lines may be at the edge of the top width of the dam, at the edge of berms or at suitable intervals along the slope, at the toe. of the dam and at 50 m and 100 m from toe if foundation soil is not firm. These may be provided both on upstream and downstream slopes except in locations on upstream slope which remain throughout the year below lake water.

Cross-Arm Installation: It consists of telescopic steel casing to which are attached horizontal cross-arms at predetermined vertical intervals. As the soil settles, sections of casing are dragged down and these are thus relocated in their new positions by lowering down the casing a problem fitted with retractable claws which engage the bottom of each section in turn or by using an electrical probe. Cross arms are used in order to eliminate any possibility of the casing sections not settling along with the surrounding soil.

Hydraulic Device: It is made from two 50 mm diameter brass pipe nipples soldered to a common diaphragm. Pipe caps are secured at both ends of the assembly which is then mounted vertically on a steel base plate for anchorage in the embankment. The diaphragm separates the upper (air) chamber from the lower (overflow) chamber and encloses a plastic float valve which prevents water from entering the air chamber during flushing of the lower chamber. Three 8-mm outer diameter plastic tubes are embedded in trenches which are excavated to maintain continuous downward slopes to the instrument terminal. The instrument terminal is equipped with a pump, air compressor and high precision pressure gauges.

Geonor Probe: It consists of a three-pronged tip connected to a double rod which is lowered down a bore hole or driven in soft ground to desired depth. When the outer rod is held and the inner rod driven with hammer, the three prongs are forced out in the surrounding soil. The outer rod is then uncovered from, the tip and withdrawn a few centimeters. The top of the inner rod, which remains in contact with the anchored tip is used as a reference point to measure the settlement of the tip. This device is particularly well suited for measuring settlements of soft foundations under-low embankments.

Foundation Settlement Measuring Device: It is a base plate placed on the foundation line with a vertical column of steel tubings. The position of the base plate is determined by a surrounding device lowered from the top open end of the steel tubings.

Magnetic Probe Extensometer: This system consists of a magnet/lead switch probe of approximately 15mm diameter connected to an indicator with a marker connecting cable. Magnetic ring markers with stainless steel spring parts are installed over a series of PVC access pipes of 33 mm outer diameter and 27 mm inner diameter jointed together. The probes when lowered through the access pipe will give indications in the indicator where the magnet marker rings are located. When settling takes place the marker rings will move with the soil and the fresh positions of the marker rings indicate the amount of settlements with respect to earlier logged position.

Induction Coil Type Extensometers: This induction coil type extensometers consist of an electrical probe made of PVC and having a diameter of 35mm or 43mm which houses a primary electrical exit. The probe is connected to an indicator electrical cable. Indicator has a volt/ammeter to measure the voltage/current increase when the primary coil enters a secondary coil, when there is a steel marker ring or plate, it will indicate a current/voltage which could be read through the indicator. Series of marker rings installed over a corrugated PVC pipe installed over a PVC access tubes or inclinometer tube should help monitoring the settlement.

Horizontal movement gauges

Cross-Arm Installation for Measurement of Horizontal Movement: This installation is similar to that described above but instead of cross-arms fixed at different sections there are two Vertical plates at the same level placed at a certain distance apart. The relative horizontal movements between the two cross-arms are measured by transmitting the same by means of a cable to a pair of counter weights, which move vertically in the tubing. A sounding probe similar to that used in measurement of vertical movement installation determines the position of the counter-weights.

Inclinometers: Plastic or aluminium tubing is placed vertically in the dam with its bottom anchored to firm unyielding stratum. The inclination of the tubing is measured by a sensitive electrical inclinometer, step by step, starting from the bottom of the tubing. Horizontal movements are computed by integrating the movements starting from the bottom, on the basis of changes in the inclination. Vertical movements may also be measured by using telescoping couplings for connecting the sections of the tubings and noting the positions of the ends of each section by a mechanical latching device, or if metal rings are embedded in the end portions of plastic tubing, by an electro magnetic device. Each section of tubing is anchored to the surrounding soil mass by fixing flanges or collars to the tubing. Alternatively, when an electromagnetic sounding device is used, the plastic tubing passes through encircling metal discs which are free to move along with the earth mass and the position of these discs are determined by the device.

Piezometers

Piezometers are installed in embankment dams to monitor the pressure of water within the soil or rock fragment. Typical installation locations of these devices are shown in Figure 58 and the details of some particular types are described below.

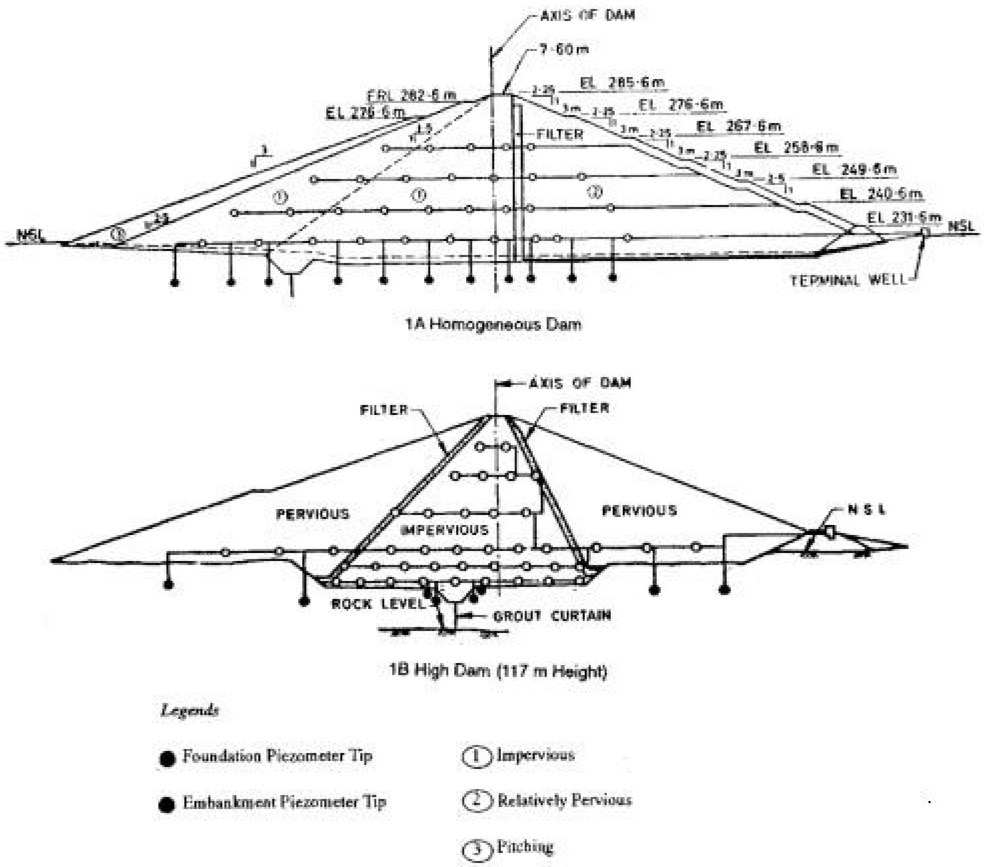


FIGURE 58. Typical installation of piezometers in embankment dams

Porous Tip/ Tube Piezometer: This is a steel or PVC pipe 10 to 40 mm in diameter placed vertically during construction or in a borehole after construction. A porous element is fixed at the bottom of the pipe or alternatively, the lower portion is perforated, and soil prevented from entering the pipe by surrounding the perforated portion by brass wire mesh and a gunny bag filled with filter material. With increase or decrease of pore water pressure in the soil near the perforated portion, water level rises or drops in the pipe and this level is noted by as electrical sounding device or a bell sounder.

Closed System Hydraulic Piezometer: It consists of a porous element which is connected by two plastic tubes to pressure gauges located in a terminal house or terminal well. The terminal house or well contains pumping and vacuum equipment, an air trap and a supply of de-aired water besides pressure gauges. Use of two plastic tubes makes possible the circulation of water through the porous element and to remove air from the system. The pore-water pressure is noted by means of gauges.

Electrical Piezometers: Electrical piezometer consists of a tip having a diaphragm which is deflected by the pore water pressure against one face. The deflection of the diaphragm is measured by a suitable strain gauge which may be suitably calibrated to read pore water pressure. The strain gauge is either electrical resistance (unbonded strain gauge) type or vibrating wire type.

Pneumatic Piezometers: In the pneumatic piezometers, the diaphragm deflection due to pore water pressure is balanced by a known air/gas pressure and recorded at the outside indicator end using pneumatic pressure gauges or pressure transducers.

Earth Pressure Cells: The usual instrument to measure earth pressure is the earth pressure cell. It uses a stiff diaphragm on which the earth pressure acts. The action is transmitted through an equalizing, confined, incompressible fluid (Mercury) on to a second pressure responsive element, the deflection of which is proportional to the earth pressure acting. The deflection is transformed into an electrical signal by a resistance wire (unbonded strain gauge) or vibrating wire strain gauge and transmitted through a cable embedded in the earth work to a receiver unit on the surface. The measure of the electrical signal indirectly indicates the earth pressure by appropriate calibration.

Instruments for measuring effects of dynamic loads due to earthquake include seismographs, accelerographs, and structural response records, details of which may be had from the Bureau of Indian Standards code IS: 4967-1968 “Recommendations for seismic instrumentation for river valley projects”.

Module
4
Hydraulic Structures for Flow Diversion and Storage

Lesson
8
Spillways and Energy Dissipators

Instructional objectives

On completion of this lesson, the student shall learn:

1. The functions of a spillways and energy dissipators in projects involving diversion and storage projects
2. Different types of spillways
3. How to determine the shape of an ogee-crested spillway and compute its discharge
4. The spillway profile in the presence of a breast wall
5. Criteria for selecting a particular type of spillway
6. Different types of energy dissipators
7. Design procedure for hydraulic jump and bucket-type energy dissipators
8. Protection measures against science downstream of energy dissipators

4.8.0 Introduction

The previous lessons dealt with storage reservoirs built by impounding a river with a dam and the common types of dams constructed by engineers. However, in rare cases only it is economical or practical for the reservoir to store the entire volume of the design flood within the reservoir without overtopping of dam. Hence, a dam may be constructed to that height which is permissible within the given topography of the location or limited by the expenditure that may be possible for investment. The excess flood water, therefore, has to be removed from the reservoir before it overtops the dam. Passages constructed either within a dam or in the periphery of the reservoir to safely pass this excess of the river during flood flows are called Spillways.

Ordinarily, the excess water is drawn from the top of the reservoir created by the dam and conveyed through an artificially created waterway back to the river. In some cases, the water may be diverted to an adjacent river valley. In addition to providing sufficient capacity, the spillway must be hydraulically adequate and structurally safe and must be located in such a way that the out-falling discharges back into the river do not erode or undermine the downstream toe of the dam. The surface of the spillway should also be such that it is able to withstand erosion or scouring due to the very high velocities generated during the passage of a flood through the spillway.

The flood water discharging through the spillway has to flow down from a higher elevation at the reservoir surface level to a lower elevation at the natural river level on the downstream through a passage, which is also considered a part of the spillway. At the bottom of the channel, where the water rushes out to meet the natural river, is usually provided with an energy dissipation device that kills most of the energy of the flowing water. These devices, commonly called as Energy Dissipators, are required to prevent the river surface from getting dangerously scoured by the impact of the outfalling water. In some cases, the water from the spillway may be allowed to drop over a free overfall, as in Kariba Dam on Zambezi River in Africa, where the free fall is over 100m.

In some projects, like the Indira Sagar Dam on River Narmada, two sets of spillways are provided – Main and Auxillary. The main spillway, also known as the service spillway is the one which is generally put into operation in passing most of the design flood. The crest levels of the auxillary spillways are usually higher and thus the discharge capacities are also small and are put into operation when the discharge in the river is higher than the capacity of the main spillway. Sometimes, an Emergency or Fuse Plug types of spillway is provided in the periphery of the reservoir which operates only when there is very high flood in the river higher than the design discharge or during the malfunctioning of normal spillways due to which there is a danger of the dam getting overtopped.

Usually, spillways are provided with gates, which provides a better control on the discharges passing through. However, in remote areas, where access to the gates by personnel may not be possible during all times as during the rainy season or in the night ungated spillways may have to be provided.

The capacity of a spillway is usually worked out on the basis of a flood routing study, explained in lesson 4.5. As such, the capacity of a spillway is seen to depend upon the following major factors:

- The inflow flood
- The volume of storage provided by the reservoir
- Crest height of the spillway
- Gated or ungated

According to the Bureau of Indian Standards guideline IS: 11223-1985 "Guidelines for fixing spillway capacity", the following values of inflow design floods (IDF) should be taken for the design of spillway:

- For large dams (defined as those with gross storage capacity greater than 60 million m^3 or hydraulic head greater than 60 million m^3 or hydraulic head between (2m and 30m), IDF should be based on the Standard Project Flood (SPF).
- For intermediate dams those with gross storage between 10 and 60 million m^3 or hydraulic head between (2m and 30m), IDF should be based on the Standard Project Flood (SPF).
- For small dams (gross storage between 0.5 to 10 million m^3 or hydraulic head between 7.5m to 12m), IDF may be taken as the 100 year return period flood.

The volume of the reservoir corresponding to various elevation levels as well as the elevation of the crest also affects the spillway capacity, as may be obvious from the flood routing procedure shown in Lesson 4.5.

If the spillway is gated, then the discharging water (Q) is controlled by the gate opening and hence the relation of Q to reservoir water level would be different from that of an ungated spillway. In the example of Lesson 4.5, an ungated spillway considered. Where as, in most practical cases, spillways are provided with gates and the gate operation is guided by a certain predetermined sequence which depends upon the inflow discharge. Hence, for an actual spillway capacity design, one has to consider not only the inflow hydrograph, but also the gate operation sequence.

Apart from spillways, which safely discharge the excess flood flows, outlets are provided in the body of the dam to provide water for various demands, like irrigation, power generation, etc. Hence, ordinarily riverflows are usually stored in the reservoir or released through the outlets, and the spillway is not required to function. Spillway flows will result during floods or periods of sustained high runoff when the capacities of other facilities are exceeded. Where large reservoir storage is provided, or where large outlet or diversion capacity is available, the spillway will be utilized infrequently. This feature may be contrasted with that of a diversion structure-like a barrage-where the storage is almost nil, and hence, the spillway there is in almost continuous operation.

Spillways are ordinarily classified according to their most prominent feature, either as it pertains to the control, to the discharge channel, or to some other component. The common types of spillway in use are the following:

1. Free Overfall (Straight Drop) Spillway
2. Overflow (Ogee) Spillway
3. Chute (Open Channel/Trough) Spillway
4. Side Channel Spillway
5. Shaft (Drop Inlet/Morning Glory) spillway
6. Tunnel (Conduit) spillway
7. Siphon spillway

These spillways are individually treated in the subsequent sections.

The water flowing down from the spillways possess a large amount of kinetic energy that is generated by virtue of its losing the potential head from the reservoir level to the level of the river on the downstream of the spillway. If this energy is not reduced, there are danger of scour to the riverbed which may threaten the stability of the dam or the neighbouring river valley slopes. The various arrangements for suppressing or killing of the high energy water at the downstream toe of the spillways are called Energy Dissipators. These are discussed at the end of this lesson.

4.8.1Free Overfall Spillway

In this type of spillway, the water freely drops down from the crest, as for an arch dam (Figure 1). It can also be provided for a decked over flow dam with a vertical or adverse inclined downstream face (Figure 2). Flows may be free discharging, as will be the case with a sharp-crested weir or they may be supported along a narrow section of the crest. Occasionally, the crest is extended in the form of an overhanging lip (Figure 3) to direct small discharges away from the face of the overfall section. In free falling water is ventilated sufficiently to prevent a pulsating, fluctuating jet.

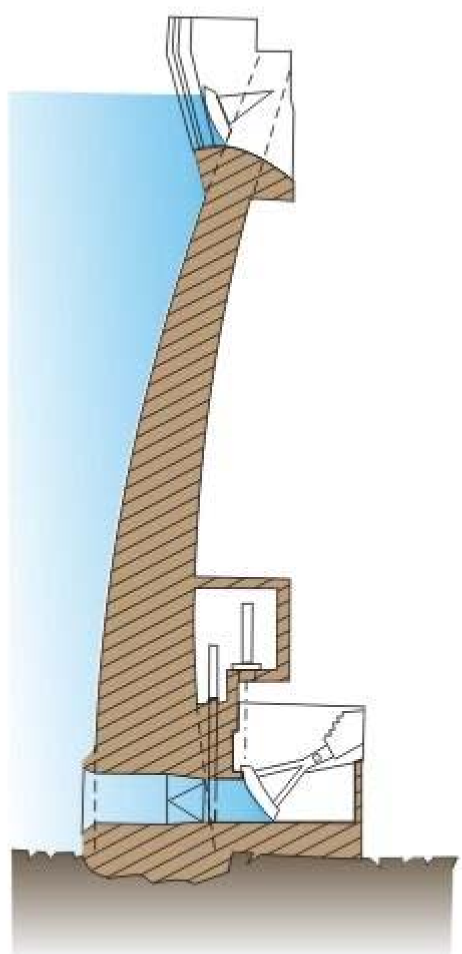
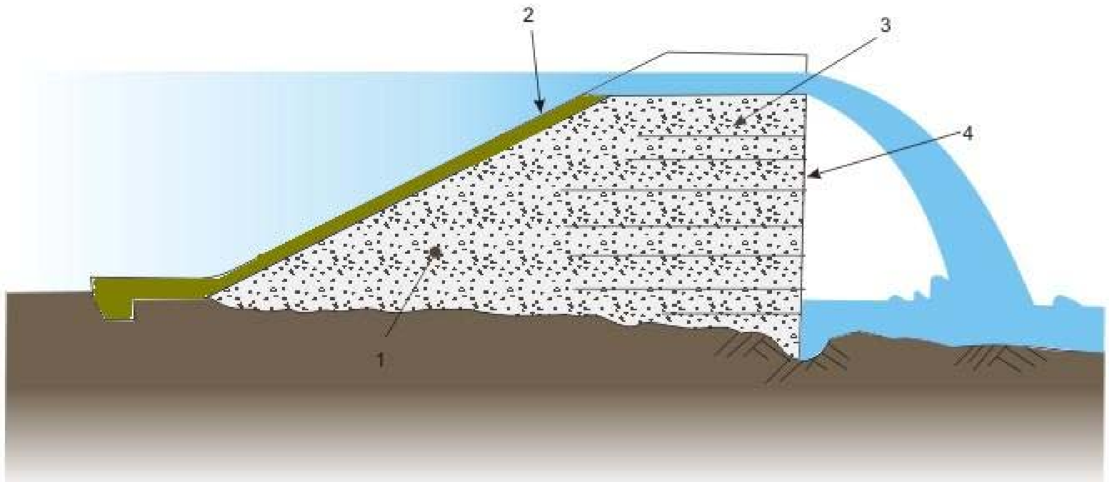


FIGURE 1. Free over fall spillway for an arch dam



LEGEND

1. RANDOM FILL 2. WATERTIGHT MEMBRANE 3. STEEL TENDONS 4. CONCRETE SLABS (1.5 M X 1.5 M).

FIGURE 2. Free over fall spillway for a decked embankment dam

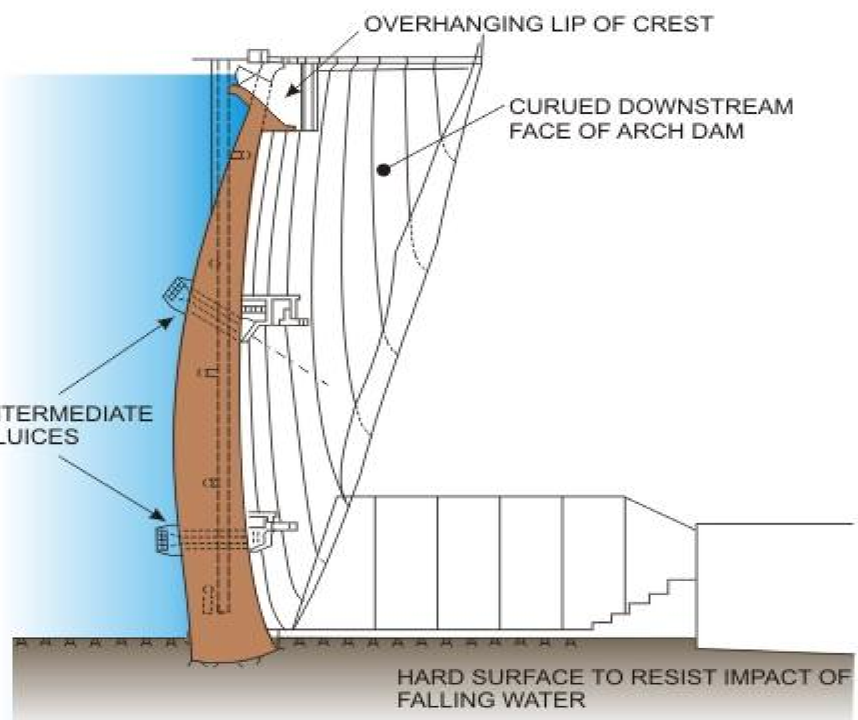


FIGURE 3. Short lip provided for overfall spilling of an arch dam

Where artificial protection is provided at the loose, as in Figure 3, the bottom may not scour but scour may occur for unprotected streambeds which will form deep plunge pool (Figure 4). The volume and the depth of the scour hole are related to the range of discharges, the height of the drop, and the depth of tail water. Where erosion cannot be tolerated an artificial pool can be created by constructing an auxiliary dam downstream of the main structure, or by excavating a basin which is then provided with a concrete apron or bucket.

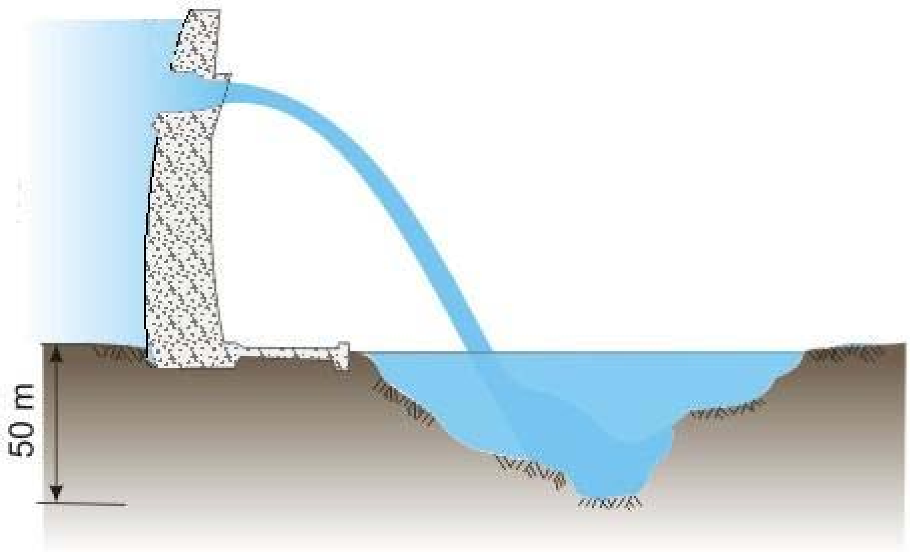


FIGURE 4. Scour below Kariba Dam Spillway , Zimbabwe

4.8.2 Overflow Spillway

The overflow type spillway has a crest shaped in the form of an ogee or S-shape (Figure 5). The upper curve of the ogee is made to conform closely to the profile of the lower nappe of a ventilated sheet of water falling from a sharp crested weir (Figure 6). Flow over the crest of an overflow spillway is made to adhere to the face of the profile by preventing access of air to the underside of the sheet of flowing water. Naturally, the shape of the overflow spillway is designed according to the shape of the lower nappe of a free flowing weir conveying the discharge flood. Hence, any discharge higher than the design flood passing through the overflow spillway would try to shoot forward and get detached from the spillway surface, which reduces the efficiency of the spillway due to the presence of negative pressure between the sheet of water and spillway surface. For discharges at designed head, the spillway attains near-maximum efficiency. The profile of the spillway surface is continued in a tangent along a slope to support the sheet of flow on the face of the overflow. A reverse curve at the bottom of the slope turns the flow in to the apron of a sliding basis or in to the spillway discharge channel.

An ogee crest apron may comprise an entire spillway such as the overflow of a concrete gravity dam (Figure 7), or the ogee crest may only be the control structure for some other type of spillway (Figure 8). Details of computing crest shape and discharges of ogee shaped crest is provided in Section 4.8.9.

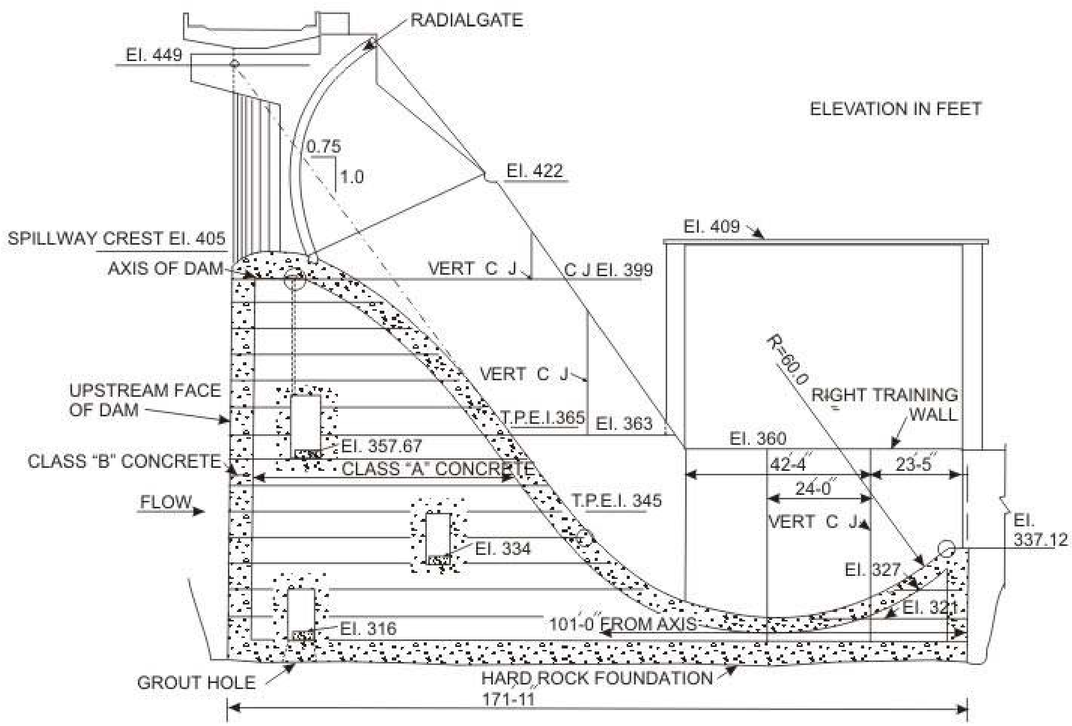


FIGURE 5. Typical overflow (ogee) spillway .Example of Panchet Dam on River Damodar

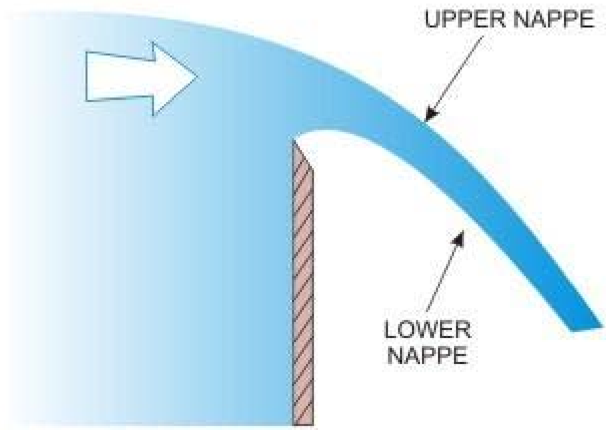


FIGURE 6. Outflow from a free-falling weir , properly ventilated from below

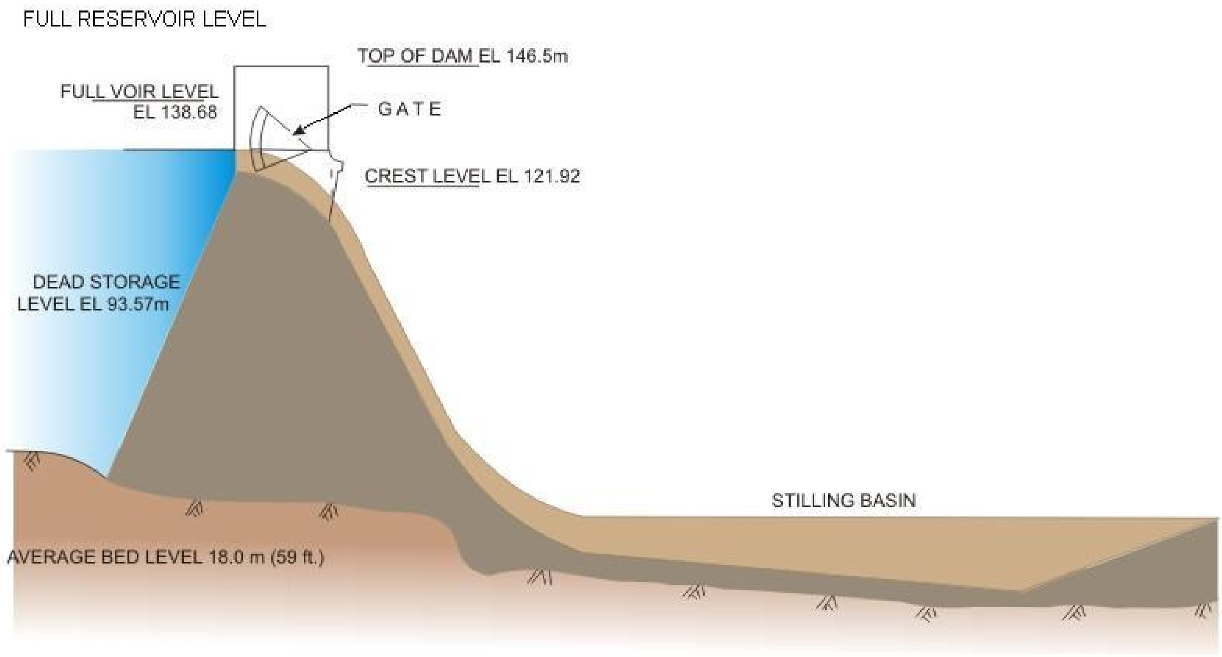


FIGURE 7. Ogee spillway & apron of Sardar Sarovar Dam spillway

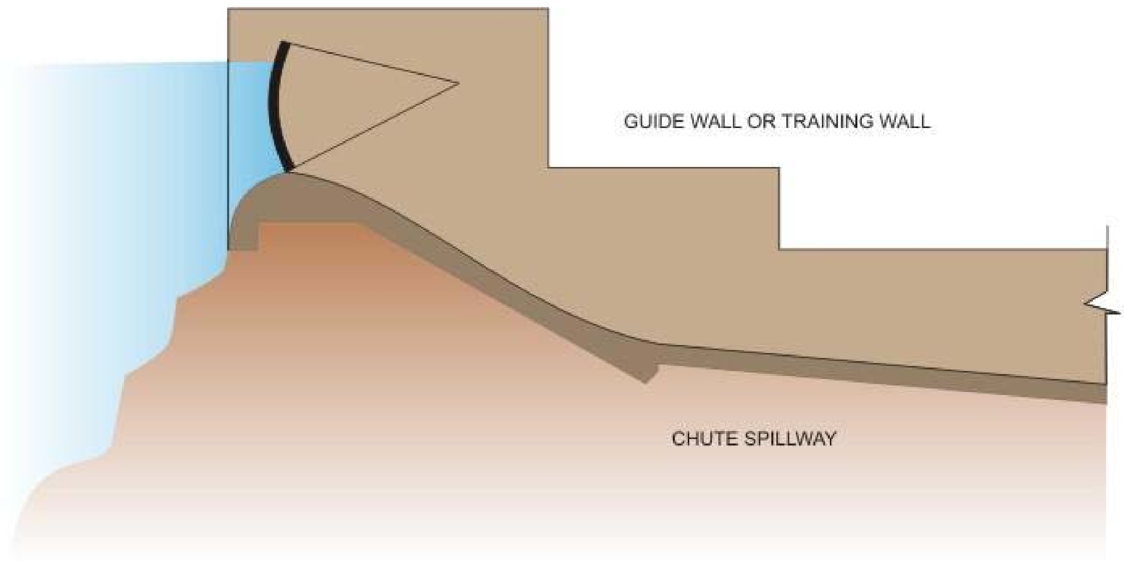


FIGURE 8. Ogee spillway for controlling flow into a chute-type spillway

4.8.3 Chute Spillway

A chute spillway, variously called as open channel or trough spillway, is one whose discharge is conveyed from the reservoir to the downstream river level through an open channel, placed either along a dam abutment or through a saddle (Figure 9). The control structure for the chute spillway need not necessarily be an overflow crest, and may be of the side-channel type (discussed in Section 4.9.4), as has been shown in Figure 10. However, the name is most often applied when the spillway control is placed normal or nearly normal to the axis of the open channel, and where the streamlines of flow both above and below the control crest follow in the direction of the axis.

Generally, the chute spillway has been mostly used in conjunction with embankment dams, like the Tehri dam, for example. Chute spillways are simple to design and construct and have been constructed successfully on all types of foundation materials, ranging from solid rock to soft clay.

Chute spillways ordinarily consist of an entrance channel, a control structure, a discharge channel, a terminal structure, and an outlet channel. Often, the axis of the entrance channel or that of the discharge channel must be curved to fit the topography. For further details, one may refer to the Bureau of Indian Standards Code IS: 5186-1994 "Design of chute and side channel spillways-criteria".

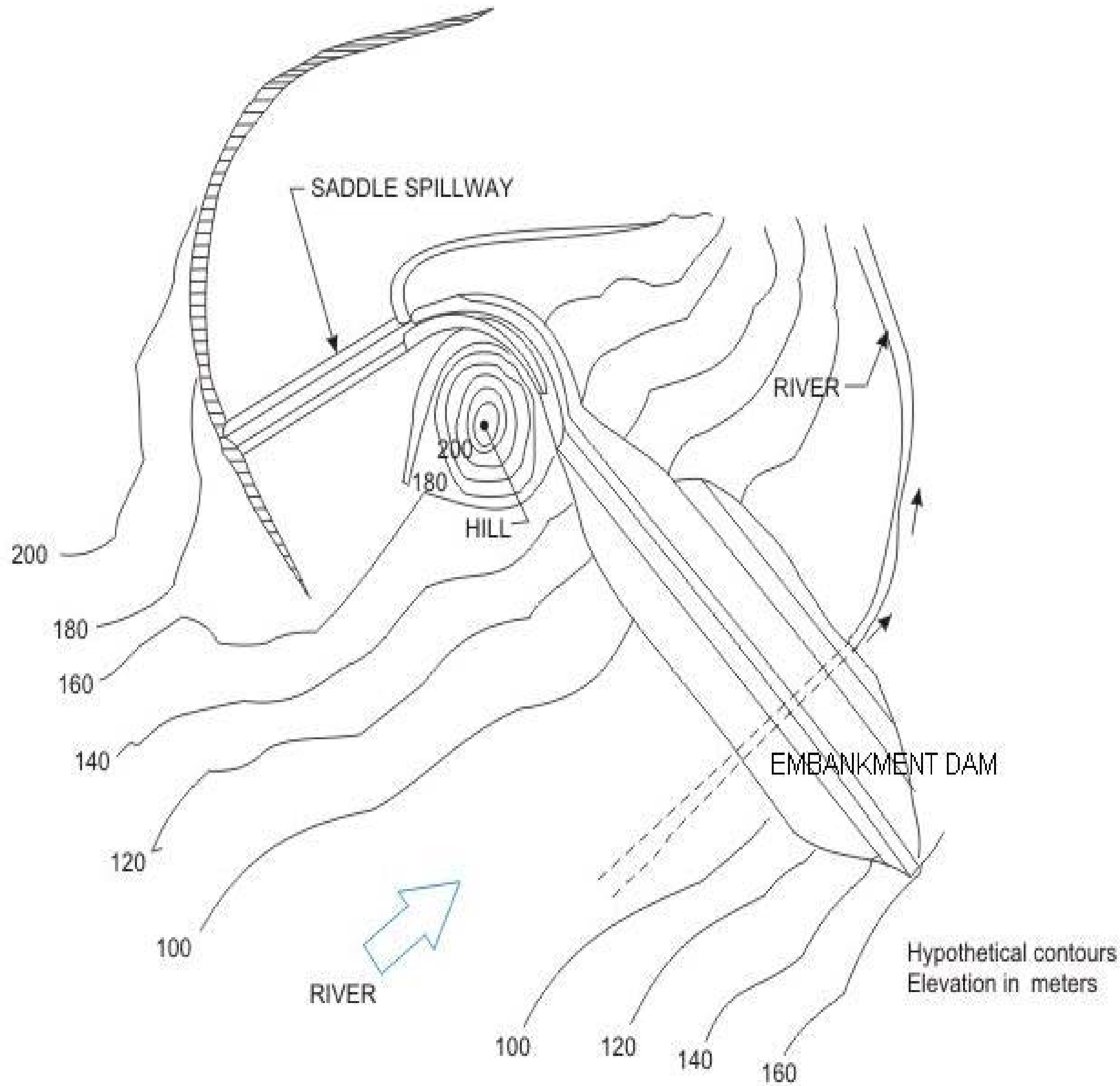


FIGURE 9. Saddle spillway

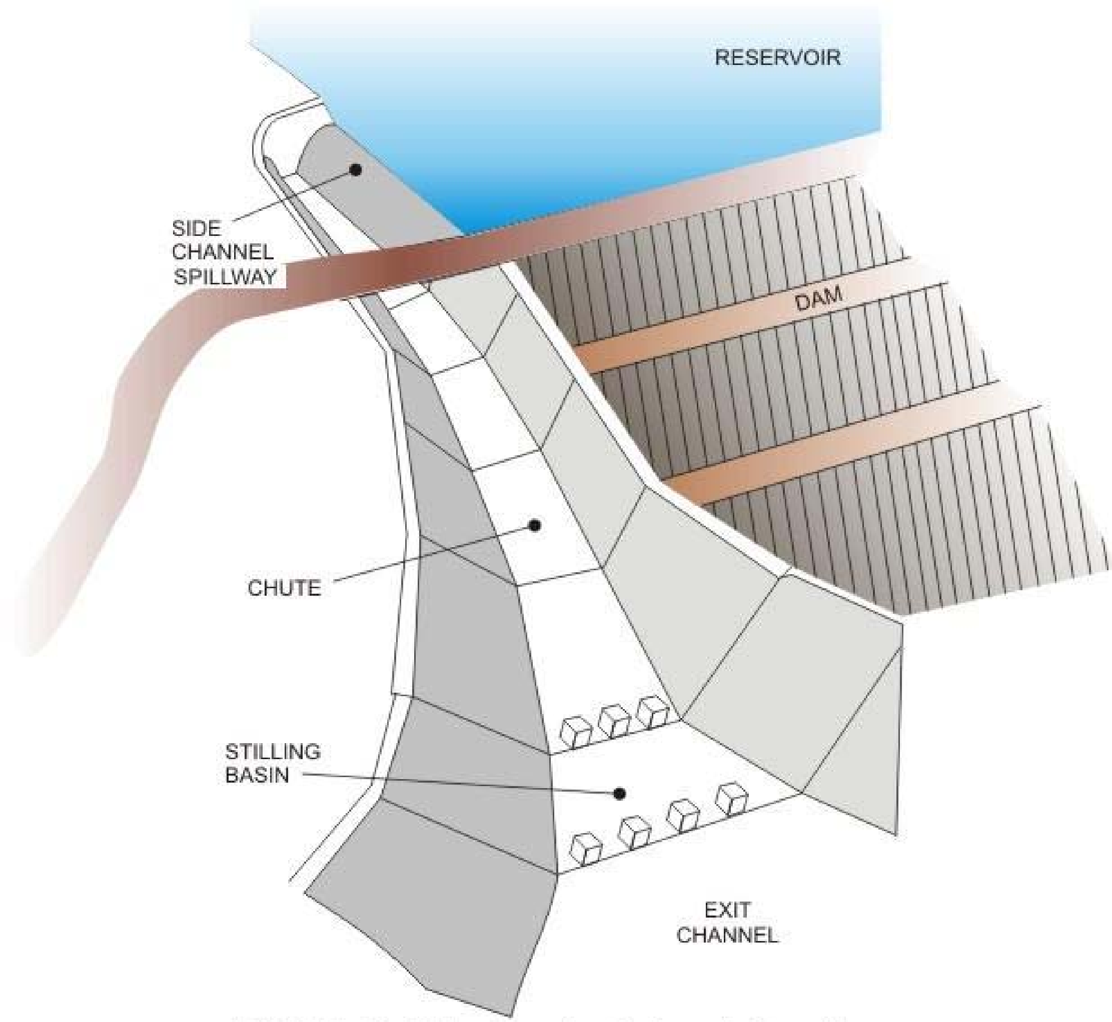


FIGURE 10. Side channel entry to a chute spillway

4.8.4 Side channel Spillway

A side channel spillway is one in which the control weir is placed approximately parallel to the upper portion of the discharge channel, as may be seen from Figure 10. When seen in plan with reference to the dam, the reservoir and the discharge channel, the side channel spillway would look typically as in Figure 11 and its sectional view in Figure 12. The flow over the crest falls into a narrow trough opposite to the weir, turns an approximate right angle, and then continues into the main discharge channel. The side channel design is concerned only with the hydraulic action in the upstream reach of the discharge channel and is more or less independent of the details selected for the other spillway components. Flow from the side channel can be directed into an open discharge channel, as in Figure 10 or 11 showing a chute channel, or in to a closed conduit which may run under pressure or inclined tunnel. Flow into the side channel might enter on only one side of the trough in the case of a steep hill side location or on both sides and over the end of the trough if it is located on a knoll or gently sloping abutment.

Discharge characteristics of a side channel spillway are similar to those of an ordinary overflow spillway and are dependent on the selected profile of the weir crest. Although the side channel is not hydraulically efficient, nor inexpensive, it has advantages which make it adoptable to spillways where a long overflow crest is required in order to limit the afflux (surcharge held to cause flow) and the abutments are steep and precipitous.

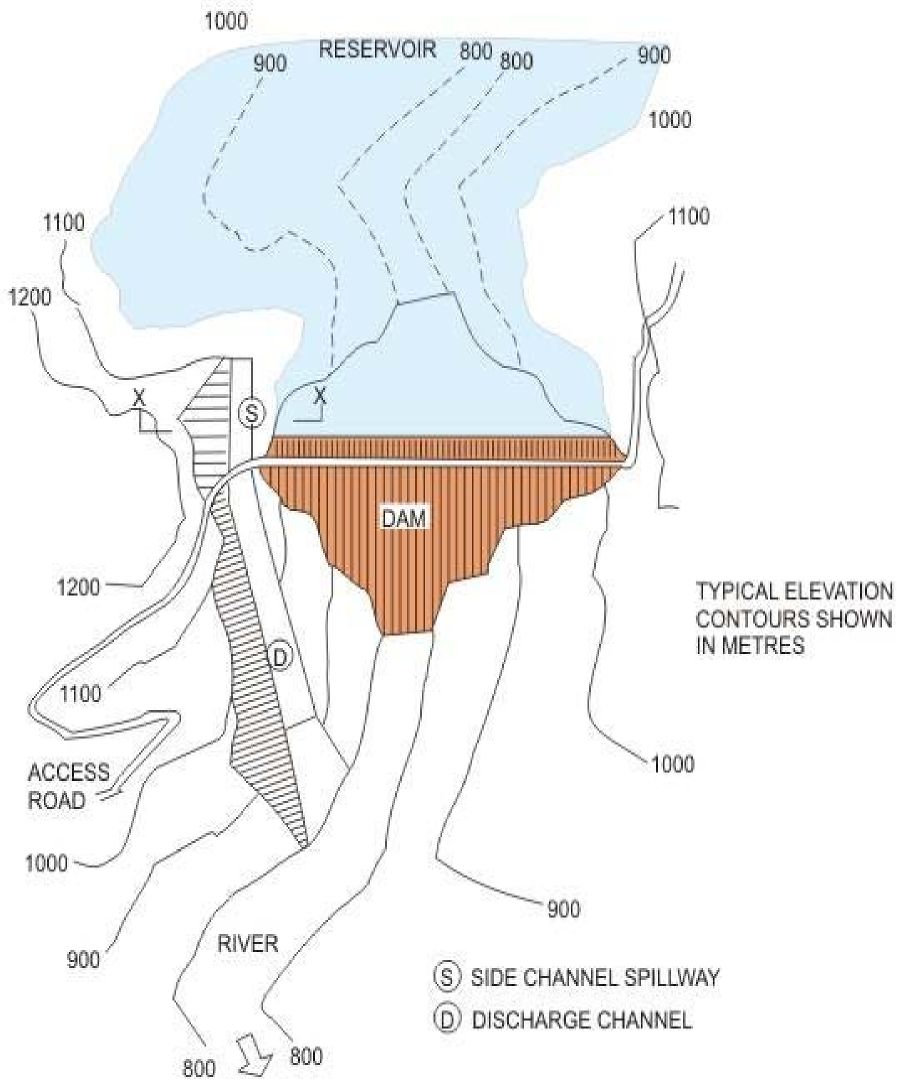


FIGURE 11. Plan of an embankment dam showing side channel spillway and chute channel

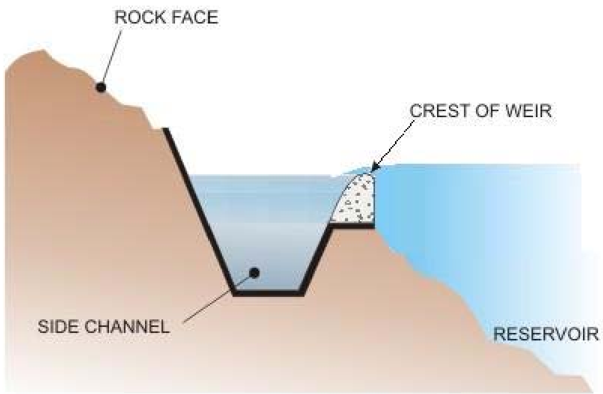


FIGURE 12. Magnified sectional view X-X through the side channel spillway shown in Figure 11

4.8.5 Shaft Spillway

A Shaft Spillway is one where water enters over a horizontally positioned lip, drops through a vertical or sloping shaft, and then flows to the downstream river channel through a horizontal or nearly horizontal conduit or tunnel (Figure 13). The structure may be considered as being made up of three elements, namely, an overflow control weir, a vertical transition, and a closed discharge channel. When the inlet is funnel shaped, the structure is called a Morning Glory Spillway. The name is derived from the flower by the same name, which it closely resembles especially when fitted with anti-vortex piers (Figure 14). These piers or guide vanes are often necessary to minimize vortex action in the reservoir, if air is admitted to the shaft or bend it may cause troubles of explosive violence in the discharge tunnel-unless it is amply designed for free flow.

Discharge characteristics of the drop inlet spillway may vary with the range of head. As the head increases, the flow pattern would change from the initial weir flow over crest to tube flow and then finally to pipe flow in the tunnel. This type of spillway attains maximum discharging capacity at relatively low heads. However, there is little increase in capacity beyond the designed head, should a flood larger than the selected inflow design flood occur.

A drop inlet spillway can be used advantageously at dam sites that are located in narrow gorges where the abutments rise steeply. It may also be installed at projects where a diversion tunnel or conduit is available for use.

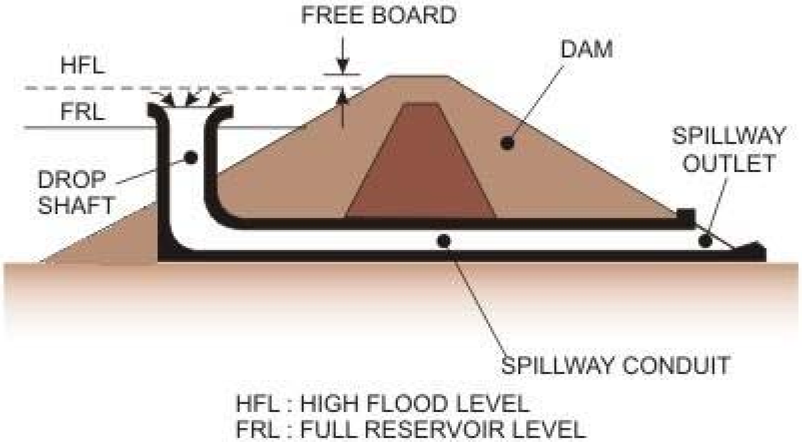


FIGURE 13. Section through a shaft spillway

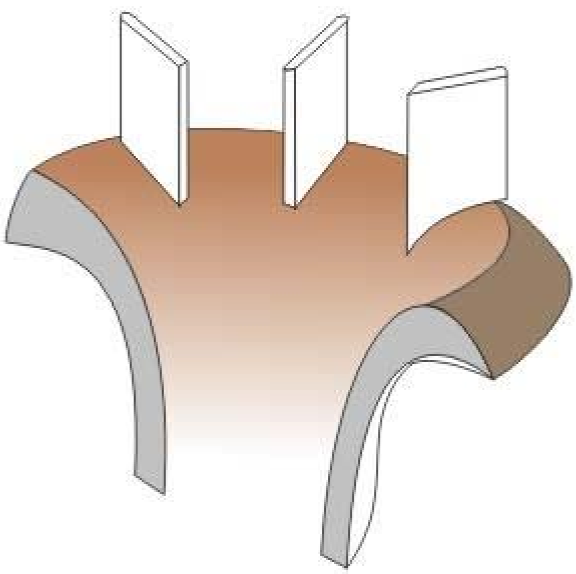


FIGURE 14. Morning glory spillway with anti-vortex piers

4.8.6 Tunnel Spillway

Where a closed channel is used to convey the discharge around a dam through the adjoining hill sides, the spillway is often called a tunnel or conduit spillway. The closed channel may take the form of a vertical or inclined shaft, a horizontal tunnel through earth or rock, or a conduit constructed in open cut and backfilled with earth materials. Most forms of control structures, including overflow crests, vertical or inclined orifice entrances, drop inlet entrances, and side channel crests, can be used with tunnel spillways. Two such examples have been shown in Figs. 15 and 16. When the closed channel is carried under a dam, as in Figure 13, it is known as a conduit spillway.

With the exception of those with orifice or shaft type entrances, tunnel spillways are designed to flow partly full throughout their length. With morning glory or orifice type control, the tunnel size is selected so that it flows full for only a short section at the control and thence partly full for its remaining length. Ample aeration must be provided in a tunnel spillway in order to prevent a fluctuating siphonic action which would result if some part of exhaution of air caused by surging of the water jet, or wave action or backwater.

Tunnel spillways are advantageous for dam sites in narrow gorges with steep abutments or at sites where there is danger to open channels from rock slides from the hills adjoining the reservoir.

Conduit spillways are generally most suited to dams in wide valleys as in such cases the use of this types of spillway would enable the spillway to be located under the dam very close to the stream bed.

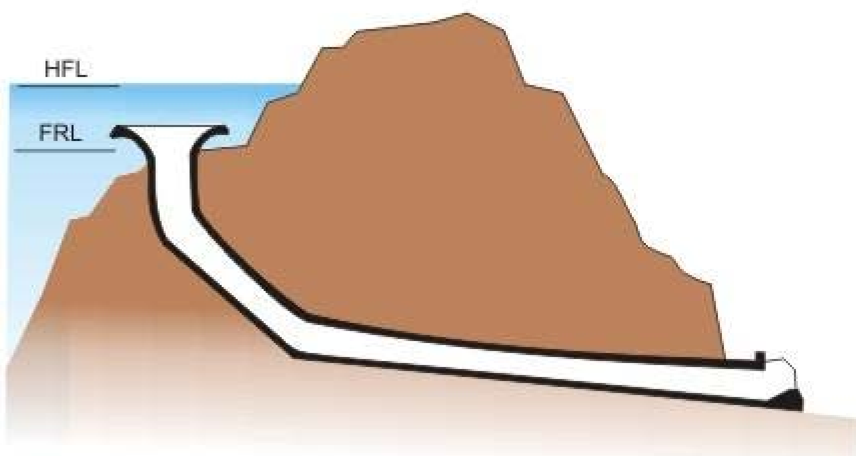


Figure 15. Tunnel spillway with a morning glory entrance.

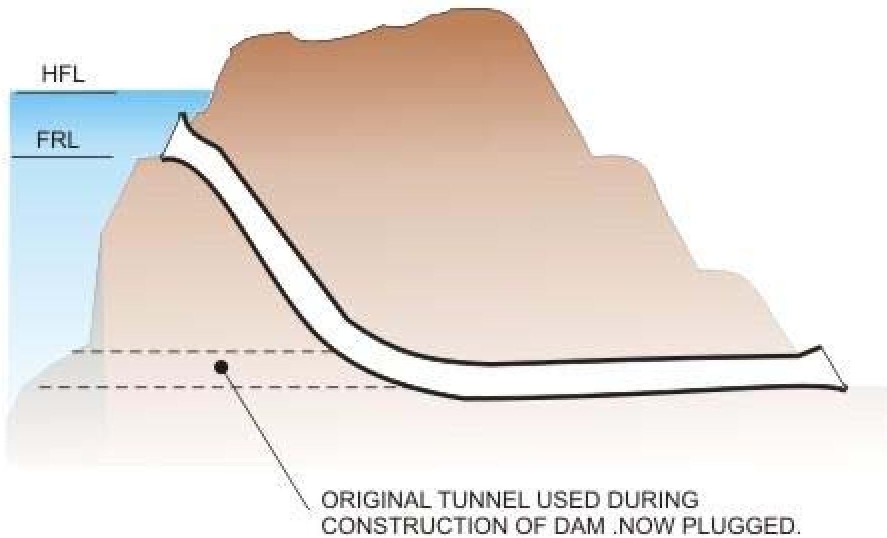


FIGURE 16. Bell mouth entry to a tunnel spillway (Gate control at entrance not shown)

4.8.7 Siphon Spillway

A siphon spillway is a closed conduit system formed in the shape of an inverted U, positioned so that the inside of the bend of the upper passageway is at normal reservoir storage level (Figure 17). This type of siphon is also called a Saddle siphon spillway. The initial discharges of the spillway, as the reservoir level rises above normal, are similar to flow over a weir. Siphonic action takes place after the air in the bend over the crest has been exhausted. Continuous flow is maintained by the suction effect due to the gravity pull of the water in the lower leg of the siphon.

Siphon spillways comprise usually of five components, which include an inlet, an upper leg, a throat or control section, a lower leg and an outlet. A siphon breaker air vent is also provided to control the siphonic action of the spillway so that it will cease operation when the reservoir water surface is drawn down to normal level. Otherwise the siphon would continue to operate until air entered the inlet. The inlet is generally placed well below the Full Reservoir Level to prevent entrance of drifting materials and to avoid the formation of vortices and draw downs which might break siphonic action.

Another type of siphon spillway (Figure 18) designed by Ganesh Iyer has been named after him. It consists of a vertical pipe or shaft which opens out in the form of a funnel at the top and at the bottom it is connected by a right angle bend to a horizontal outlet conduit. The top or lip of the funnel is kept at the Full Reservoir Level. On the surface of the funnel are attached curved vanes or projections called the volutes.

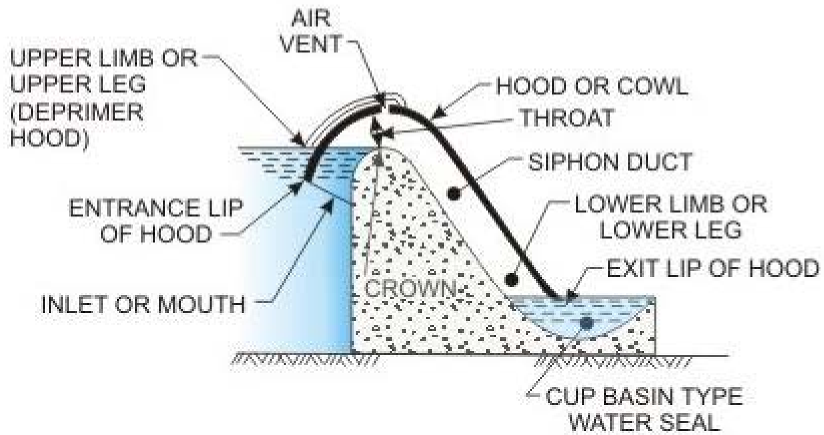
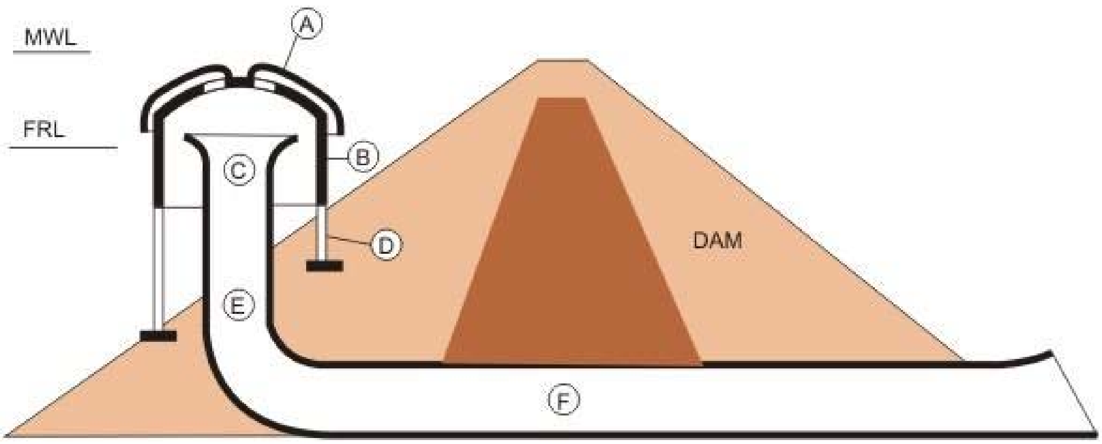


FIGURE 17. Saddle Siphon



A : DOME
B : DRUM
C : FUNNEL
D : PILLAR SUPPORTS
E : SHAFT
F : CONDUIT

FIGURE 18. Volute of siphon spillway components

4.8.8 Special types of spillways

Apart from the commonly used spillways, a few other types of spillways are used sometimes for a project, which are explained below.

Saddle Spillway

In some basins formed by a dam, there may be one or more natural depressions for providing spillway. They are sometimes preferred for locating main spillway or emergency or auxiliary spillways. A site which has a saddle is very desirable and economical, if the saddle is suitable for locating the spillway. An example for such a spillway may be seen in Figure 9.

Fuse plug

It may be a simple earth bank, flash board or other device designed to fail when overtopped. Such plugs may be used where the sudden release of a considerable volume of water is both safe and not over destructive to the environment. “For example, the saddle spillway of Figure 9 may be constructed as an earthen embankment dam, with its crest at a slightly higher elevation than the High Flood Level (HFL) of the reservoir. In the occurrence of a flood greater than the design flood which may cause rise in the reservoir water would overtop the earthen embankment dam and cause its collapse and allow the flood water to safely pass through the saddle spillway.

Sluice Spillway

The use of large bottom openings as spillways is a relatively modern innovation following the greater reliance on the safety and operation of modern control gates under high pressure. A distinct advantage of this type of spillway is that provision can usually be made for its use for the passage of floods during construction. One disadvantage is that, once built, its capacity is definite whereas the forecasting of floods is still indefinite. A second disadvantage is that a single outlet may be blocked by flood debris, especially where in flow timber does not float. Figure 20 shows an example of a sluice spillway.

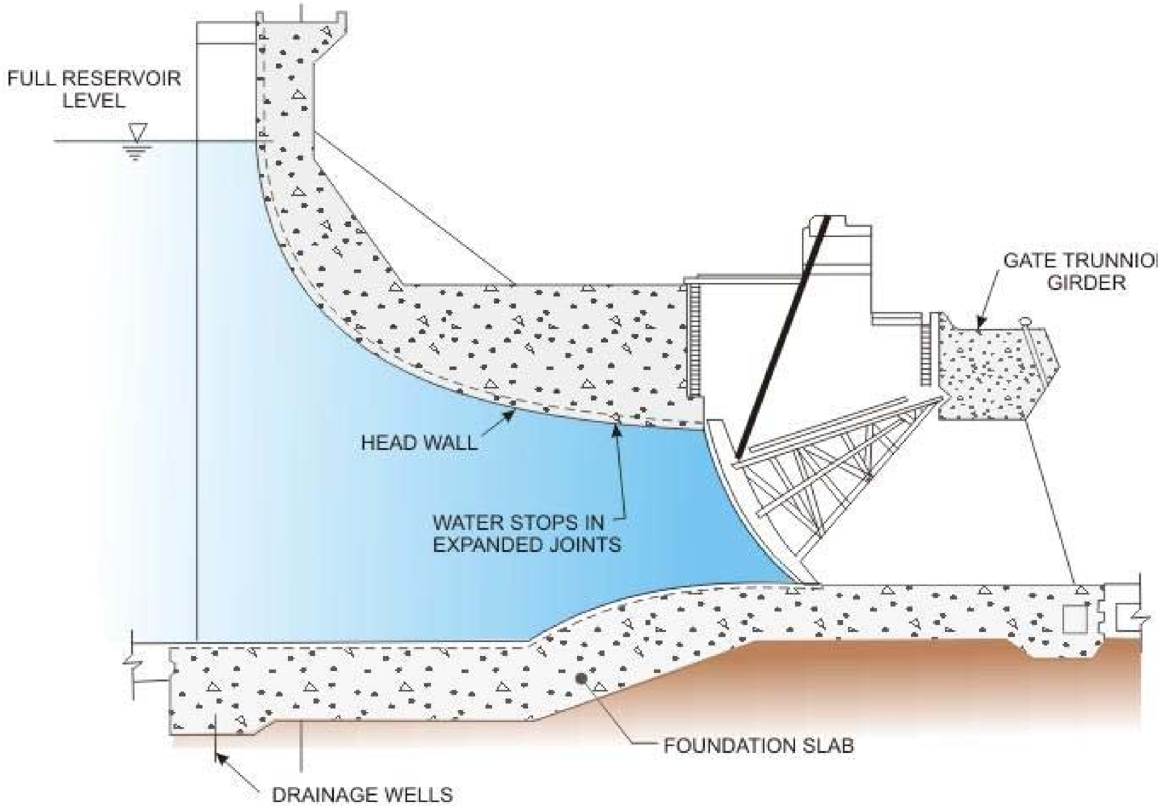


FIGURE 19. Sluice spillway

Duck-bill Spillway

This is a spillway with a rectangular layout projections into the reservoir comprising three straight overflow lengths intersecting at right angles. The layout could be trapezoidal in which case the corner angles will be other than 90 degree. The flow from the three reaches of the spillway interacts in the trough portion and is further conveyed through a discharge carrier on to a terminal structure to provide for energy dissipation. An example of this type of spillway is shown in Figure 21.

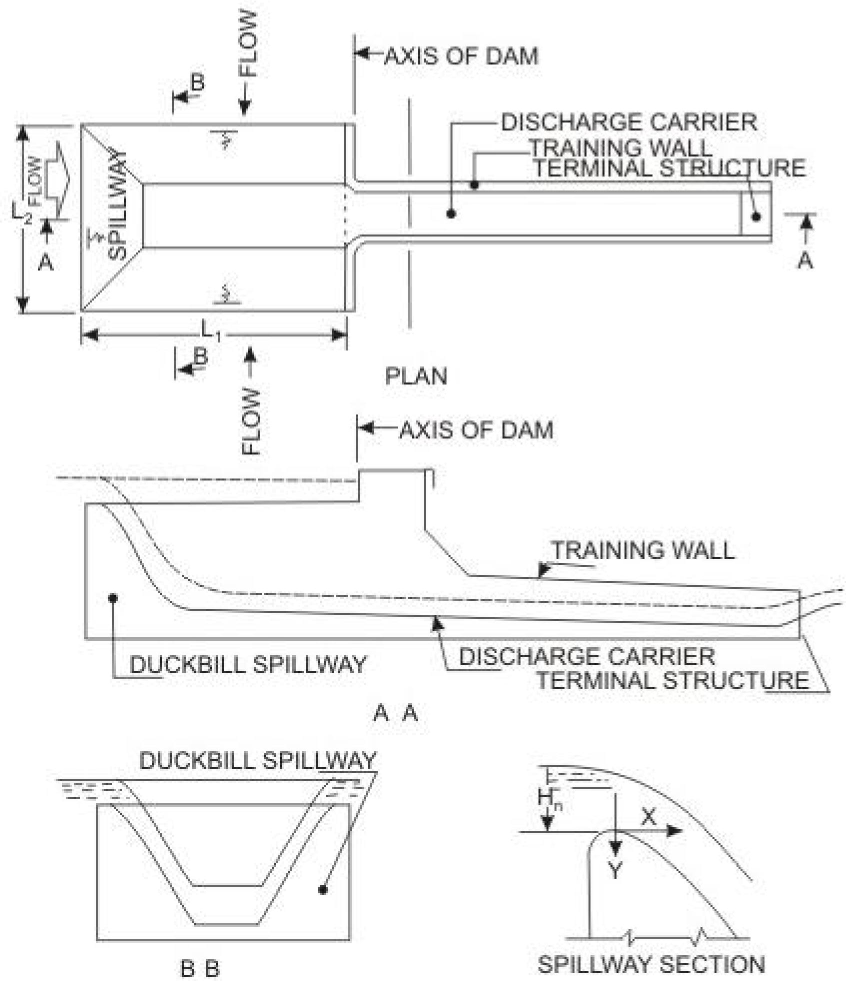


FIGURE 20. Duckbill Spillway

4.8.9 Shape and Hydraulics of Ogee-Crest

Crest shape

The ogee shaped crest is commonly used as a control weir for many types of spillways-Overflow (Figure 5), Chute (Figure 8), Side Channel (Figure 12) etc. The ogee shape which approximates the profile of the lower nappe of a sheet of water flowing over a sharp-crested weir provides the ideal form for obtaining optimum discharges. The shape of such a profile depends upon the head, the inclination of the upstream face of the flow section, and the height of the overflow section above the floor of the entrance channel (which influences the velocity of approach to the crest). The ogee profile to be acceptable should provide maximum possible hydraulic efficiency, structural stability and economy and also avoid the formation of sub atmospheric pressures at the surface (which may induce cavitations).

Ogee crested control structures are also sensitive to the upstream shape and hence, three types of ogee crests are commonly used and shown in Figure 21. These are as follows:

1. Ogee crests having vertical upstream face
2. Ogee crests having inclined upstream face
3. Ogee crests having over hang on up stream face

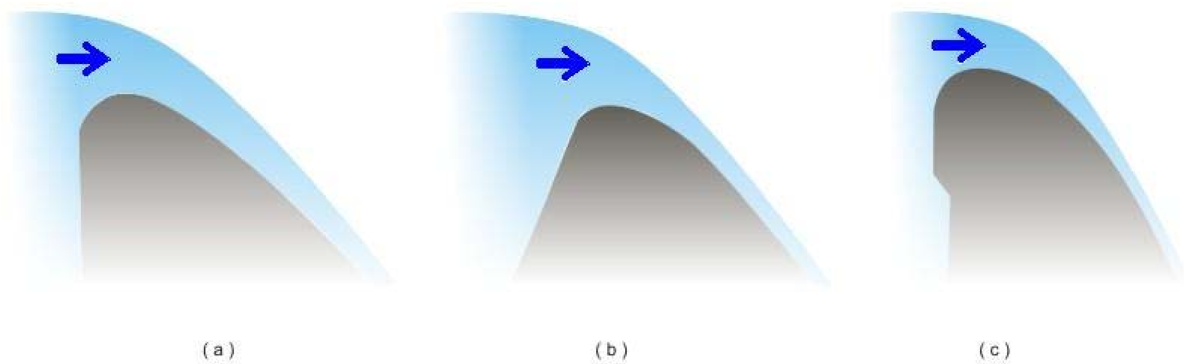


FIGURE 21. Ogee crest control weirs with
(a) Vertical upstream face
(b) Inclined upstream face
(c) Overhangs on the upstream

However, the same general equations for the up stream and down stream quadrants are applicable to all the three cases, as recommended by the Bureau of Indian Standards code IS: 6934-1998 “Hydraulic design of high ogee over flow spillways-recommendations” and are outlined in the following paragraphs.

1. *Ogee crests with vertical upstream face*

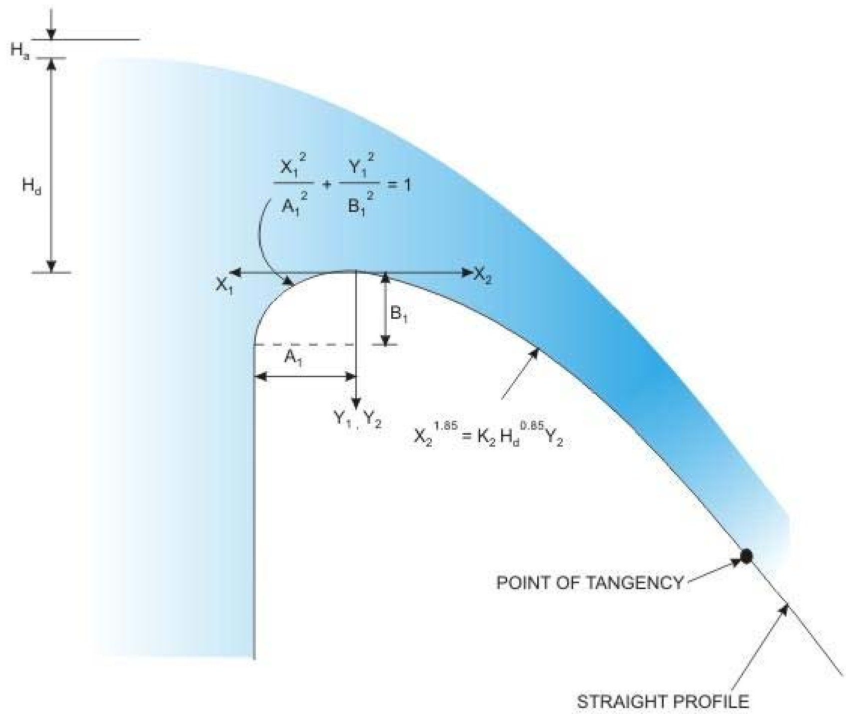


FIGURE 22. Ogee crest shape with vertical upstream wall . Coefficients to be determined from Figure 21

The upstream quadrant of the crest (Figure 22) may confirm to the equation of an ellipse as given below:

$$\frac{x_1^2}{A_1^2} + \frac{y_1^2}{B_1^2} = 1 \qquad (1)$$

Where the values of A_1 and B_1 may be determined from the graphs given in (Figure 23).

The downstream profile of the ogee crest may confirm to the following equation:

$$x_2^{1.85} = K_2 H_d^{0.85} Y_2 \qquad (2)$$

Where the magnitude of K_2 may be read from the relevant graph shown in Figure 23.

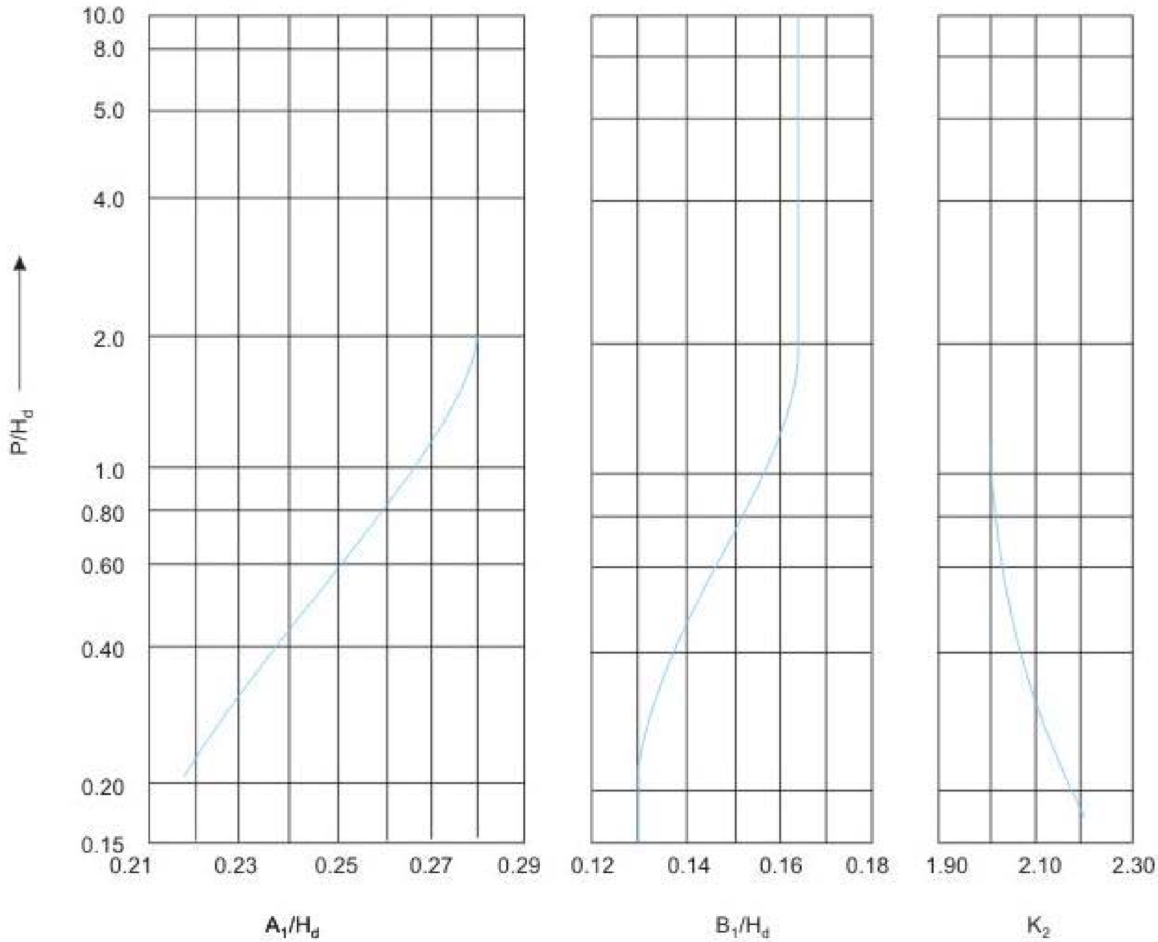


FIGURE 23. Coefficients for Figure 22

2. *Ogee crests with sloping up stream face*

In this case, the desired inclination of the upstream face is made tangential to the same elliptical profile as provided for a crest with a vertical face. The down stream face equation remains unchanged.

3. *Ogee crests with overhang*

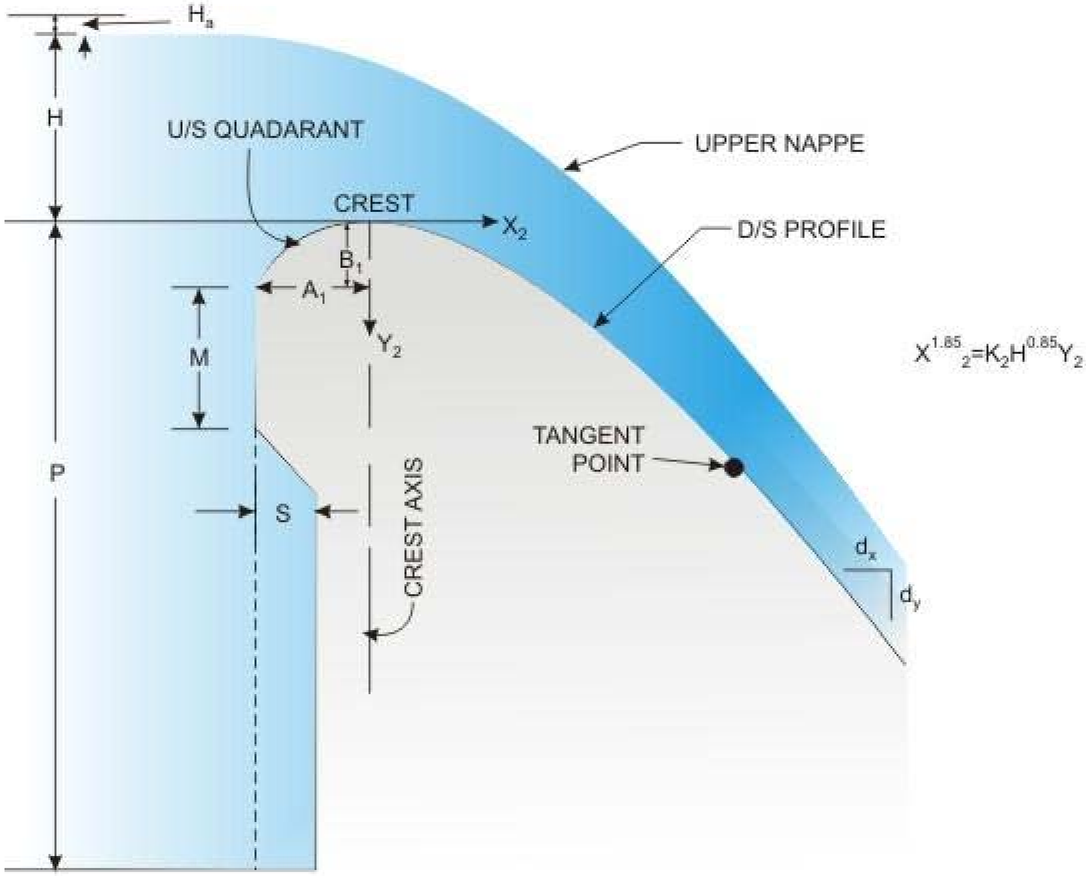


FIGURE 24. Overhang details of ogee crest

Whenever structural requirements permit, the upstream vertical face of an ogee crested spillway (Figure 22) may be offset inside, (Figure 24). It is recommended that the ratio of the rises M to the design head H_d, should be at least 0.6 or greater for flow conditions to be stable. The crest shapes on the up stream and downstream may be provided the same as for an ogee crest with vertical up stream wall if the condition M/ H_d >0.6 is satisfied.

Discharge characteristics of ogee crests-uncontrolled flow

For an ogee crested control weir for a spillway without any control with a gate, the free flow discharge equation is given as

$$Q = C_d L_e H_e^{3/2} \qquad (3)$$

Where Q is the discharge (in m^3/s), C_d is the coefficient of discharge, L_e is the effective length of crest (in m), including velocity of approach head. The discharge coefficient, C_d, is influenced by a number of factors, such as:

1. Depth of approach
2. Relation of the actual crest shape to the ideal nappe shape
3. Upstream face slope
4. Downstream apron interference, and
5. Downstream submergence

The effect of the above mentioned factors on the variation of discharge and calculation for effective length are mentioned in the following paragraphs.

1. Effect of depth of approach

For a high sharp-crested ogee shaped weir, as that of a Overfall spillway of a large dam, the velocity of approach is small and the lower nappe flowing over the weir attains maximum vertical contraction. As the approach depth is decreased, the velocity of approach increases and the vertical contraction diminishes. For sharp-crested weirs whose heights are not less than about one-fifth of the head producing the flow, the coefficient of discharge remains fairly constant with a value of about 1.82 although the contraction diminishes. For weir heights less than about one-fifth the head, the contraction of the flow becomes increasingly suppressed and the crest coefficient decreases. This is the case of an ogee crested chute spillway control section. When the weir height becomes zero, the contraction is entirely suppressed and the weir turns into a broad crested one, for which the theoretical coefficient of discharge is 1.70. The relationship of the ogee crest coefficient of discharge C_d for various values of P/H_d where P is the height of the weir above base and H_d is the design head, is given in Figure 25. The coefficients are valid only when ogee is formed to the ideal nappe shape.

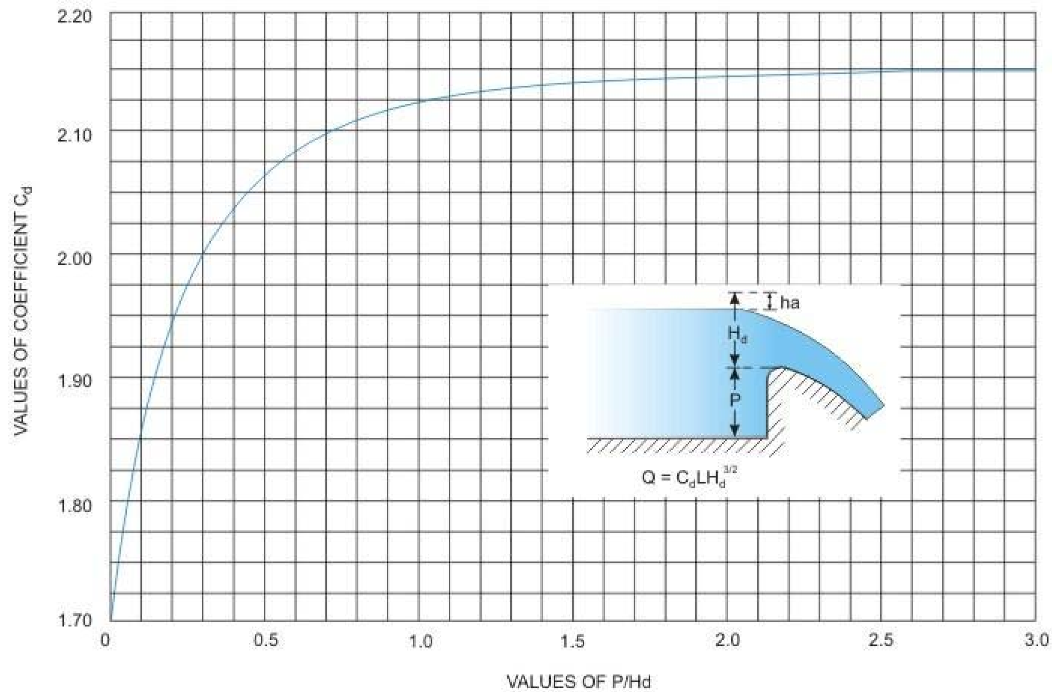


FIGURE 25. Coefficient of discharge (C_d) variation due to height of a vertical faced ogee crest

2. Effect of the crest shape differing from the ideal nappe shape

When the ogee crest is formed to a shape differing from the ideal nappe shape or when the crest has been shaped for a head larger or smaller than the one under consideration, the coefficient of discharge will differ from that given in the previous section. A wider crest shape will reduce the coefficient of discharge while a narrower Crest Shape will reduce the coefficient. The application of this concept is required to deduce the discharge flowing over a spillway when the flow is less or more than the design discharge. The variation of the coefficient of discharge in relation to H/H_d, where H is the actual head and H_d is the design head, is shown in Figure 26.

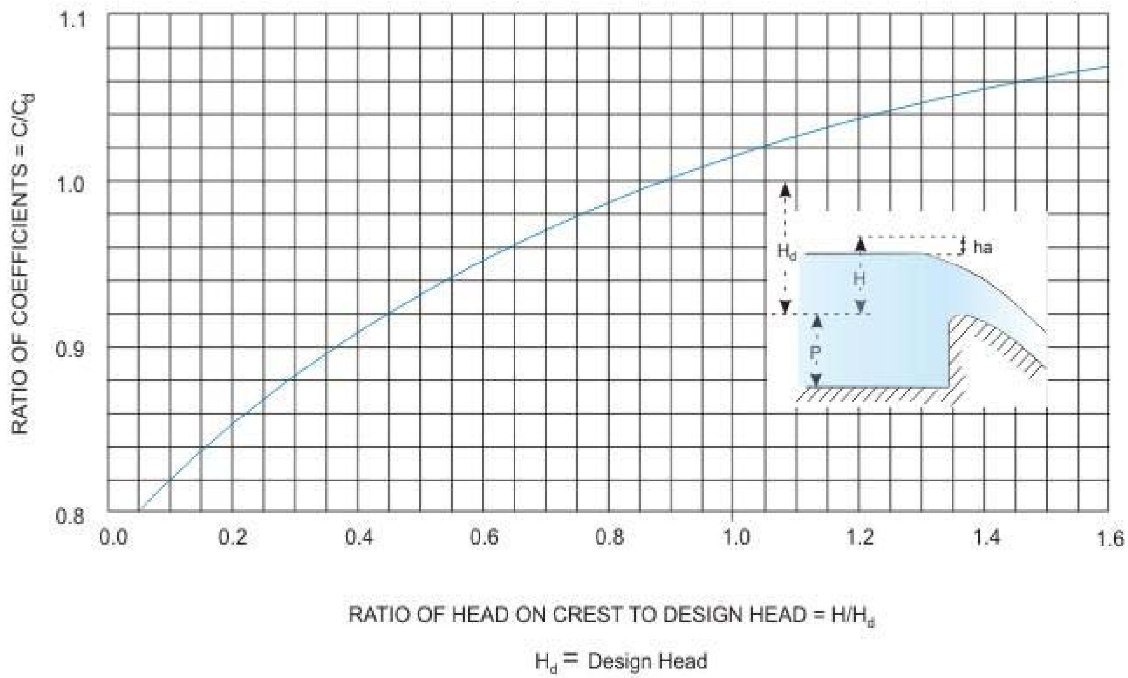


FIGURE 26. Coefficient of discharge other than the design head

3. Effect of upstream face slope

For small ratio of P/H_d where P is the height of the weir and H_d the design head, as for the approach to a chute spillway, increase of the slope of upstream face tends to increase the coefficient of discharge, as shown in Figure 27. This figure shows the ratio of the coefficient for ogee crest with a sloping face to that with vertical face. For large ratios of P/H_d, the effect is a decrease of the coefficient. The coefficient of discharge is reduced for large ratios P/H_d only for relatively flat upstream slopes.

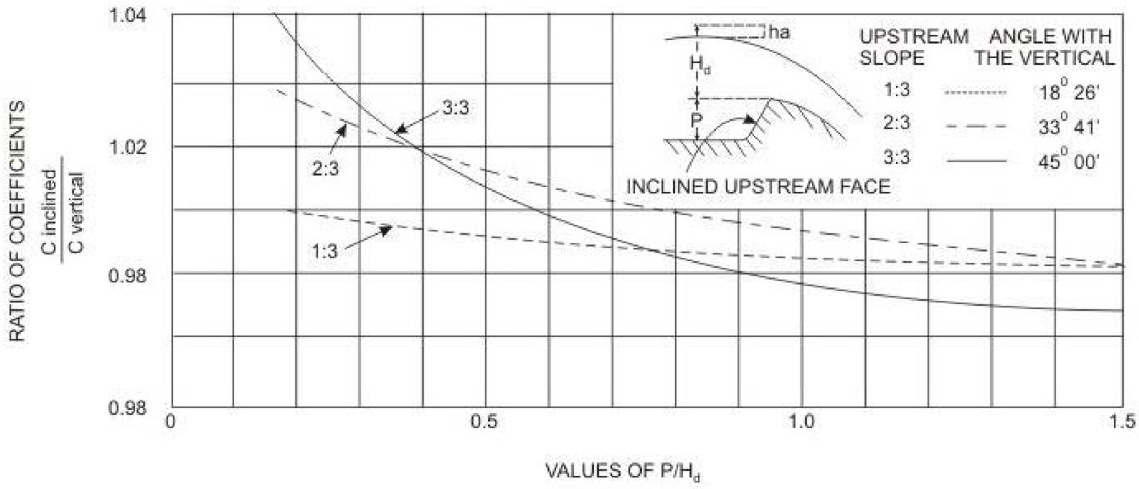


Figure 27. Coefficient of discharge variation with upstream face inclination.

4. Effect of downstream apron interference and downstream submergence

This condition is possible for dams of relatively small heights compared to the natural depth of the river, when the water level downstream of the weir crest is high enough to affect the discharge, the condition being termed as submerged. The conditions that after the coefficient of discharge in this case are the vertical distance from the crest of the over flow to the downstream apron and the depth of flow in the downstream channel, measured above the apron.

Five distinct characteristic flow conditions can occur below an overflow crest, depending on the relative positions of the apron and the downstream water surface:

A. The flow will continue at supercritical stage
B. A partial or incomplete hydraulic jump will occur immediately downstream from the crest
C. A true hydraulic jump will occur
D. A drowned jump will occur in which the high-velocity jet will follow the face of the overflow and then continue in an erratic and fluctuating path for a considerable distance under and through the slower water, and
E. No jump will occur - the jet will break away from the face of the overflow and ride along the surface for a short distance and then erratically intermingle with the slow moving water underneath.

According to USBR (1987), the relationship of the floor positions and downstream submergences which produce these distinctive flows can be shown in a graph as in Figure 28.

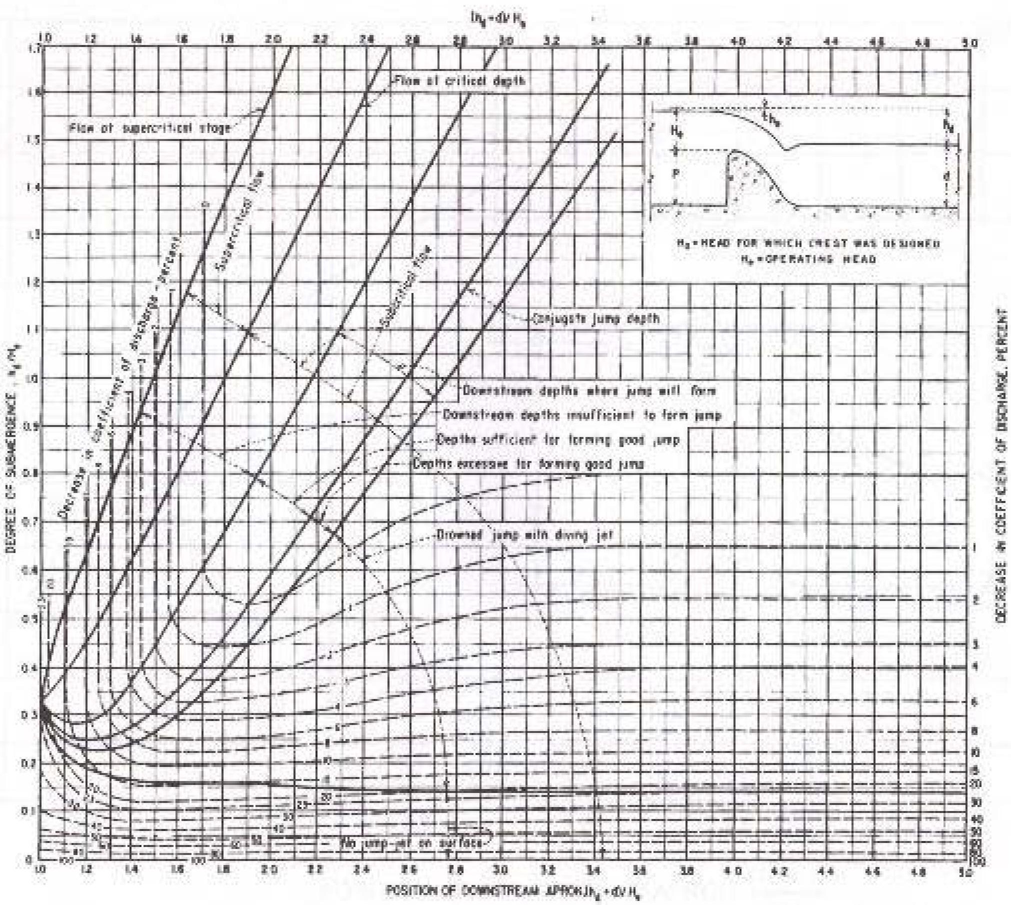


FIGURE 28. Effects of downstream influences on flow over weir crests

(Courtesy: United States Bureau of Reclamation, Design of Small Dams)

Usually for large dams the cases A,B or C with dominate and the decrease in the coefficient of discharge is due principally to the back pressure effect of the downstream apron and is independent of any submergence effect due to tail water. Cases D and E can be expected t o be found in low-height dams like small height diversion or navigation dam. Figure 29, adapted from USBR (1987), shows the effect of downstream apron conditions on the coefficient of discharge. It may be noted that this curve plots the same data represented by the vertical dashed lines of Figure 28 in a slightly different form. As the downstream apron level nears the crest of the overflow ($\frac{h_d + d}{H_e}$ approaches 1.0), where h_d is the difference of total energy on upstream and the water level downstream, d is the downstream water depth and H_e is the total energy upstream measured above the crest of the weir, the coefficient of discharge is about 77 percent of that for un-retarded flow. From Figure 29 it can be seen that when the ratio of $\frac{h_d + d}{\mathrm{H}_e}$ values exceed about 1.7, the downstream floor position has little effect on the coefficient, but there is a decrease in the coefficient caused by tail water submergence. Figure 30 shows the ratios of the coefficient of discharge where affected by tailwater conditions, to that coefficient for free flow conditions. This curve plots the data represented by the horizontal dashed lines on Figure 28 in a slightly different form. Where the dashed lines of Figure 28 are curved, the decrease in the coefficient is the result of a combination of tail-water effects and downstream apron position.

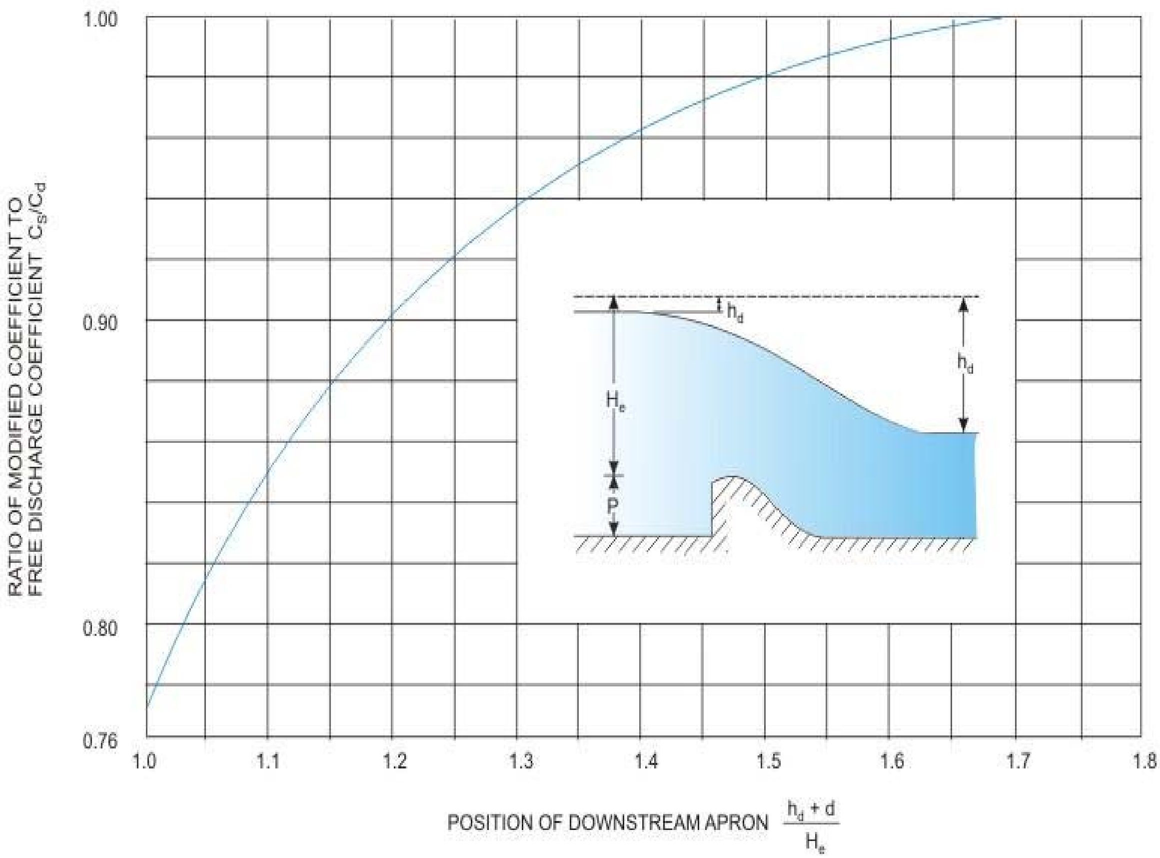


FIGURE 29. Ratio of discharge coefficients resulting from the effect of the apron on flow

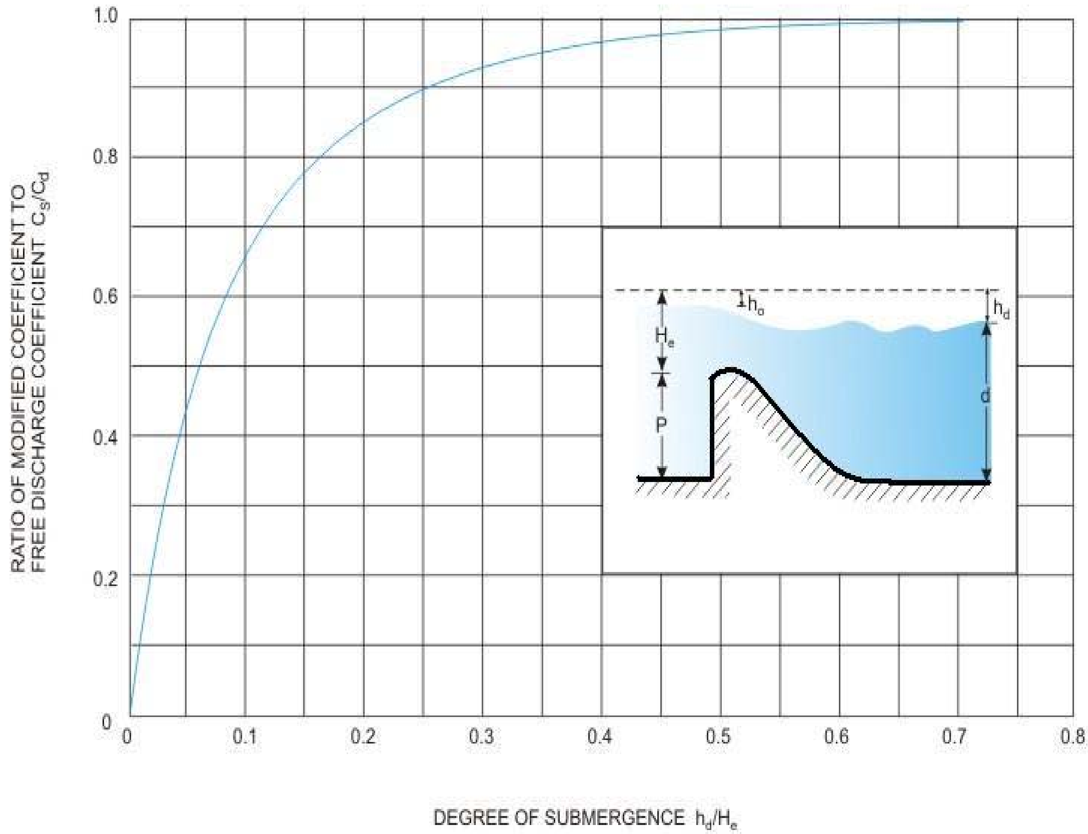


FIGURE 30. Ratio of discharge coefficients caused by tailwater effects

If the ordinate of Figure 30 is changed from $\frac{h_d}{H_e}$ to $1-\frac{h_d}{H_e}$, that is, equal to $\frac{H_e - h_d}{H_e} = \frac{h}{H_e}$ where *h* is the downstream water depth measured above crest, then the curve of Figure 30 may be transposed as in Figure 31.

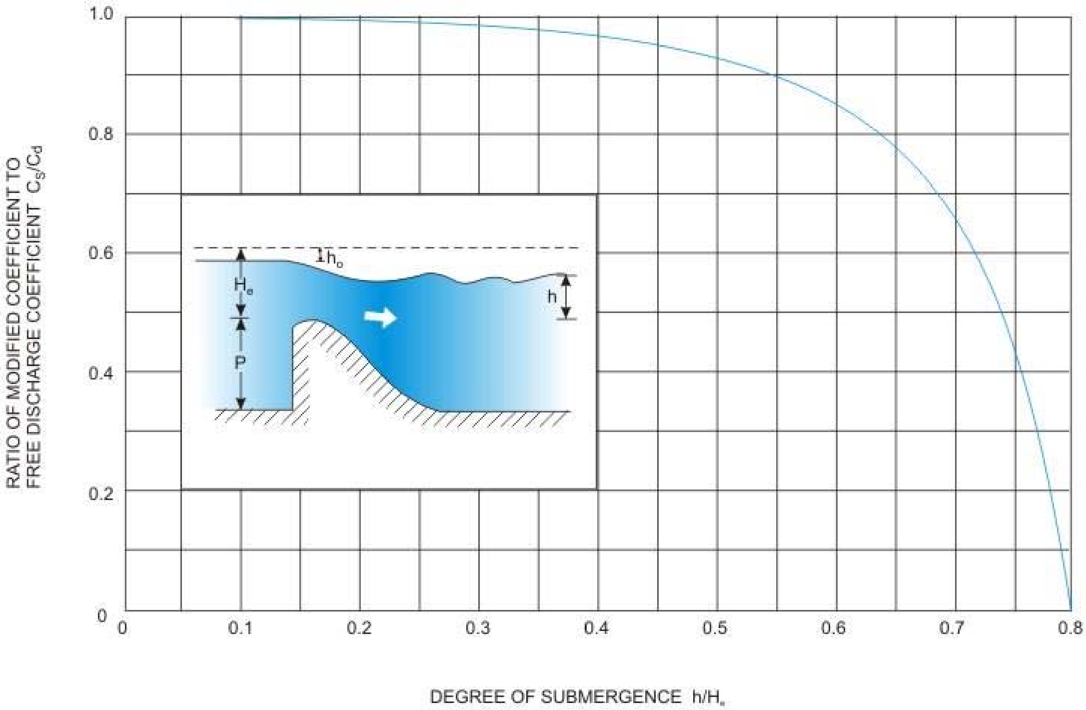


FIGURE 31. Ratio of submerged discharge coefficient to that without submergence effect
This figure is similar to that given in FIGURE 30, but with different ordinate value

The total head on the crest He, does not include allownces for approach channel friction losses due to curvature into the inlet section, and inlet or transition losses. Where the design of the approach channel results in appreciable losses, they must be added to H_e to determine reservoir elevations corresponding to the discharges given by the discharge equation.

Where the crest piers and abutments are shaped to cause side contractions of the overflow, the effective length, L_e, will be the net length of the crest, L. The effect of the end contractions may be taken into account by reducing the net length of crest as given below:

$$L_e = L - 2\left(N \cdot \mathrm{K}_p + \mathrm{K}_a\right) \cdot \mathrm{H}_e$$

Where L, L_e and H have been explained before, N is the number of piers and K_p and K_a are the pier and abutment contraction coefficients. The reason for the reduction of the net length may be appreciated from Figure 32.

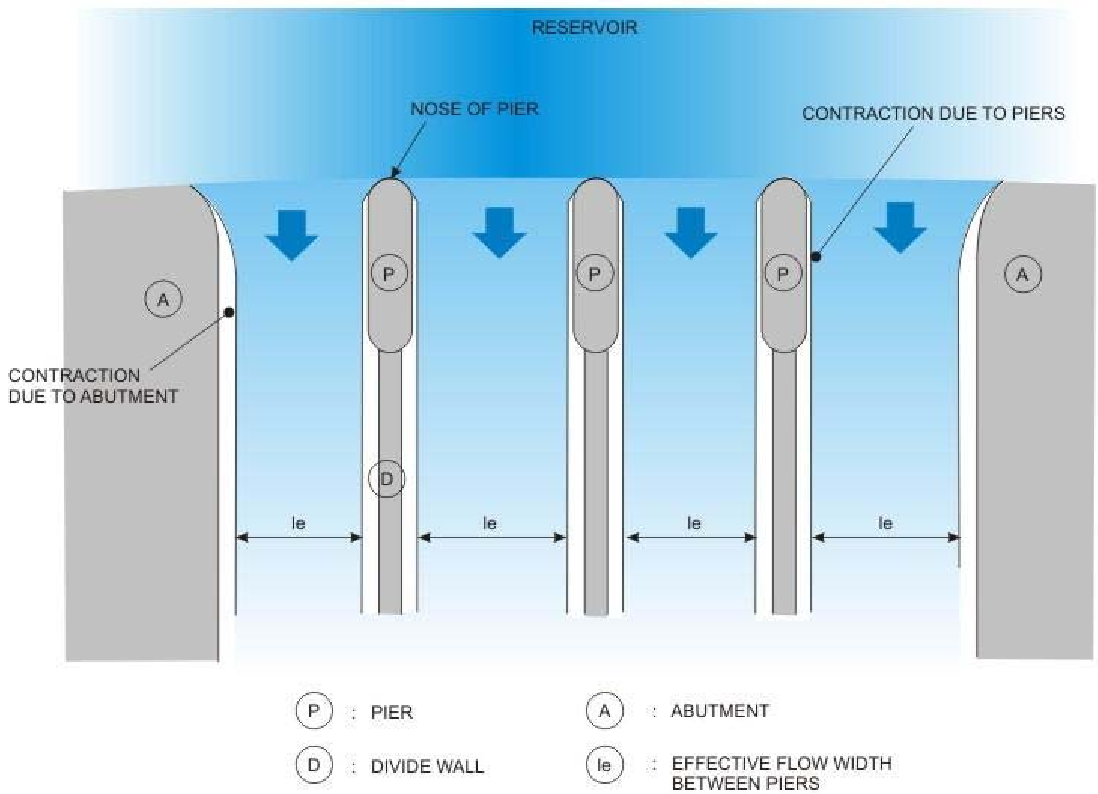


FIGURE 32. Abutment and pier contractions shown on a spillway plan

The pier contraction coefficient K_p depends upon the following factors:

1. Shape and location of the pier nose
2. Thickness of the pier
3. Head in relation to the design head
4. Approach velocity

For the condition of flow at the design head, the average values of pier contraction coefficients may be assumed as shown in Figure 33.

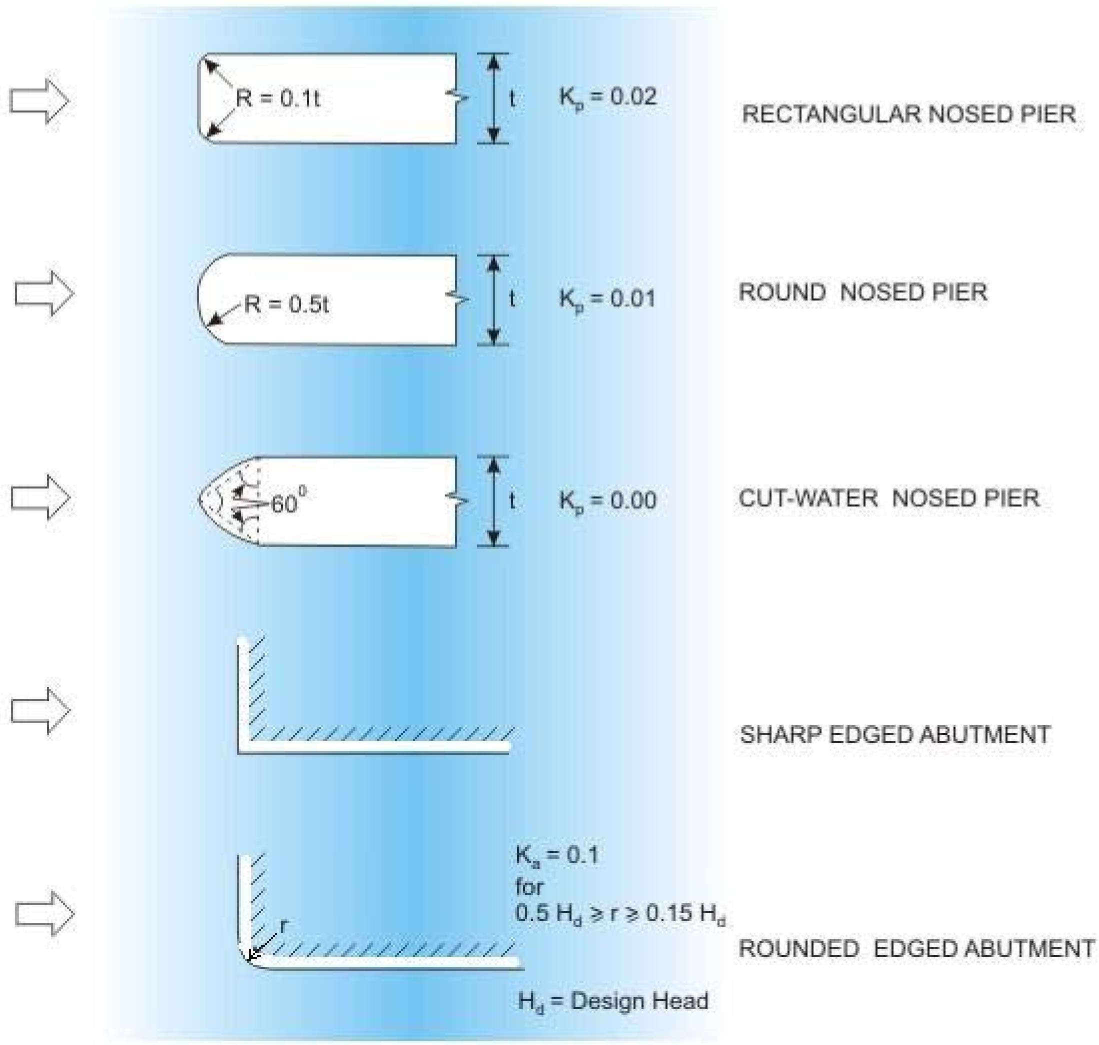


FIGURE 33. Recommended values of K_p and K_a

The abutment contraction coefficient is seen to depend upon the following factors:

1. Shape of abutment
2. Angle between upstream approach wall and the axis of flow
3. Head, in relation to the design head
4. Approach velocity

For the condition of flow at the design head, the average value of abutment contraction coefficients may be assumed as shown in Figure 33.

For flow at head other than design head, the values of K_p and K_a may be obtained from graphical plots given in IS: 6934-1973 “Recommendations for hydraulic design of high ogee overflow spillways”.

Discharge characteristics of ogee crests-controlled spillway

The discharge for gated crests at partial gate opening is similar to flow through a low-head orifice and may be computed by the following equation recommended by the Bureau of Indian Standards code IS:6934-1998 "Hydraulic design of high ogee overflow spillways-recommendations".

$$Q = C_g \cdot G_0 \cdot L_e \sqrt{2g} H_e \tag{4}$$

Where Q is the discharge (in m^3/s), C_g is the gated coefficient of discharge, G_0 is the gate opening (in m), L_e is the effective length of crest, g is the acceleration due to gravity, and H_e is the hydraulic head measured from the centre of the orifice (in m).

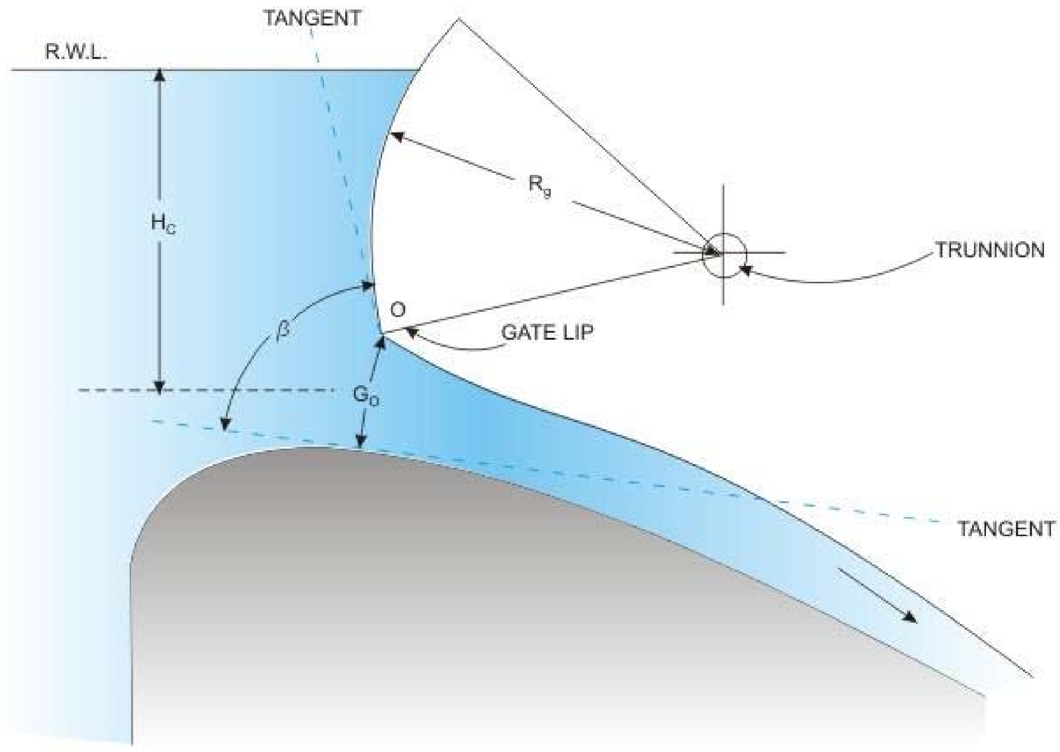


FIGURE 34. Partially opened radial gate discharging flow

Usually for high head spillways, radial gates are common and Figure 34 shows the position of a partially opened radial gate over an ogee-crested spillway. The gate opening G_0 may be seen to be measured as the shortest distance from the gate lip to the ogee crest profile meeting at G. The angle β is seen to be measured between the tangent at G and the tangent of the radial gate at gate lip. Figure 35 presents a curve relating the coefficient of gated discharge C_g with the angle β.

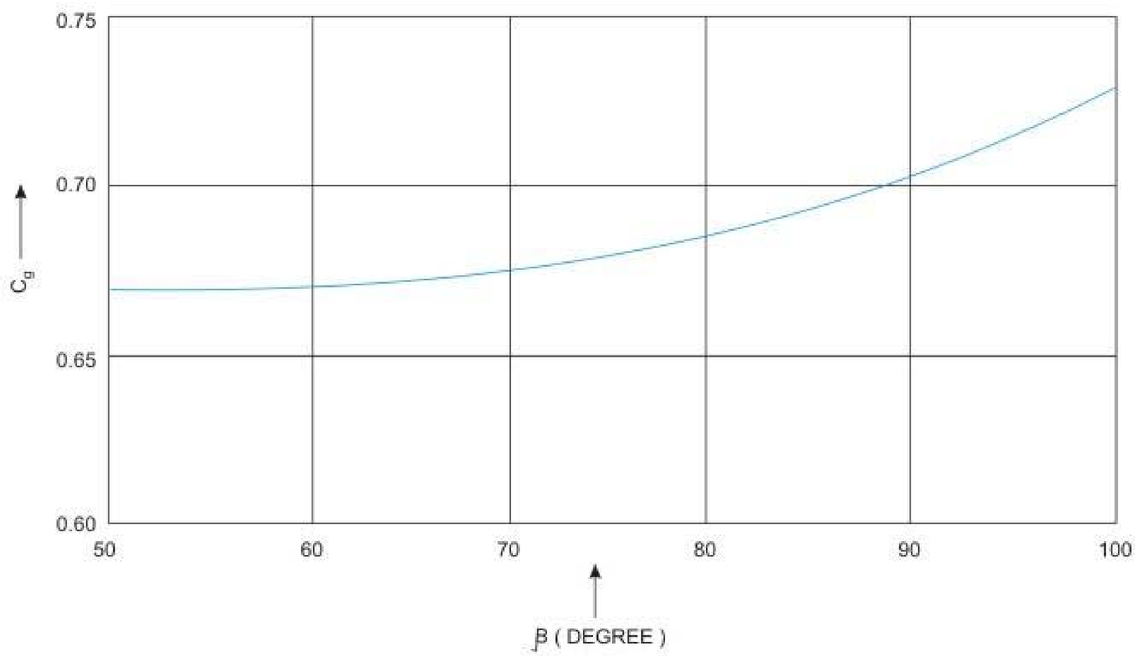


FIGURE 35. Coefficient of gated discharge Cg variation with lip angle β

The curve presents an average value of C_g determined for various approach and downstream conditions and may be used for preliminary design purposes. In fact, it may be noticed that the discharge equation mentioned above for calculating flow through a gated spillway as recommended in IS: 6934-1998 may not be strictly correct as the gate opening becomes larger, comparable to the hydraulic head H_c.

4.8.10 Spillway profile with breast wall

Spillways, generally the ogee-crested type, are sometimes provided with a breast wall from various considerations such as increasing the regulating storage of flood discharge, reducing the height of the gate, minimizing the cost of gate operating mechanism, etc.

For the spillways with breast wall, the following parameters are required to be determined:

a) Profile of the spillway crest including the upstream and downstream quadrants,
b) Profile of the bottom surface of the breast wall, and
c) Estimation of discharge efficiency of the spillway.

The flow through a spillway with breast wall has been idealised as two-dimensional flow through a sharp edged orifice in a large tank. The following guidelines for determining the parameters mentioned above may be used for preparing preliminary designs and studies on hydraulic model may be conducted for confirming or improving on the preliminary design. Figure 36 shows pertinent details of various profiles of the spillway with a breast wall.

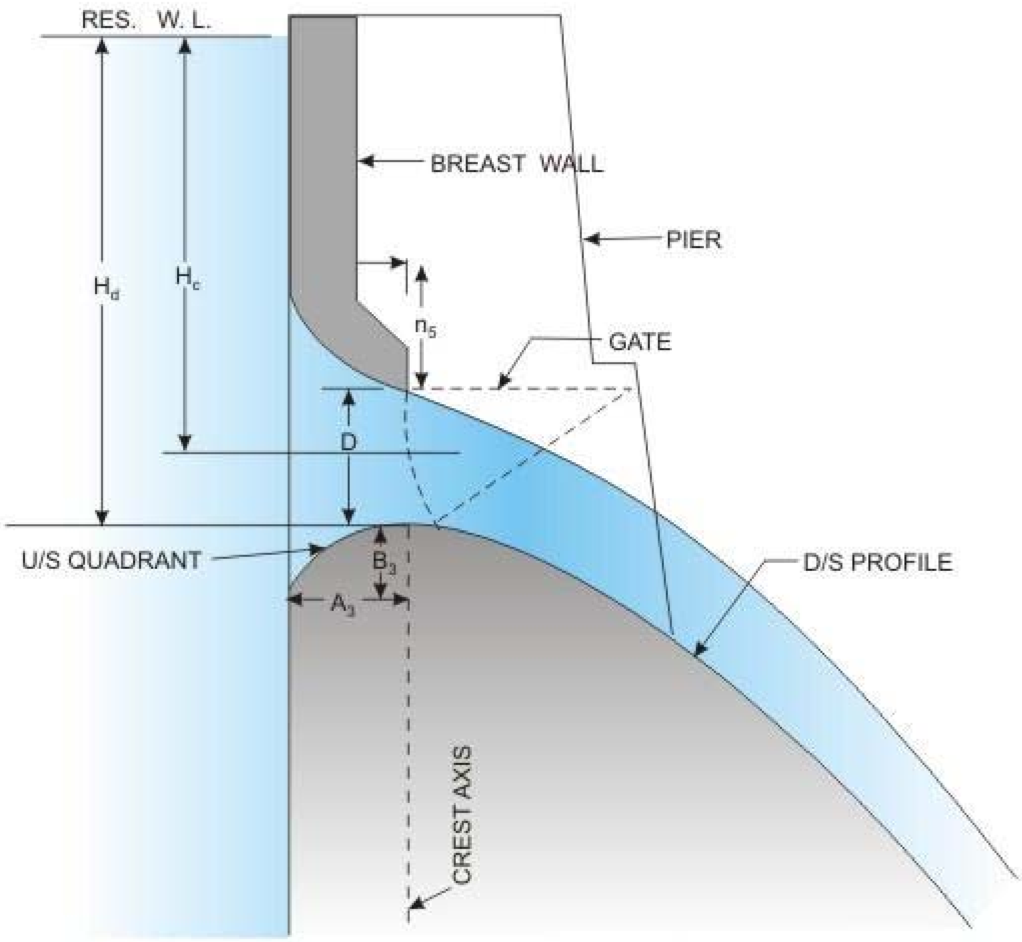


FIGURE 36. Spillway with breast wall

Ogee Profile - Upstream Quadrant'

The upstream quadrant may conform to an ellipse with the equation:

$$\frac{X_3^2}{A_3^2} + \frac{Y_3^2}{B_3^2} = 1 \tag{5}$$

where

$A_3 = 0.541\, D\, (H_d/D)^{0.32}$ and
$B_3 = 0.3693\, D\, (H_d/D)^{0.04}$

Ogee Profile - Downstream profile

The downstream profile may conform to equation:

$$X_4^{n_4} = K_4 \cdot H_d^{n_4 - 1} \cdot Y_4 \tag{6}$$

where

$$K_4 = 0.04 - 0.025 \frac{H_d}{D} \tag{7}$$

and

$$n_4 = 1.782 - 0.0099 \left(\frac{H_d}{D} - 1 \right) \tag{8}$$

Bottom Profile of the Breast Wall

The bottom profile of the breast wall may conform to the equation:

$$X_5 = \frac{K_5}{n_5^{2.4}} \cdot Y_5^{2.4} \tag{9}$$

where

$$K_5 = 0.541 D \left(\frac{H_d}{D} \right)^{0.32} \tag{10}$$

$$n_5 = 0.4 D \tag{11}$$

The upstream edge of the breast wall is in line with the upstream edge of the spillway and the downstream edge is in line with the spillway crest axis, as shown in Figure 36.

The details of the upstream curve of the crest and bottom profile of breast wall are shown in Figure 37.

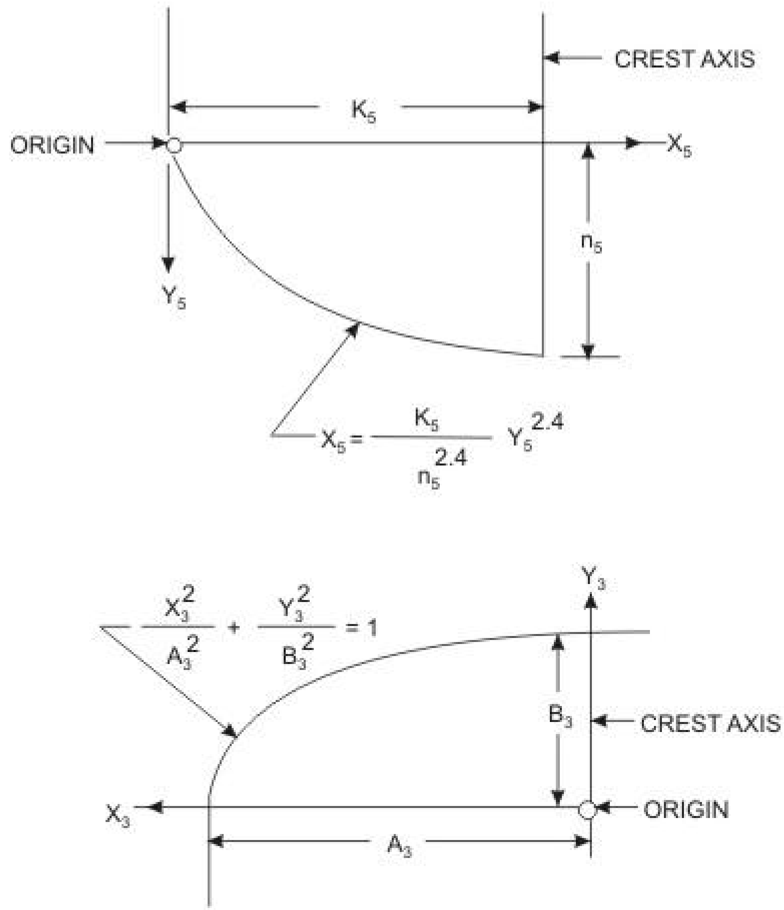


FIGURE 37. Upstream profile of ogee crest and bottom profile of breast wall details

Discharge Computation

The discharge through the breast wall spillway may be estimated by the equation:

$$Q = C_b \cdot L \cdot D\left\{2g\left(\mathrm{H}_c + \frac{V_a^2}{2g}\right)\right\}^{0.5} \qquad (12)$$

The following equation relates C_h with the parameter (H/H_d) in the range of $H/H_d = 0.8$ to 1.33.

$$C_b = 0.148631 + 0.945305(H/H_d) - 0.326238(H/H_d)^2 \qquad (13)$$

Typical values of C_b are:

H/H_d	C_b

0.80	0.696
1.00	0.769
1.15	0.797
1.33	0.829

4.8.11 Selection of spillways

The Bureau of Indian Standards code IS: 10137-1982 "Guidelines for selection of spillways and energy dissipators" provide guidelines in choosing the appropriate type of spillway for the specific purpose of the project. The general considerations that provide the basic guidelines are as follows:

Safety Considerations Consistent with Economy

Spillway structures add substantially to the cost of a dam. In selecting a type of spillway for a dam, economy in cost should not be the only criterion. The cost of spillway must be weighed in the light of safety required below the dam.

Hydrological and Site Conditions

The type of spillway to be chosen shall depend on:
a) Inflow flood;
b) Availability of tail channel, its capacity and flow hydraulics;
c) Power house, tail race and other structures downstream; and
d) Topography

Type of Dam

This is one of the main factors in deciding the type of spillway. For earth and rockfill dams, chute and ogee spillways are commonly provided, whereas for an arch dam a free fall or morning glory or chute or tunnel spillway is more appropriate. Gravity dams are mostly provided with ogee spillways.

Purpose of Dam and Operating Conditions

The purpose of the dam mainly determines whether the dam is to be provided with a gated spillway or a non-gated one. A diversion dam can have a fixed level crest, that is, non-gated crest.

Conditions Downstream of a Dam

The rise in the downstream level in heavy floods and its consequences need careful consideration. Certain spillways alter greatly the shape of the hydrograph downstream of a dam. The discharges from a siphon spillway may have surges and break-ups as priming and depriming occurs. This gives rise to the wave travelling downstream in the river, which may be detrimental to navigation and fishing and may also cause damage to population and developed areas downstream.

Nature and Amount of Solid Materials Brought by the River

Trees, floating debris, sediment in suspension, etc, affect the type of spillway to be provided. A siphon spillway cannot be successful if the inflow brings too much of floating materials. Where big trees come as floating materials, the chute or ogee spillway remains the common choice.

Apart from the above, each spillway can be shown as having certain specific advantages under particular site conditions. These are listed below which might be helpful to decide which spillway to choose for a particular project.

Ogee Spillway

It is most commonly used with gravity dams. However, it is also used with earth and rockfill dams with a separate gravity structure; the ogee crest can be used as control in almost all types of spillways; and it has got the advantage over other spillways for its high discharging efficiency.

Chute Spillway

a) It can be provided on any type of foundation,
b) It is commonly used with the earth and rockfill dams, and
c) It becomes economical if earth received from spillway excavation is used in dam construction.

The following factors limit its adaption:
a) It should normally be avoided on embankments;
b) Availability of space is essential for keeping the spillway basins away from the dam paving; and
c) If it is necessary to provide too many bends in the chute because of the topography, its hydraulic performance can be adversely affected.

Side Channel Spillways

This type of spillway is preferred where a long overflow crest is desired in order to limit the intensity of discharge, It is useful where the abutments are steep, and it is useful where the control is desired by the narrow side channel.

The factor limiting its adoption is that this type of spillway is hydraulically less efficient.

Shaft Spillways (Morning Glory Spillway)

a) This can be adopted very advantageously in dam sites in narrow canyons, and
b) Minimum discharging capacity is attained at relatively low heads. This characteristic makes the spillway ideal where the maximum spillway outflow is to be limited. This characteristic becomes undesirable where a discharge more than the design capacity is

to be passed. So, it can be used as a service spillway in conjunction with an emergency spillway.

The factor limiting its adoption is the difficulty of air-entrainment in a shaft, which may escape in bursts causing an undesirable surging.

Siphon Spillway

Siphon spillways can be used to discharge full capacity discharges, at relatively low heads, and great advantage of this type of spillway is its positive and automatic operation without mechanical devices and moving parts.

The following factors limit the adoption of a siphon spillway:

It is difficult to handle flows materially greater than designed capacity, even if the reservoir head exceeds the design level; Siphon spillways cannot pass debris, ice, etc; There is possibility of clogging of the siphon passage way and breaking of siphon vents with logs and debris; In cold climates, there can be freezing inside the inlet and air vents of the siphon; When sudden surges occur and outflow stops; The structure is subject to heavy vibrations during its operation needing strong foundations; and Siphons cannot be normally used for vacuum heads higher than
8 m and there is danger of cavitation damage.

Overfall or Free Fall Spillway

This is suitable for arch dams or dams with downstream vertical faces; and this is suitable for small drops and for passing any occasional flood.

Tunnel or Conduit Spillway

This type is generally suitable for dams in narrow valleys, where overflow spillways cannot be located without risk and good sites are not available for a saddle spillway. In such cases, diversion tunnels used for construction can be modified to work as tunnel spillways. In case of embankment dams, diversion tunnels used during construction may usefully be adopted. Where there is danger to open channels from snow or rock slides, tunnel spillways are useful.

4.8.12 Energy dissipators

Different types of energy dissipators may be used along with a spillway, alone or in combination of more than one, depending upon the energy to be dissipated and erosion control required downstream of a dam. Broadly, the energy dissipators are classified under two categories – Stilling basins or Bucket Type. Each of these are further sub-categorized as given below.

Stilling basin type energy dissipators

They may fundamentally be divided into two types.

a) Hydraulic jump type stilling basins

1. Horizontal apron type (Figure 38)

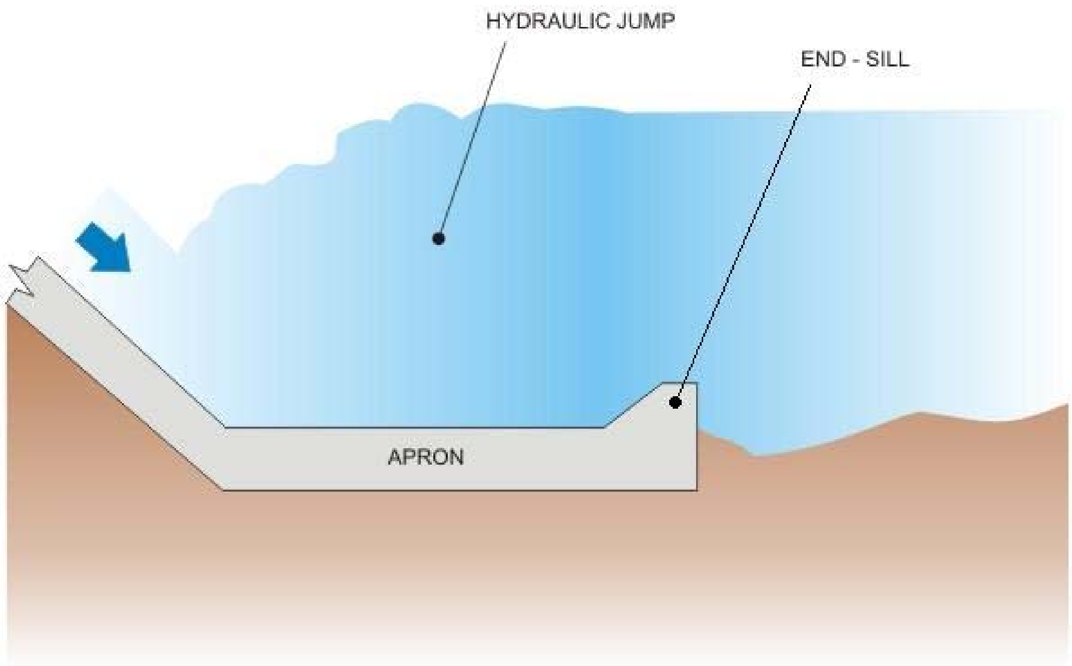


FIGURE 38. Horizontal apron stilling basin with end-sill

2. Sloping apron type (Figure 39)

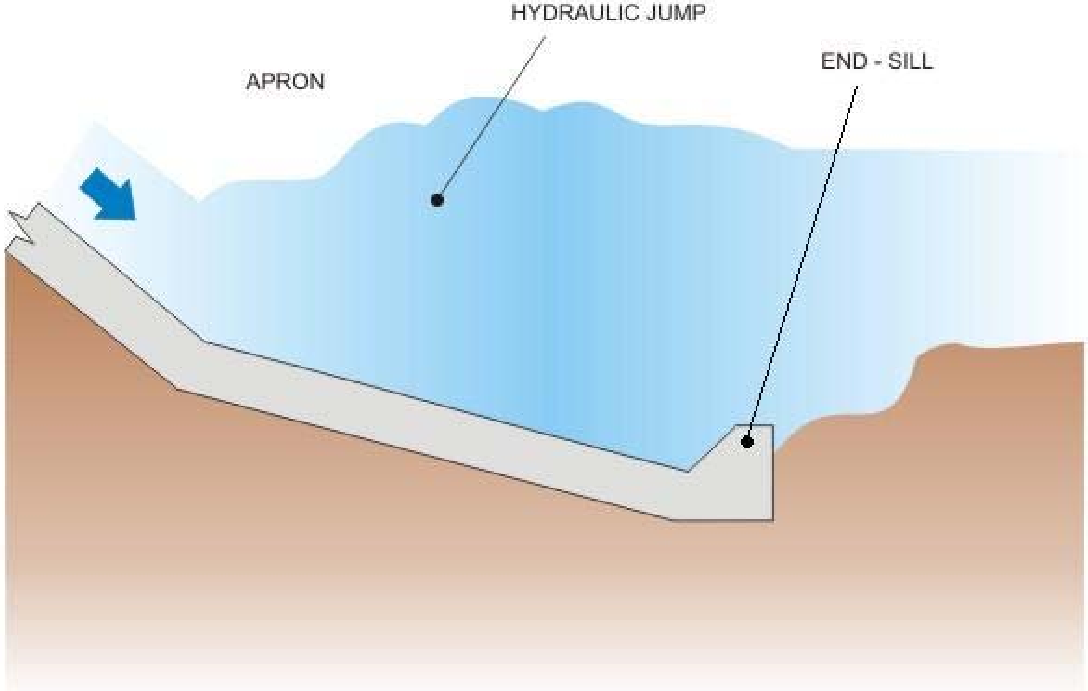


FIGURE 39. Sloping apron stilling basin with end-sill

b) Jet diffusion type stilling basins

   1. Jet diffusion stilling basins (Figure 40)

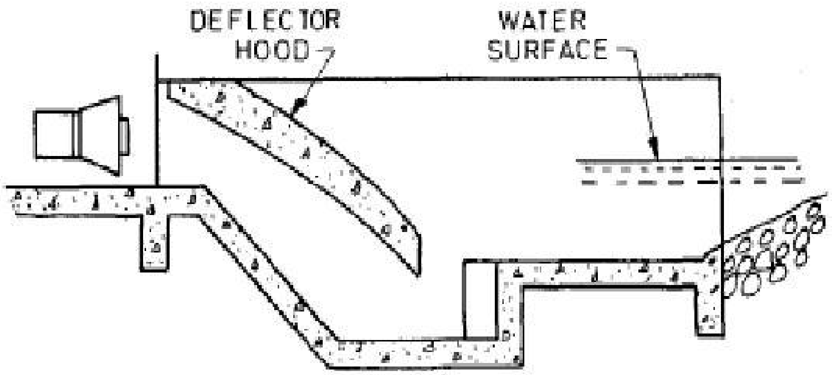
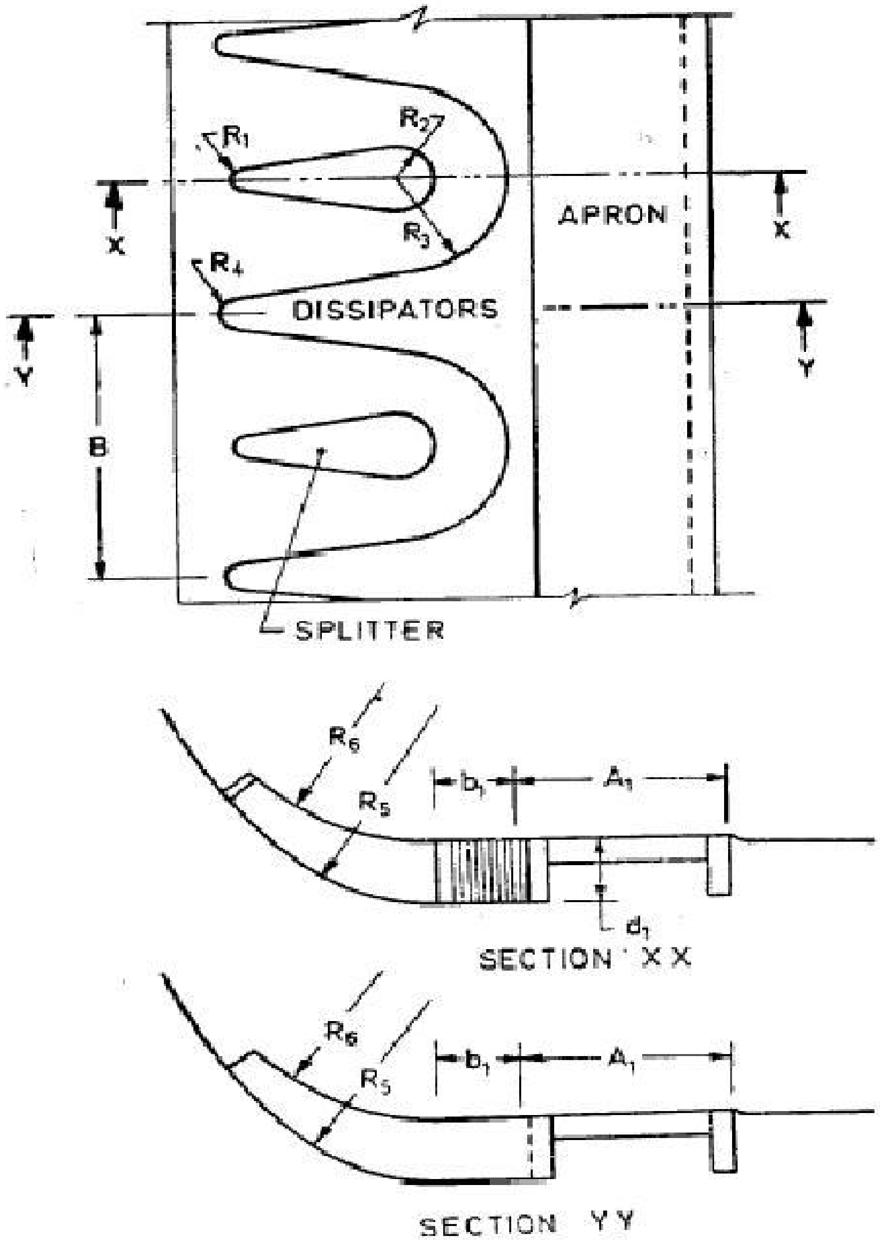


FIGURE 40. Jet diffusion stilling basin

(Image courtesy: IS 10137)

2. Interacting jet dissipators (Figure 41)



$R_1 = 0{\cdot}03B$
$R_2 = 0{\cdot}13B$
$R_3 = 0{\cdot}37B$
$R_4 = 0{\cdot}03B$
$R_5 = 0{\cdot}66B$

$R_6 = 0{\cdot}66B$
$d_1 = 0{\cdot}18B$
$A_1 = 0{\cdot}71B$
$b_1 = 0{\cdot}25B$

where B is the bay width

FIGURE 41. Interacting jet dissipators

(Image courtesy: IS 10137)

3. Free jet stilling basins (Figure 42)

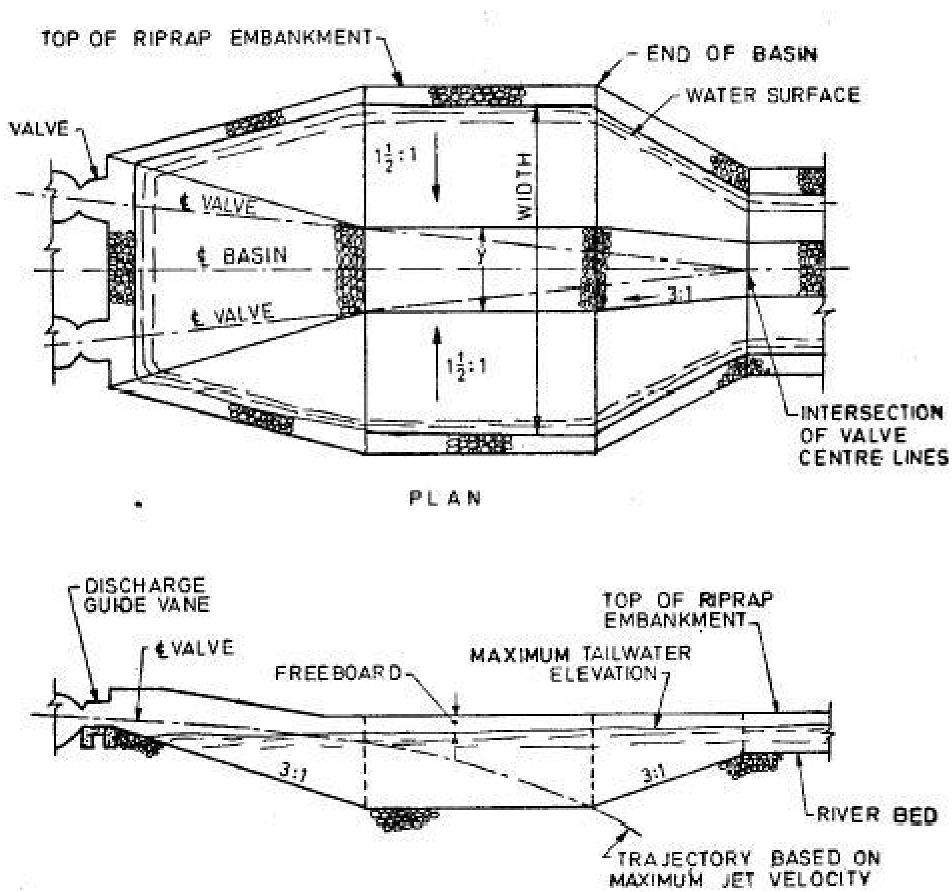


FIGURE 42. Free jet stilling basin

(Image courtesy: IS 10137)

4. Hump stilling basins (Figure 43)

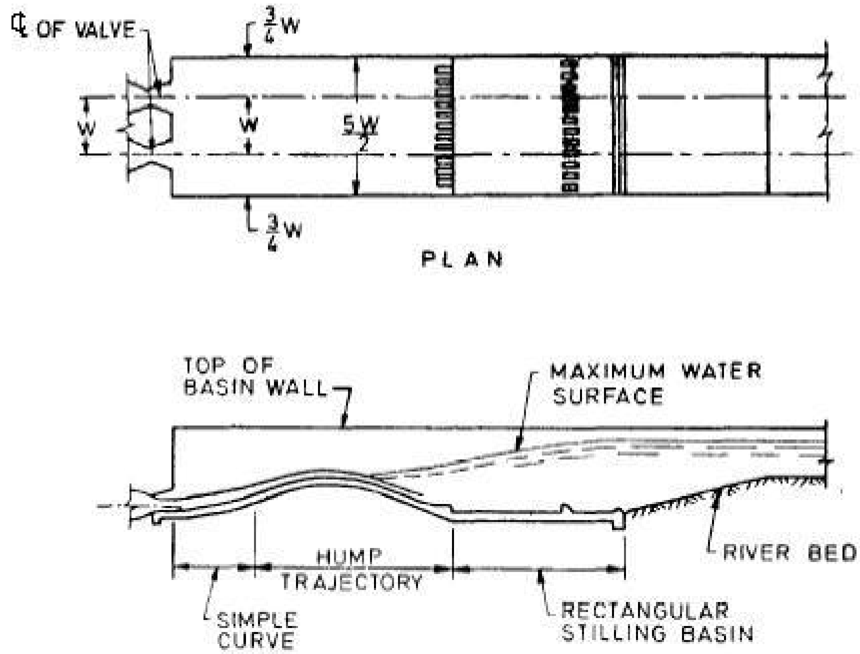


FIGURE 43. Hump stilling basin

(Image courtesy: IS 10137)

5. Impact stilling basins (Figure 44)

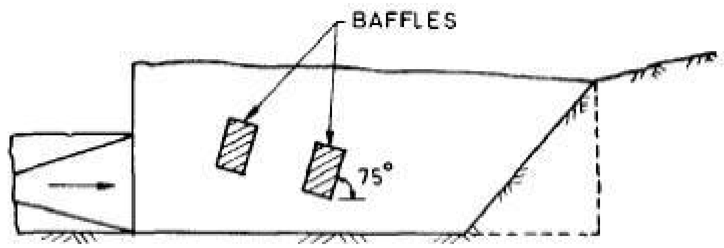


Fig. 15 Impact Stilling Basin with Inclined Baffles

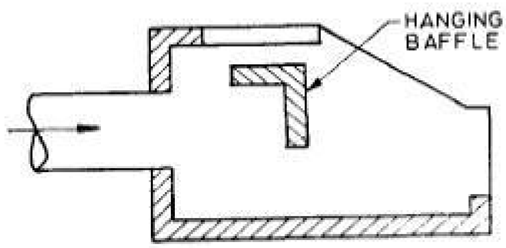


FIGURE 44. Impact stilling basin

(Image courtesy: IS 10137)

Bucket type energy dissipators

This type of energy dissipators includes the following:

1. Solid roller bucket
2. Slotted roller bucket
3. Ski jump (Flip/Trajectory) bucket

The shapes of the different types of bucket-type stilling basins have been given in section 4.8.14. Usually the hydraulic jump type stilling basins and the three types of bucket-type energy dissipators are commonly used in conjunction with spillways of major projects. The detailed designs of these are dealt in subsequent sections.

Since energy dissipators are an integral part of a dam's spillway section, they have to be viewed in conjuction with the latter. Two typical examples have been shown in Figures 45 and 46, though it must be remembered that any type of energy dissipator may go with any type of spillway, depending on the specific site conditions.

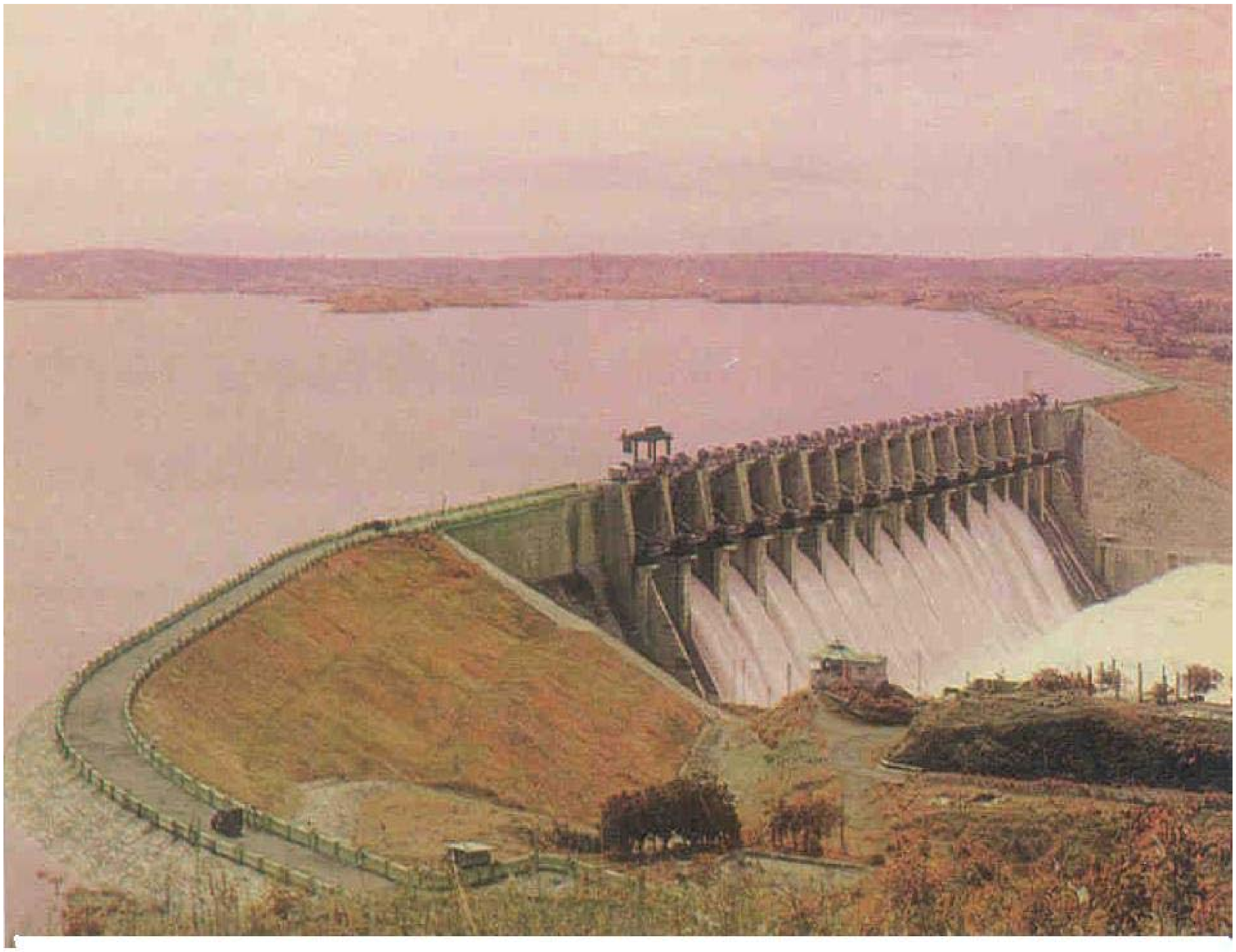


FIGURE 45. Mahi Bajaj Sagar Dam across river Mahi in Rajasthan showing ski-jump bucket energy dissipators in action

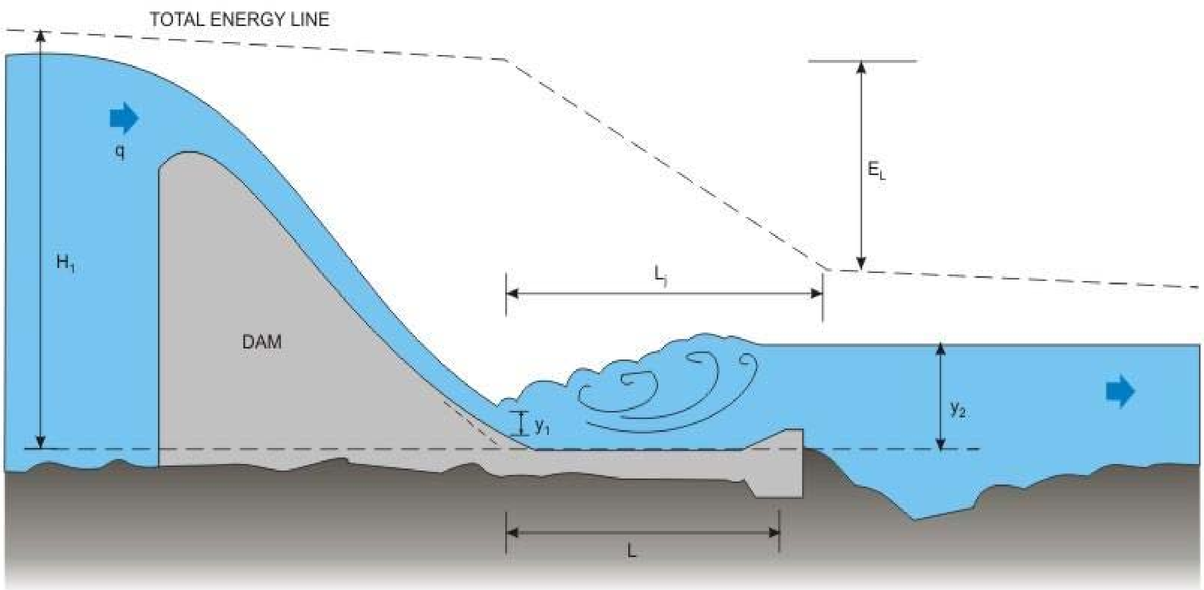
(Image courtesy: Web-site of Ministry of Water Resources, Government of India)

FIGURE 46. Salal project on river Chenab showing energy being dissipated by ski-jump bucket type energy dissipators

(Image courtesy: Web-site of Ministry of Water Resources, Government of India)

4.8.13 Design of Hydraulic Jump Stilling Basin type energy dissipators

A hydraulic jump is the sudden turbulent transition of supercritical flow to subcritical. This phenomena, which involves a loss of energy, is utilized at the bottom of a spillway as an energy dissipator by providing a floor for the hydraulic jump to take place (Figure 47). The amount of energy dissipated in a jump increases with the rise in Froude number of the supercritical flow.



L : LENGTH OF STILLING BASIN APRON

L_j : LENGTH OF HYDRAULIC JUMP

q : DISCHARGE PER UNIT WIDTH

H_1 : TOTAL ENERGY UPSTREAM OF JUMP

y_1 : PRE-JUMP (SUPER CRITICAL FLOW) DEPTH

y_2 : POST-JUMP (SUB CRITICAL FLOW) DEPTH

E_L : ENERGY LOST IN JUMP

FIGURE 47. Definition sketch of hydraulic jump & associated parameters

The two depths, one before (**y_1**) and one after (**y_2**) the jump are related by the following expression:

$$\frac{y_1}{y_2} = \frac{1}{2}\left(-1 + \sqrt{1 + 8F_1^2}\right) \tag{14}$$

Where F_1 is the incoming Froude number = $\dfrac{V_1}{\sqrt{gy_1}}$

Alternatively, the expression may be written in terms of the outgoing Froude number F_2 $\left(= \dfrac{V_2}{\sqrt{gy_2}}\right)$ as

$$\frac{y_2}{y_1} = \frac{1}{2}\left(-1 + \sqrt{1 + 8F_2^2}\right) \tag{15}$$

where **V_1** and **V_2** are the incoming and outgoing velocities and ***g*** is the acceleration due to gravity.

The energy lost in the hydraulic jump (E_L) is given as:

$$E_L = \frac{(y_2 - y_1)^3}{4y_1 y_2} \qquad (16)$$

In most cases, it is possible to find out the pre-jump depth (y_1) and velocity (V_1) from the given value of discharge per unit width (**q**) through the spillway. This is done by assuming the total energy is nearly constant right from the spillway entrance up to the beginning of the jump formation, as shown in Figure 47. V_1 may be assumed to be equal to $\sqrt{2gH_1}$ **,** where H_1 is the total energy upstream of the spillway, and neglecting friction losses in the spillway. The appropriate expressions may be solved to find out the post-jump depth (y_2) and velocity (V_2).

The length of the jump (L_j) is an important parameter affecting the size of a stilling basin in which the jump is used. There have been many definitions of the length of the jump, but it is usual to take the length to be the horizontal distance between the toe of the jump upto a section where the water surface becomes quite level after reaching a maximum level. Because the water surface profile is very flat towards the end of the jump, large personal errors are introduced in the determination of the jump length.

Bradley and Peterka (1975) have experimentally found the length of hydraulic jumps and plotted them in terms of the incoming Froude number (F_1), and post-jump depth (y_2) as shown in Figure 48. It is evident that while L_j/y_2 varies most for small values of F_1, at higher values, say above 5 or so, L_j/y_2 is practically constant at a value of about 6.1.

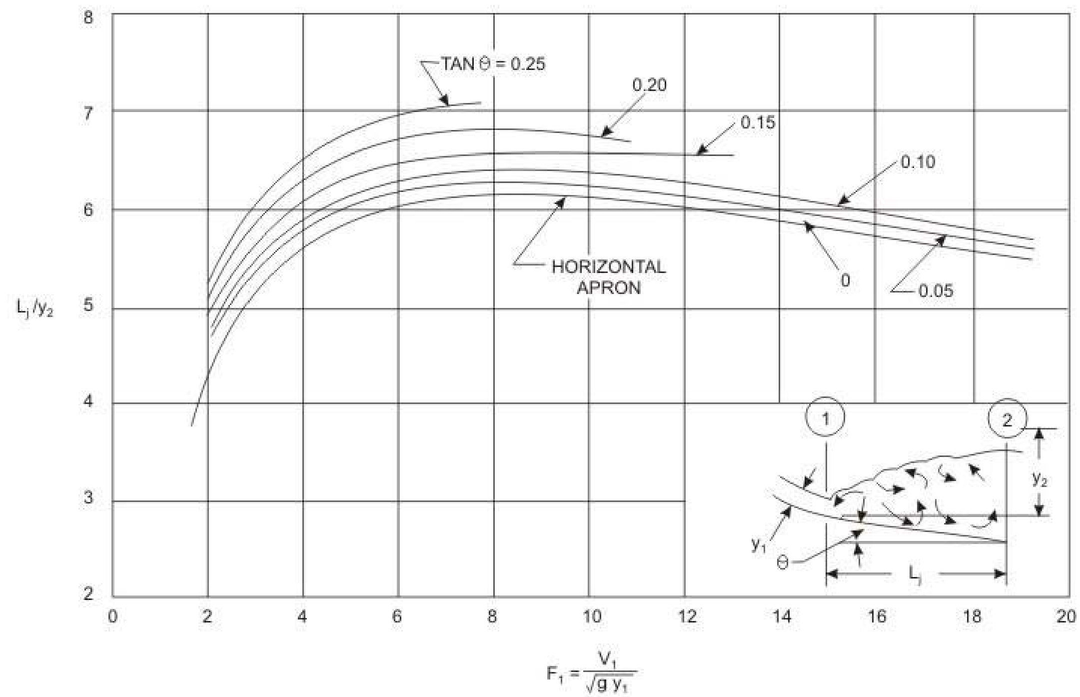


FIGURE 48. Length of hydraulic jump on a horizontal or inclined floor

The depth of water in the actual river downstream of the stilling basin (**y_2^***) is determined from the river flow observations that have been plotted as a stage-discharge curve (Figure 49).

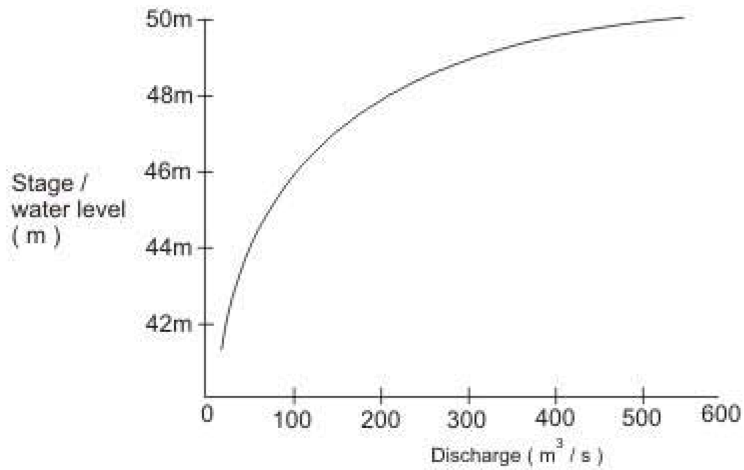


FIGURE 49. A typical stage-discharge curve for a river

Subtracting the stilling basing apron level from the stage or water level corresponding to the total discharge passing through the spillway gives the tail-water depth ($\mathbf{y_2}^*$). Since the stage-discharge curve gives indications about the tail-water of the spillway, it is called the Tail-Water Rating Curve (TRC), usually expressed as the water depth ($\mathbf{y_2}^*$) versus unit discharge (**q**), as shown in Figure 50(a).

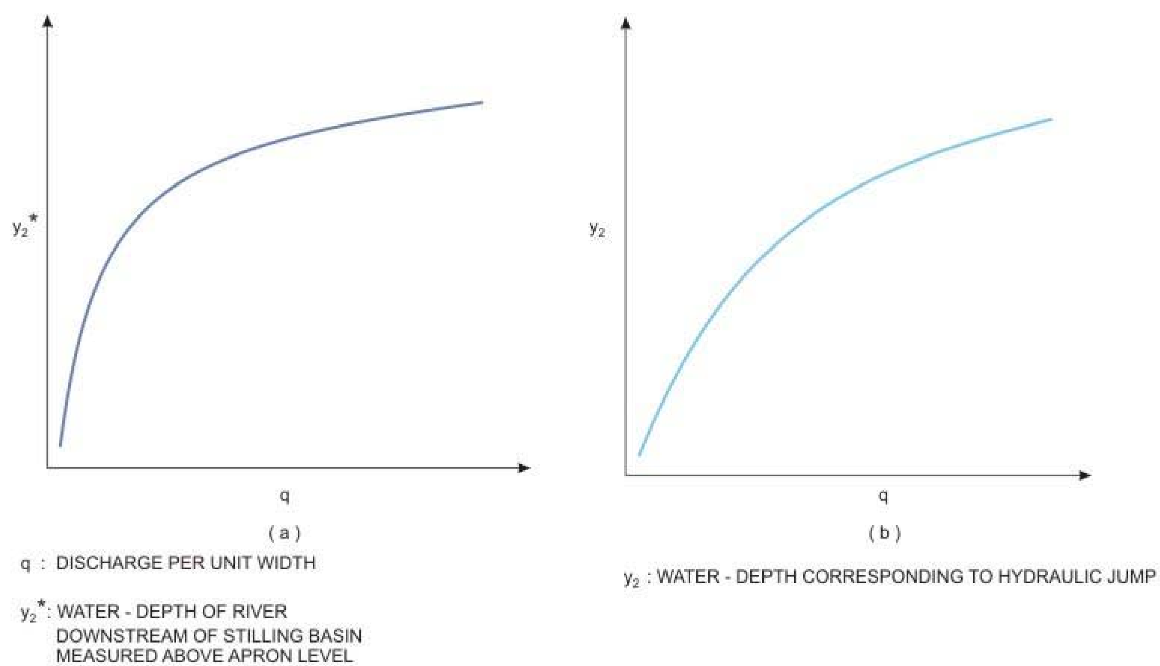


FIGURE 50. Water Level curves
(a) : Tail - water Rating Curve (TRC)
(b) : Jump Rating Curve (JRC)

At the same time, using the formula relating unit discharge (**q**) with the post-jump depth (**y_2**), a similar graph may be obtained, as shown in Figure 50(b). Since this graph gives indication about the variation of the post-jump depth, it is called the Jump Rating Curve (JRC).

In general, the JRC and TRC would rarely coincide, if plotted on the same graph, as shown in Figure 51.

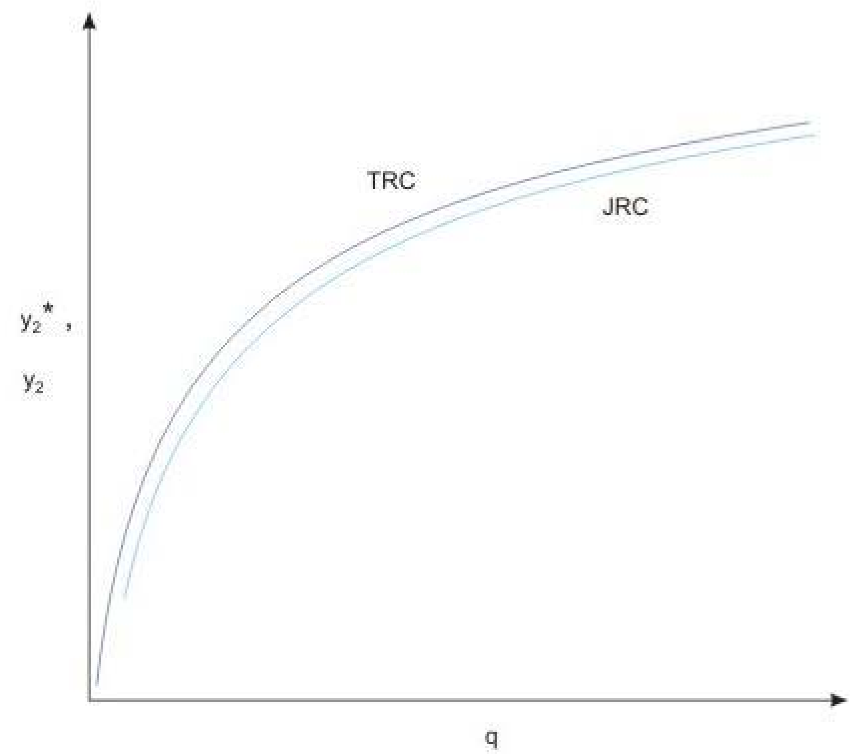


FIGURE 51. TRC & JRC coinciding

At times, the TRC may lie completely below the JRC (Figure 52), for all discharges, in which case the jump will be located away from the toe of the spillway resulting in possible erosion of the riverbed.

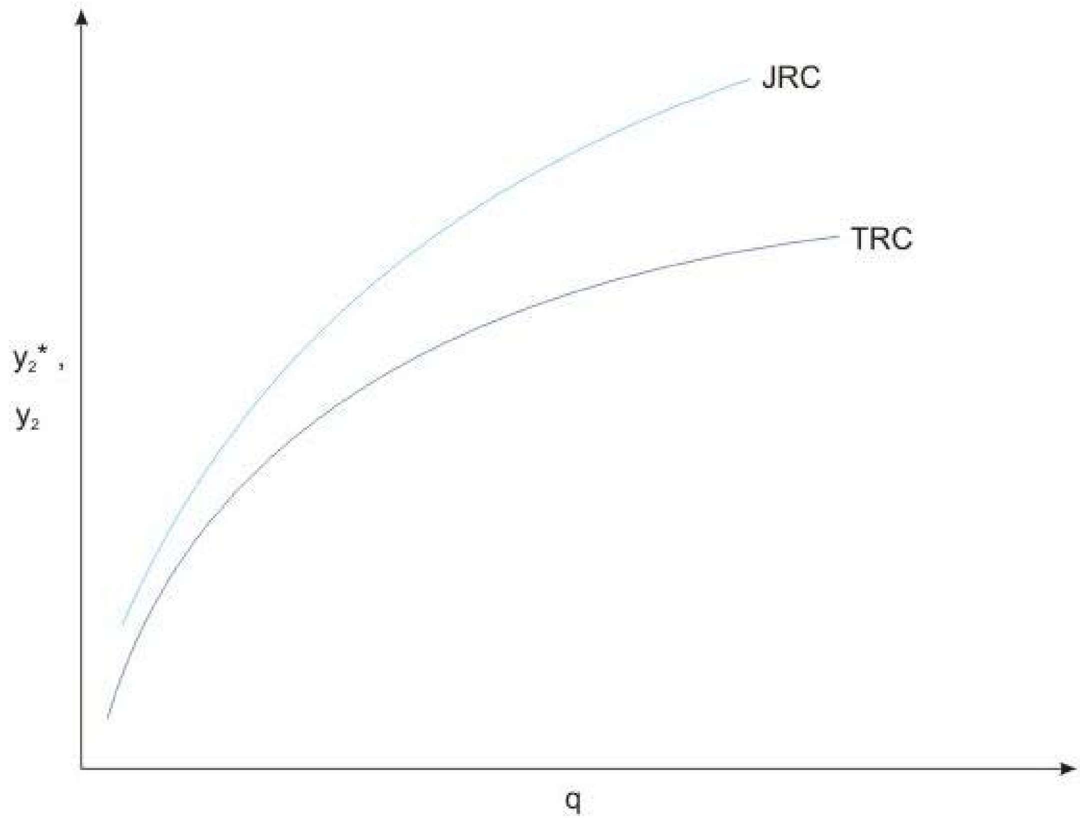


FIGURE 52. TRC below JRC for all discharges

If the TRC is completely above the jump would be located so close to the spillway to make it submerged which may not dissipate the energy completely. (Figure 53)

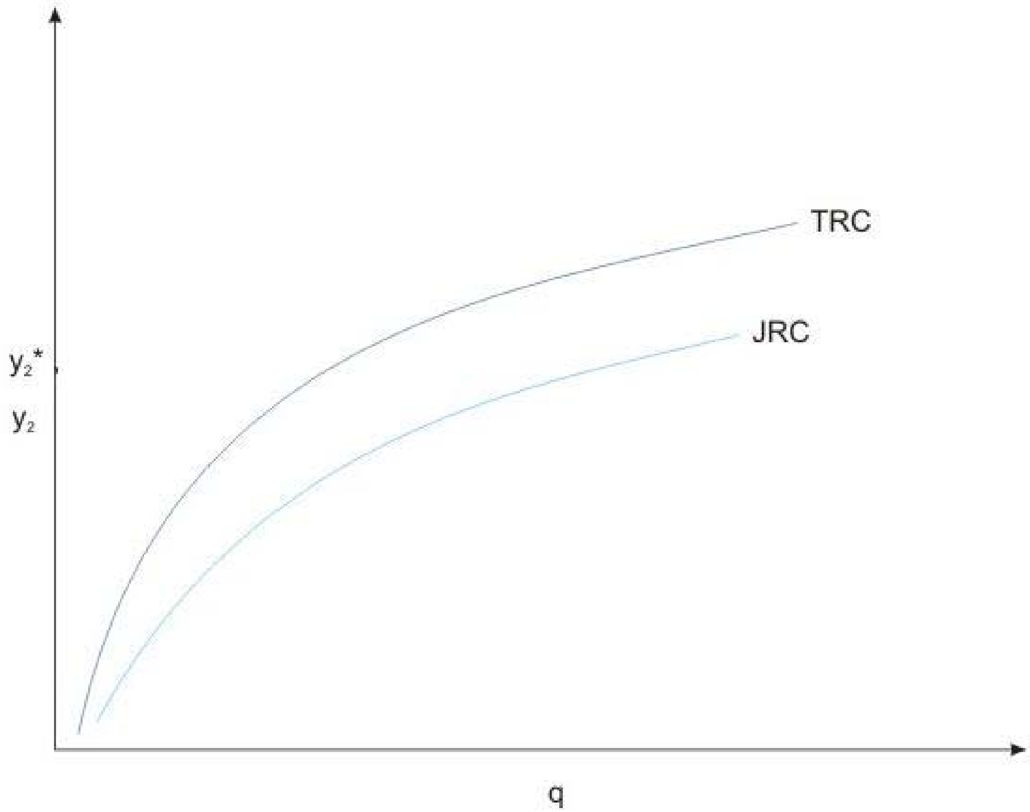


FIGURE 53. TRC above JRC for all discharges

It may also be possible in actual situations that the TRC may be below the JRC for some discharges above for the rest, as shown in Figs. 54 and 55.

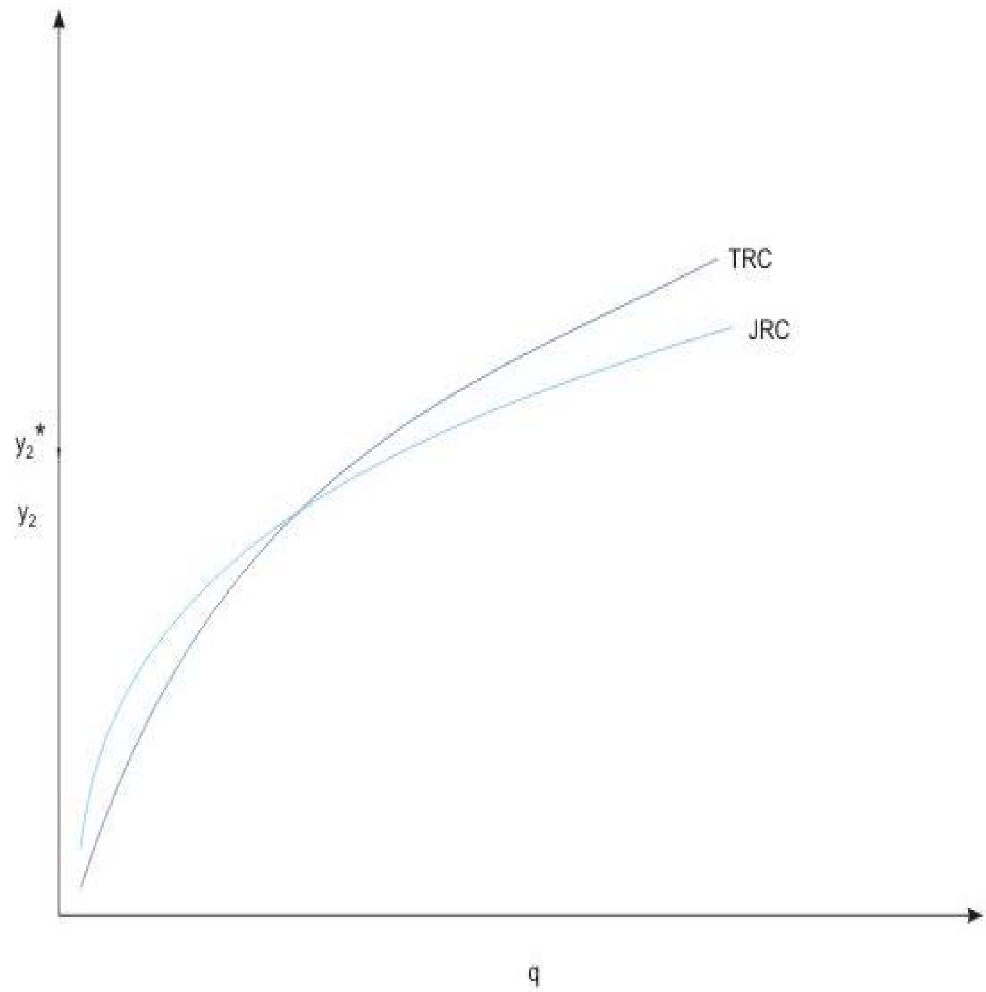


FIGURE 54. TRC below JRC for low discharges and above for high discharges

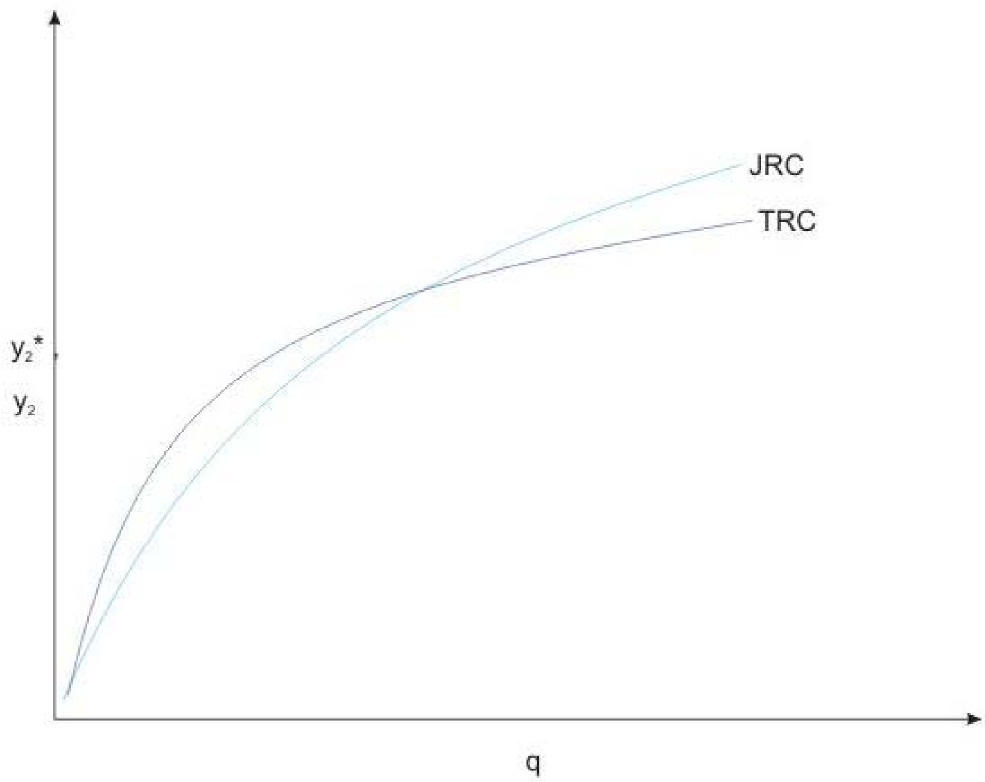


FIGURE 55. TRC above JRC for low discharges and below for high discharges

In these cases two, favourable location of jump may not be possible. In view of the above situations, the following recommendations have been made for satisfactory performance of the hydraulic jumps.

Case1 (Figure 51)
This is the ideal case in which the horizontal apron provided on the riverbed downstream from the toe of the spillway would suffice. The length of the apron should be equal to the length of the jump corresponding to the maximum discharge over the spillway.

Case2 (Figure 52)
It is apparent that the tail water depth as provided by the natural river is not sufficiently for the jump to form. This may be over come by providing a stilling basin apron that is depressed below the average riverbed level (Figure56) or by providing a sill or baffle of sufficient height at the end of the spillway (Figure 57)

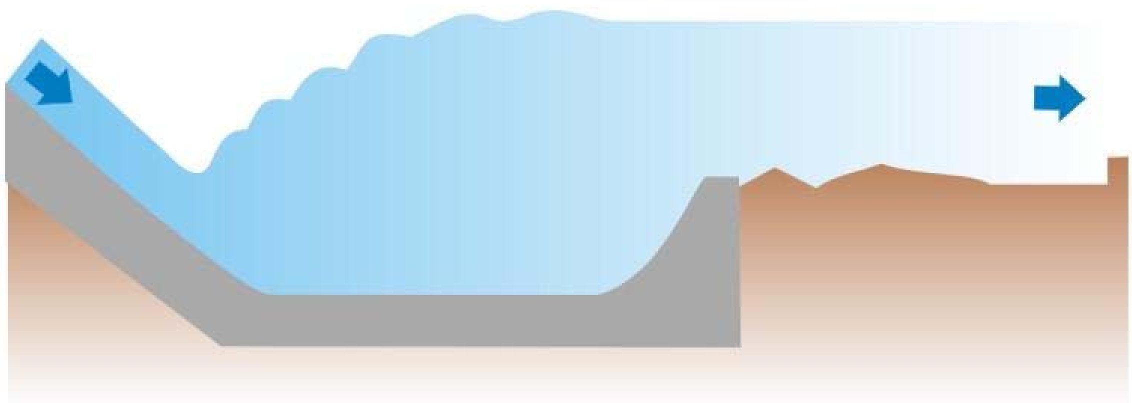


FIGURE 56. Depressed floor of stilling basin apron

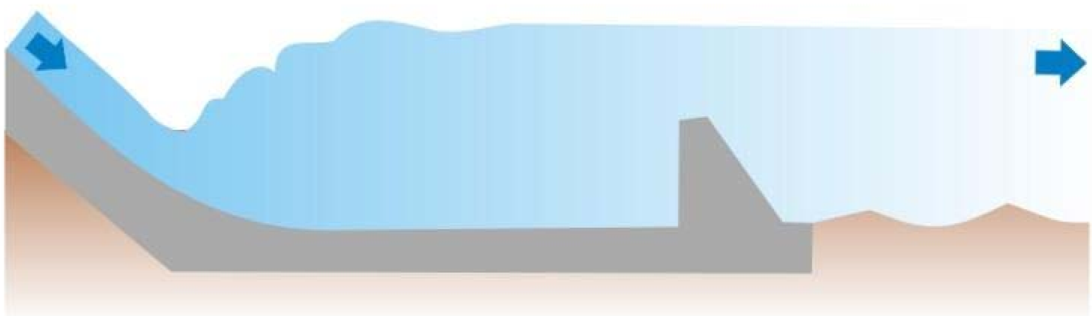


FIGURE 57. High end - sill or baffle at toe of stilling basin

Case 3 (Figure 53)
Since this situation results in submergence results in submergence of the jump, it is necessary to raise the floor in order to form a clear jump. In practice, it is done by providing an inclined apron of the stilling basin (Figure 58).

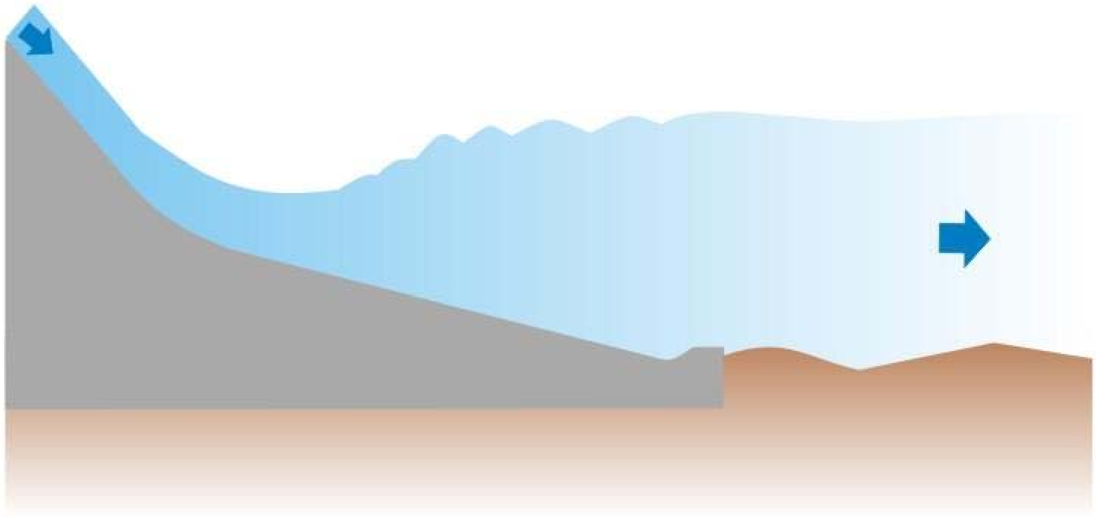


FIGURE 58. Inclined stilling basin

Case 4 (Figure 54)
This situation may be taken care of by providing an inclined floor in the upper portion of the stilling basin and providing either a depressed floor in the lower portion of the basin or provide a baffle at the end of the basin.

Case 5 (Figure 55)
In this case a sloping apron may be provided which lies partly above and partly below the riverbed. So that the jump will form on the higher slope at low discharges and on the lower slope at high discharges.

The type of Stilling Basins that may be provided under different situations is recommended by the Bureau of Indian Standards code IS: 4997-1968 “Criteria for design of hydraulic jump type stilling basins with horizontal and sloping aprons”. In all, these are four types of basin shapes recommended. Types I and II are meant for basins with horizontal floors and types III and IV for basins with inclined floors.

4.8.14 Design of bucket-type energy dissipators

Hydraulic behaviour of bucket type energy dissipator depends on dissipation of energy through:

a. Interaction of two rollers formed, one in the bucket, rolling anti-clockwise (if the flow is from the left to the right) and the other downstream of the bucket, rolling clockwise; or
b. Interaction of the jet of water, shooting out from the bucket lip, with the surrounding air and its impact on the channel bed downstream.

Bucket type energy dissipators can be either:

a) Roller bucket type energy dissipator; or
b) Trajectory bucket type energy dissipator.

The following two types of roller buckets are adopted on the basis of tailwater conditions and importance of the structure:

a) Solid roller bucket, and
b) Slotted roller bucket.

These are shown in Figure 59.

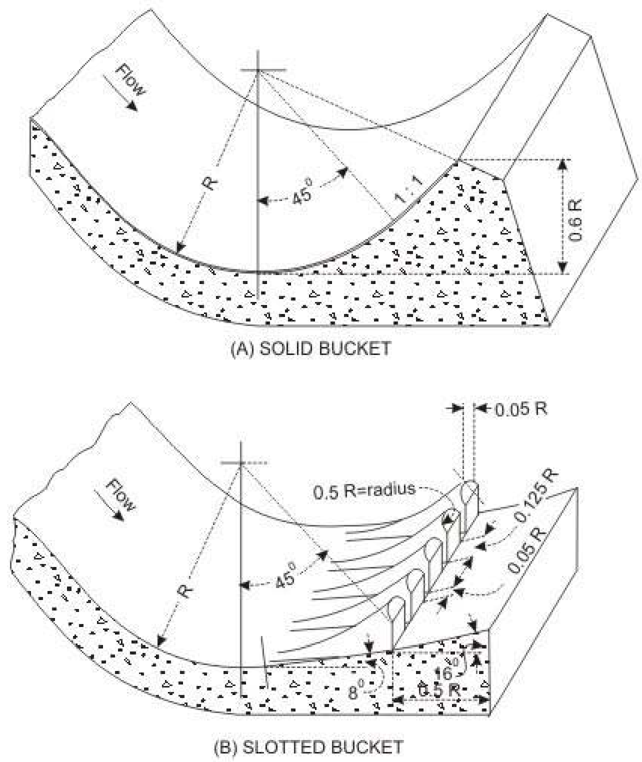


FIGURE 59. Roller buckets ; (A) Solid ; (b) Slotted

Roller bucket type energy dissipator is preferred when:

a) Tailwater depth is high (greater than 1.1 times sequent depth preferably 1.2 - times sequent depth), and
b) River bed rock is sound.

Trajectory bucket type energy dissipator is generally used when:

a) Tailwater depth is much lower than the sequent depth of hydraulic jump, thus preventing formation of the jump;
b) By locating at higher level it may be used in case of higher tailwater depths also, if economy permits; and
c) Bed of the river channel downstream is composed of sound rock.

This is shown in Figure 60.

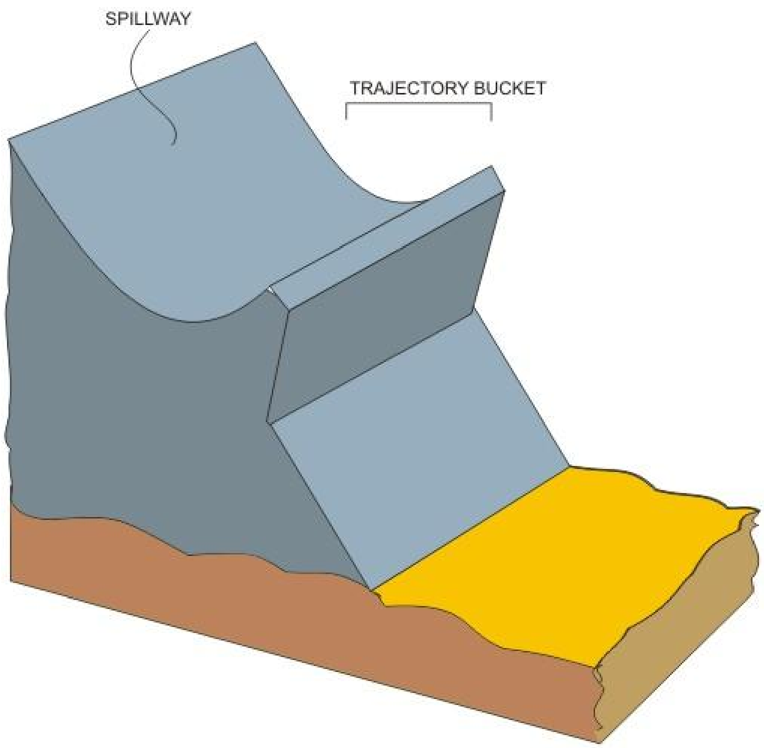


FIGURE 60. Trajectory bucket type energy dissipator

Action of the various types of bucket-type energy dissipators is given below:

Hydraulic Design of Solid Roller Bucket

An upturn solid bucket is used when the tailwater depth is much in excess of sequent depth and in which dissipation of considerable portion of energy occurs as a result of formation of two complementary elliptical rollers, one in bucket proper, called the surface roller, which is anticlockwise (if the flow is to the right) and the other downstream of the bucket, called the ground roller, which is clockwise.

In the case of solid roller bucket the ground roller is more pronounced and picks up material from downstream bend and carried it towards the bucket where it is partly deposited and partly carried away downstream by the residual jet from the lip. The deposition in roller bucket is more likely when the spillway spans are not operated equally, setting up horizontal eddies downstream of the bucket. The picked up material which is drawn into the bucket can cause abrasive damage to the bucket by churning action.

For effective energy dissipation in a solid roller bucket, both the surface or dissipating roller and the ground or stabilizing roller, should be well formed. Otherwise, hydraulic phenomenon of sweep out or heavy submergence occurs depending upon which of the rollers is inhibited.

Design Criteria - The principal features of hydraulic design of solid roller bucket consists of determining:

a) The bucket invert elevation,
b) The radius of the bucket, and
c) The slope of the bucket lip or the bucket lip angle.

The various parameters are shown in Figure 61 (a).

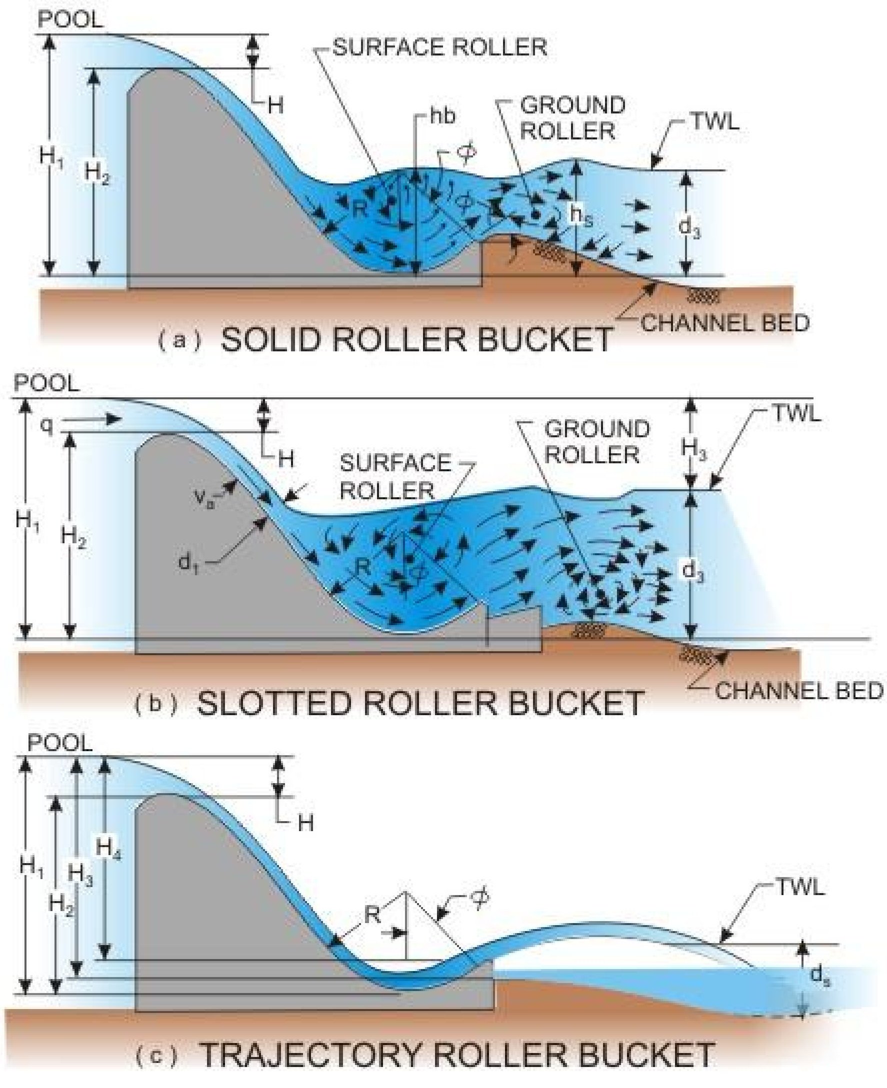


FIGURE 61. Sketches for bucket type energy dissipators

An example of the use of a solid roller bucket is the energy dissipator of the Maithan Dam Spillway (Figure 62)

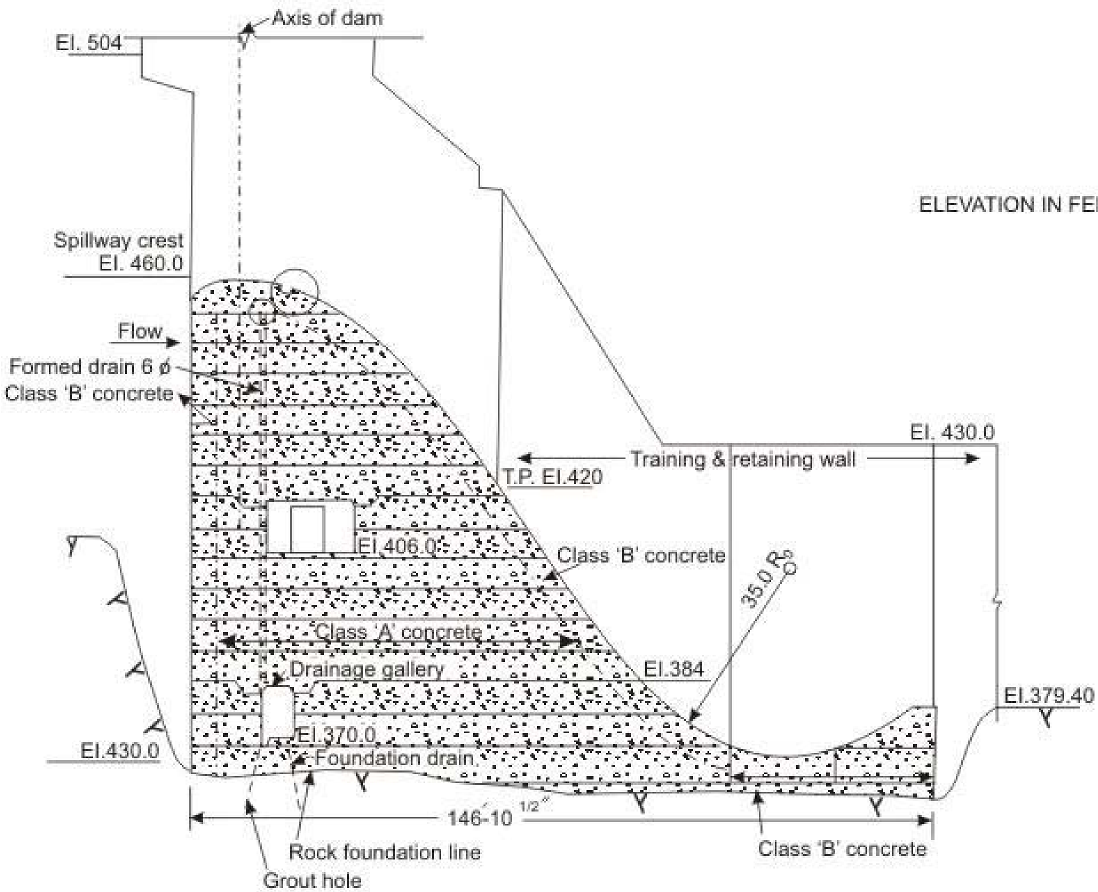


FIGURE 62. Maithon Dam spillway

Drawal of Bed Materials - A major problem with the solid roller bucket would be the damage due to churning action, caused to the bucket because of the downstream bed material brought into the bucket by the pronounced ground roller. Even in a slotted roller bucket downstream material might get drawn due to unequal operation of gates. The channel bed immediately downstream of the bucket shall be set at 1 to 1.5 m below the lip level to minimize the possibility of this condition. Where the invert of the bucket is required to be set below the channel general bed level the channel should be dressed down in one level to about 1 to 1.5 m below the lip level in about 15 m length downstream and then a recovery slope of about 3 (horizontal) to 1 (vertical) should be given to meet the general bed level as shown in Figure 62. Careful model studies should be done to check this tendency. If possible, even provision of solid apron or cement concrete blocks may be considered to avoid trapping of river bed material in the bucket as it may cause heavy erosion on the spillway face, bucket and side training wall.

In the case of slotted roller bucket a part of the flow passes through the slots, spreads laterally and is lifted away from the channel bottom by a short apron at the downstream end of the bucket. Thus the flow is dispersed and distributed over a greater area resulting in a less violent ground roller. The height of boil is also reduced in case of slotted roller bucket. The slotted bucket -provides a self-cleaning action to reduce abrasion in the bucket.

In general the slotted roller bucket is a improvement over the solid roller bucket for the range of tailwater depths under which it can operate without sweepout or diving. However, it is necessary that specific model experiments should be conducted to verify pressure on the teeth so as to avoid cavitation conditions. In case of hydraulic structures in boulder stages slotted roller buckets need not be provided. Heavy boulders rolling down the spillway face can cause heavy damage to the dents thereby making them ineffective and on the contrary, increasing the chances of damage by impact, cavitation and erosion.

Hydraulic Design of Slotted Roller Bucket

An upturned bucket with teeth in it used when the tailwater depth is much in excess of sequent depth and in which the dissipation of energy occurs by lateral spreading of jet passing through bucket slots in addition to the formation of two complementary rollers as in the solid bucket.

In the slotted roller bucket, a part of the flow passes through the slots, spreads laterally and is lifted away from the channel bottom by a short apron at the downstream end of the bucket. Thus the flow is dispersed and distributed over a greater area providing less violent flow concentrations compared to those in a solid roller bucket. The velocity distribution just downstream of the bucket is more akin to that in a natural stream, that is, higher velocities at the surface and lower velocities at the bottom. While designing a slotted roller bucket, for high head spillway exceeding the total head of 50 m or so, specific care should be taken especially for design of the teeth, to ensure that the teeth will perform cavitation free. Specific model tests should therefore be conducted to verify pressures on the teeth and the bucket invert should accordingly be fixed at such an elevation as to restrict the subatmospheric pressures to the permissible magnitude.

Design Criteria - The principal features of hydraulic design of the slotted roller bucket consists of determining in sequence:

a) bucket radius;
b) bucket invert elevation;
c) bucket lip angle; and
d) bucket and tooth dimensions, teeth spacing and dimensions and profile of short apron.

The various parameters are shown in Figure 61(b)

An example of the use of a slotted roller bucket is the energy dissipator provided in the Indira Sagar Dam Spillway (Figure 63).

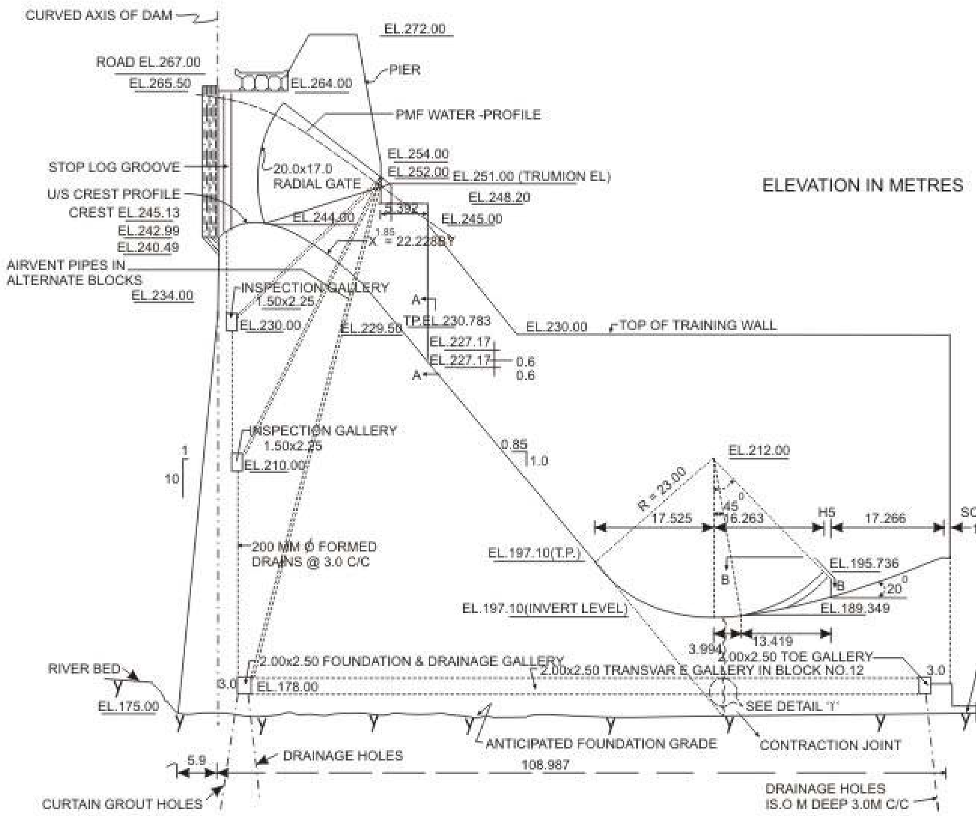


FIGURE 63. Indira Sagar Dam Spillway

Hydraulic Design of Trajectory Bucket Type Energy Dissipator

An upturn solid bucket used when the tailwater depth is insufficient for the formation of the hydraulic jump, the bed of the river channel downstream comprises sound rock and is capable of withstanding, without excessive scour, the impact of the high velocity jet. The flow coming down the spillway is thrown away from toe of the dam to a considerable distance downstream as a free discharging upturned jet which falls into the channel directly, thereby avoiding excessive scour immediately downstream of the spillway. There is hardly any energy dissipation within the bucket itself. The device is used mainly to increase the distance from the structure to the place where high velocity etc. hits the channel bed, thus avoiding the danger of excessive scour immediately downstream of the spillway. Due to the throw of the jet in the shape of a trajectory, energy dissipation takes place by

a) internal friction within the jet,
b) the interaction between the jet and surrounding air,
c) the diffusion of the jet in the tailwater, and
d) the impact on the channel bed.

When the tailwater depth is insufficient for the formation of the hydraulic jump and the bed of the channel downstream comprises sound rock which is capable of withstanding the impact of the high velocity jet, the provision of a trajectory bucket is considered more suitable as provision of conventional hydraulic jump type apron or a roller bucket involves considerable excavation in hard strata forming the bed. It is also necessary to have sufficient straight reach in the downstream of a skijump bucket. The flow coming down the spillway is thrown away in air from the toe of the structure to a considerable distance as a free discharging upturned jet which falls on the channel bed *d/s.* The hard bed can tolerate the spray from the jet and erosion by the plunging jet would not be a significant problem for the safety of the structure. Thus, although there is very little energy dissipation within the bucket itself, possible channel bed erosion close to the downstream toe of the dam is minimized. In the trajectory bucket, only part of the energy is dissipated through interaction of the jet with the surrounding air. The remaining energy is imparted to the channel bed below. The channel bed should consist of sound, hard strata and should be free from laminations, joints and weak pockets to withstand the impact of jet. The design of the trajectory bucket presupposes the formation of large craters or scour holes at the zone of impact of the jet during the initial years of operation and, therefore, the design shall be restricted to sites where generally sound rock is available in the river bed. Special care shall be taken to concrete weak pockets in the bed located in a length of

Design Criteria - The principal features of hydraulic design of tra jectory bucket consist of determining:

a) Bucket shape,
b) Bucket invert elevation, radius or principal geometrical parameters of the bucket, lip elevation and exit angle, trajectory length, and
c) Estimation of scour downstream of the spillway.

The various parameters are shown in Figure 61(c)

An example of the use of a trajectory bucket is the one provided in the Srisailam Dam Spillway (Figure 64).

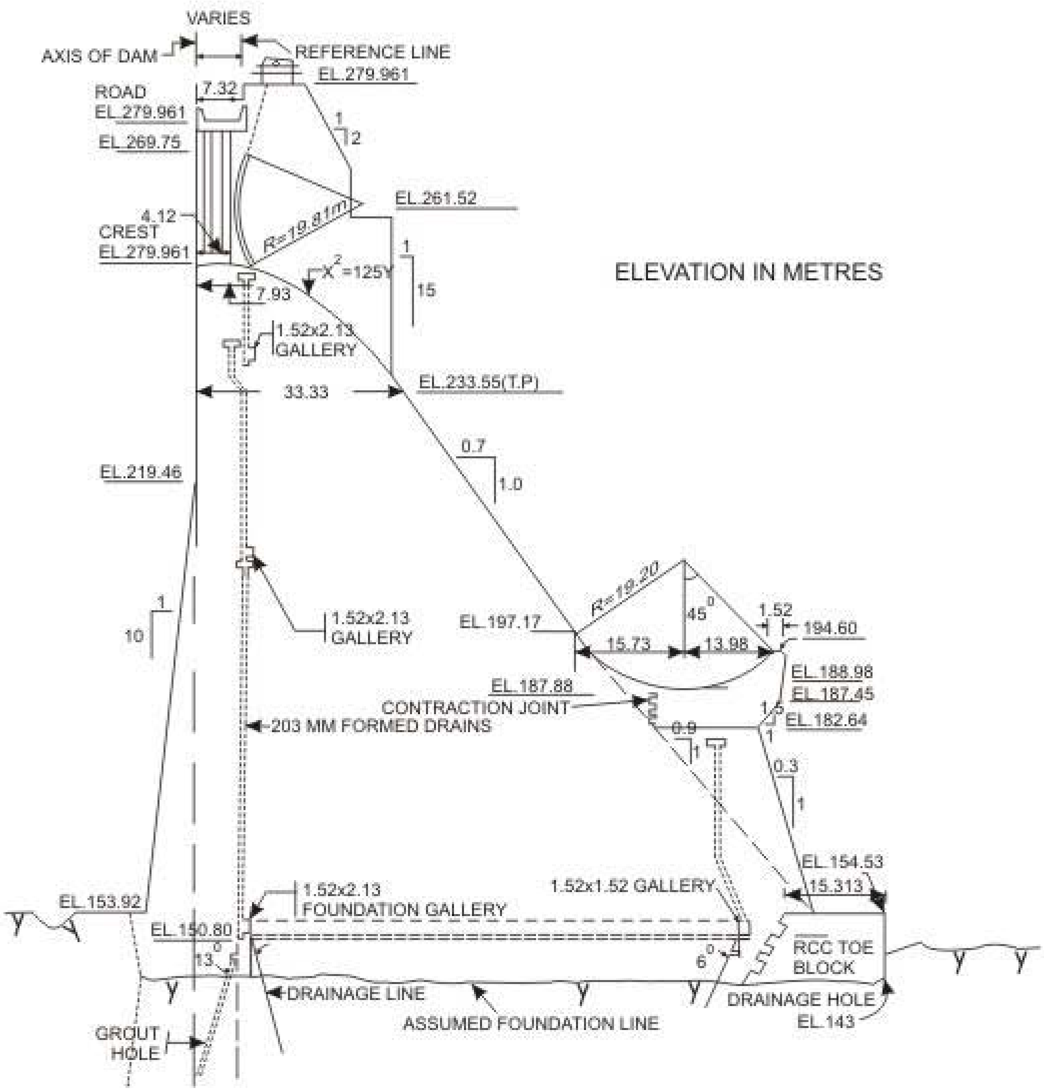


FIGURE 64. Srisailam Dam Spillway

Further details about the design of bucket type energy dissipators may be had from the Bureau of Indian Standards Code IS: 7365-1985 “Criteria for hydraulic design of bucket type energy dissipators”

4.8.15 Protection of downstream of spillways from scour

It may be noted that inspite of the provision of the best suited energy dissipator for a specific spillway under the prevailing site conditions, there may be still some energy is expected to be maximum for the trajectory type spillway, followed by the solid and slotted roller buckets and finally the hydraulic jump type stilling basins. In order to protect the downstream riverbed from these undesirable scour, the following types of protection works have been recommended by the Bureau of Indian Standards code IS:

13195-1991 “Preliminary design, operation and maintenance of protection works downstream of spillways-guidelines”.

1. *Training Walls at the Flanks of the Spillways-* Training walls extended beyond the end-sill of the stilling basins or buckets generally serve to guide the flow into the river channel, protect the wrap-rounds of the adjacent earth dams,
river banks or power house bays and tail race channels. To this extent, the training walls are -considered to be downstream protection works.

2 *Protective Aprons Downstream of Bucket Lips or End-sills of Stilling Basins-* *Protective* aprons of concrete laid on fresh rock or acceptable strata immediately downstream of bucket lips or end-sill of stilling basin, protect the energy dissipator against undermining due to excessive scour during or after construction of the spillways. A suitable concrete key is normally provided, at the downstream end of the apron. Where the normal river bed level is higher than the end-sill and a recovery slope is , provided, it sometimes becomes necessary to lay a concrete apron on such a recovery slope also for protection.

3*. Concrete Blocks or Concrete Filling on River Bed Downstream of Energy Dissipator -* Concrete blocks or concrete fillings are sometimes provided on the river bed downstream of energy dissipators to safeguard against excessive scour and prevent further scour.

4. *Protective Pitchings on Natural or Artificial Banks Downstream of Spillways-* Protective pitchings of stone rip rap, masonry or concrete blocks are provided on natural river banks or artificially constructed embankments of diversion channels, power house tail race channels or guide banks, for protecting them against high velocity flows or waves.

Figure 65 shows the various types of protection works that may typically be used downstream of a spillway.

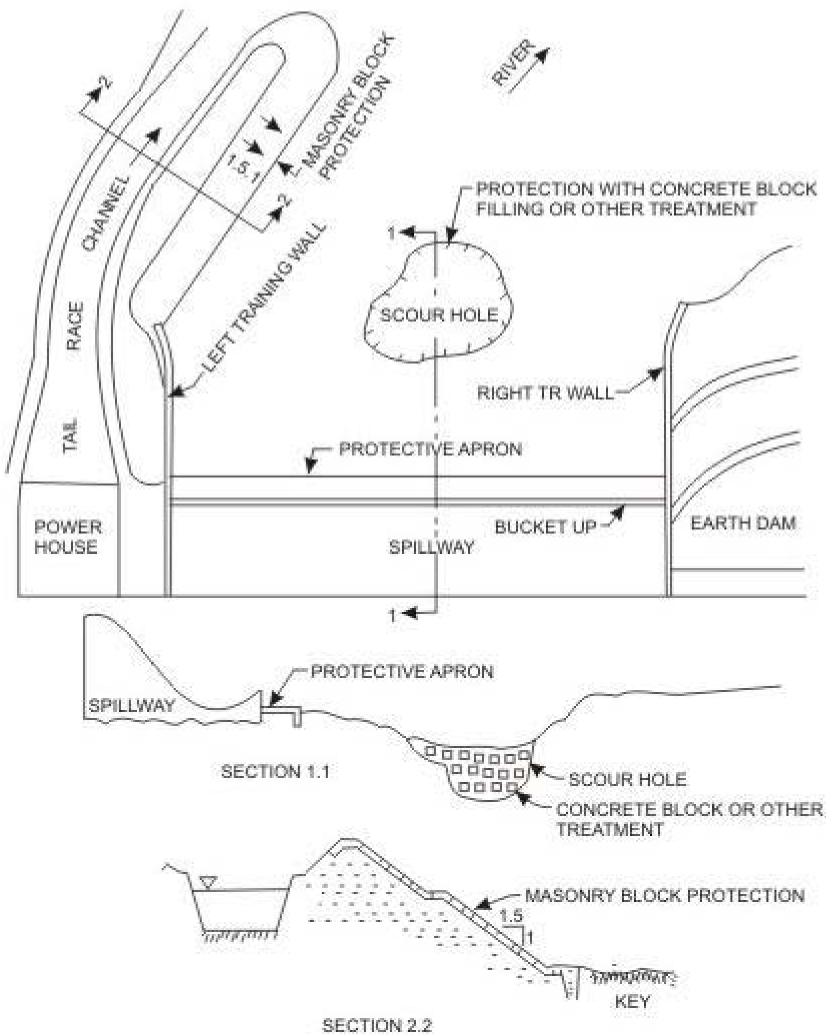


FIGURE 65. Different types of protection works downstream of a typical spillway project (Courtesy: IS 13195)

The importance of providing protection below a spillway, especially of the trajectory type may be noted from the incidence of deep scour on the downstream of the Srisailam dam spillway.

Case Study

Srisailam dam spillway (Figure 64) across river Krishna was constructed during 1977-83. It is a 137 m high concrete dam, with 12 spans of 18’3 m **x** 16’8 m. The river bed is composed of quartzites and shales. In the immediate downstream vicinity of the spillway, there were horizontal shear zones 0’2 m to 0’9 m thick, where the quartzites are crushed and sheared. During the monsoons of 1977 to 1980, the construction stage flood passed over the partially constructed spillway bays, spilling over 7 bays which were at different levels having a maximum difference of level of 23 m. The difference in level between the lip of the ski-jump bucket and downstream rock was about 44 m.

Shorter throw of the water spilling over the bucket lip, as a cascading flow caused deep scour in the immediate vicinity of the bucket lip. During subsequent floods, the scour holes were concreted and leveled as protective aprons in some part of the spillway. Such aprons were however, subjected to repeated damage and undermining. By April 1985, depth of scour below blocks 11 to 13 reached from 9 m to 22 m below the protective apron. Cavities of undermining below the apron were also present at a depth of 6 to 9 m.

The protection work consisted of providing an underwater massive concrete block touching the apron and filling the eroded cavities below the apron. The water level at downstream toe varied from the top of existing apron to about 1.5 m below it.

The scheme involved forming 4 cells with steel cylinder walls and filling concrete in each cell followed by concrete capping. Heavy concrete blocks (approximate 1 metre cube) were placed downstream of the cylinder watls to further protect the rock from the water jump damage.

Since the construction of the above protection works, the spillway was completed to final levels and crest gates have also been installed. Hydraulic model studies were conducted to evolve an operation of the spillway in such a way that the throw of the trajectory fall further away from the toe of the dam. This together with the protective measures already implemented is expected to prevent further erosion at the toe of the dam.

Module
4
Hydraulic Structures for Flow Diversion and Storage

Lesson 9

Reservoir Outlet Works

Instructional objectives

On completion of this lesson, the student shall learn:

1. Functions of outlet works
2. Classification of outlet works
3. Determination of design discharge and elevation of outlet works
4. Choice of outlet works and their layout
5. Components of outlet works
6. Hydraulic design of outlet works

4.9.0 Introduction

Water from the reservoir of a dam is released through two principal types of structures:

1. Spillways, which are provided for storage and detention dams to release surplus water or floodwater that cannot be contained in the allotted storage space, and for diversion dams to bypass flows exceeding those turned into the diversion system.

2. Outlet works, which regulate or release water impounded by a dam. It can release incoming flows at a retarded rate, as does a detention dam; it can divert incoming flows into canals or pipelines, as does a diversion dam; or it can release stored waters at rates dictated by downstream needs, by evacuation considerations, or by a combination of multiple-purpose requirements.

Outlet works are so named because they release out water from the reservoir. Some of these are equipped with an Intake Structure if the water is fed into a canal or a conduit for serving some specific purpose like meeting irrigation water requirement or hydropower generation, etc.

Occasionally, the outlet works may be placed at a level high enough to deliver water to a canal, while a bypass is extended to the river to furnish necessary flows below the dam. Such bypass flows may be required to satisfy prior-right uses downstream or to maintain a live stream for abatement of stream pollution, preservation of aquatic life, or other purposes. Dams constructed to provide reservoirs principally for recreation or for fish and wildlife conservation require a fairly constant reservoir level. For such dams an outlet works may be needed only to release the minimum flows necessary to maintain a live stream below the dam. In certain cases, the outlet works of a dam may be used in lieu of a service spillway combined with an auxiliary or secondary spillway. In such a case, the usual outlet works installation might be modified to include a bypass overflow so that the structure can serve as both an outlet works and a spillway.

An outlet work may act as a flood control regulator to release waters temporarily stored in flood control storage space or to deplete the storage of a reservoir in anticipation of flood inflows. Furthermore, the outlets may be used to empty the reservoir to permit inspection, to allow needed repairs, or to maintain the upstream face of the dam or other structures normally inundated. The outlets may also aid in lowering the reservoir storage when controlling objectionable aquatic life in the reservoir is desired. Sometimes reservoirs are lowered and raised in a sequence to control the menace of malaria.

4.9.1 Classification of outlet works

Outlet works structures can be classified according to their purpose, their physical and structural arrangement, or their hydraulic operation. An outlet work that empties directly into a river could be designated a "river outlet"; one that discharges into a canal could be designated a "canal outlet"; and one that delivers water into a closed pipe system could be designated a "pressure pipe outlet." An outlet work may be described according to whether it consists of an open-channel or closed-conduit waterway, or whether the closed waterway is a conduit in cut-and-cover or in a tunnel. An outlet works may also be classified according to its hydraulic operation: whether it is gated or ungated or, for a closed conduit, whether it flows under pressure for part or all of its length or only as a free flow waterway.

Reservoir outlet works facilities may also be divided as those provided in concrete gravity dams or embankment dams. Outlet works through concrete gravity dams are usually called sluices while those through embankment dams are called conduits (if laid below an embankment dam or constructed by cut-and-cover procedure) or tunnels (if excavated through the abutment or foundation). Certain specific requirement of sluices through concrete gravity dams and embankment dams are given below:

a. Concrete Gravity Dams. Generally, sluices that traverse through concrete gravity dams have rectangular cross sections and are short in comparison with conduits through embankment dams of comparable height. Use of a number of small sluices, at one or more elevations provides flexibility in flow regulation and in quantity of water released downstream. Sluices are controlled by gates at the upstream face and/or by gates or valves operated from a gallery in the interior of the dam. Sluices are usually designed so that the outflow discharges onto the spillway face and/or directly into the stilling basin. When sluices traverse through non-overflow sections, a separate energy dissipator must be provided.

b. Embankment Dams. Conduits and/or tunnels for embankment dams may have circular, rectangular, horseshoe, or oblong cross sections and their length is primarily determined by the base width of the embankment. Due to the greater length, it is usually more economical to construct a single large conduit than a number of small conduits. Conduits should be tunneled through the abutment as far from the embankment as practicable, or placed in an open cut through rock in the abutment or on the valley floor. Gates and/or valves in an intake tower in the reservoir, in a central control shaft in the abutment or embankment, or at the outlet portal are used to control the flow. Generally, placement of the control device at the outlet portal should be avoided when the conduit passes through the embankment due to the inherent dangers of a possible rupture of a conduit subject to full reservoir head. Diversion during construction or reservoir evacuation requirements, especially on large streams, may govern the size and elevation of the conduit(s).

4.9.2 Functions of outlet works

1. **Flood Control.** Flood control outlets are designed for relatively large capacities where close regulation of flow is less important than are other requirements. Although control of the outflow by gates is usually provided, the conduits may be ungated, in which case the reservoir is low or empty except in time of flood. When large discharges must he released under high heads, the design of gates, water passages, and energy dissipator should be carefully developed. Multilevel release provisions are often necessary for water quality purposes.

2. **Navigation.** Reservoirs that store water for subsequent release to downstream navigation usually discharge at lower capacity than flood control reservoirs, but the need for close regulation of the flow is more important. The navigation season often coincides with the season of low rainfall, and close regulation aids in the conservation of water. Outlet works that control discharges for navigation purposes are required to operate continuously over long periods of time. The designer should consider the greater operation and maintenance problems involved in continuous operation.

3. **Irrigation.** The gates or valves for controlling irrigation flows are often basically different from those used for flood control due to the necessity for close regulation and conservation of water in arid regions. Irrigation discharge facilities are normally much smaller in size than flood regulation outlets. The irrigation outlet sometimes discharges into a canal or conduit rather than to the original riverbed. These canals or conduits are usually at a higher level than the bed of the stream. This has been implemented for the canal off-taking from the Sardar Sarovar Dam on River Narmada. At other times, irrigation releases are let down to the river from where a barrage downstream picks up the flow and diverts it into an irrigation canal. For example, Bhakra Dam and Nangal Barrage on River Sutlej, Maithon dam and Durgapur Barrage on River Damodar and Hirakud Dam and Mahanadi Barrage on River Mahanadi etc. were built in conjunction with one another to serve a similar purpose.

4. **Water Supply.** Municipal water supply intakes are sometimes provided in dams built primarily for other purposes. Such problems as future water supply requirements and peak demands for a municipality or industry should be determined in cooperation with engineers representing local interests. Reliability of service and quality of water are of prime importance in water supply problems. Multiple intakes

and control mechanisms are often installed to assure reliability, to enable the water to be drawn from any selected reservoir level to obtain water of a desired temperature, and/or to draw from a stratum relatively free from silt or algae or other undesirable contents. Ease of maintenance repair without interruption of service is of primary importance. An emergency closure gate for priority use by the resident engineer is required for water supply conduits through the dam.

5. **Power.** For generation of hydropower, intake structures direct water from the reservoir into the penstock or power conduit. Gates or valves are used to shut off the flow of water to permit emergency unit shutdown or turbine and penstock maintenance. Racks or screens prevent trash and debris from entering the turbine units. Where the powerhouse is integral with the dam, the intake is part of the dam structure. Where the powerhouse is not part of the dam, a separate intake structure must be provided. Projects that are required to use water at a selected temperature must have multi-level intakes in order to control inlet water quality by mixing waters obtained from different levels.

6. **Low-Flow Requirements.** Continuous low-flow releases are required at some dams to satisfy environmental objectives, water supply, downstream water rights, etc. To meet these requirements multilevel intakes, skimmer weirs, or other provisions must be incorporated separately or in combination with other functions of the outlet works facility. Special provisions for these purposes have to be incorporated in concrete gravity dam non-overflow sections. Embankment dams with mid-tunnel control shafts also require special considerations for low-flow releases.

7. **Diversion.** Flood control outlets may be used for total or partial diversion of the stream from its natural channel during construction of the dam. Such use is especially adaptable for earth dams.

8. **Drawdown.** Requirements for low-level discharge facilities for drawdown of impoundments may also provide flexibility in future project operation for anticipated needs, such as major repairs of the structure, environmental controls, or changes in reservoir regulation.

4.9.3 Determination of design discharge capacities

Outlet works are designed to release water at specific rates. These rates are dictated by downstream needs, by flood control regulation, by storage considerations, by power generation needs (where the outlet works is used as the penstock for power plants), and by legal requirements. Delivery of irrigation water is usually determined from project or farm needs and is related to the consumptive use and to the special water requirements of the irrigation system. Delivery for domestic use can be similarly established. Releases of flows to satisfy prior rights must generally be included with other needed releases. Minimum downstream flows for pollution abatement, fish preservation, and associated needs are often accommodated through other required releases. A small bypass pipe is often used to provide these minimum releases. This pipe usually originates at the gate chamber or in the downstream control structure, depending on the type of outlet works.

Irrigation outlet capacities are determined from reservoir operation studies. They must be based on a consideration of a critical period of low runoff when reservoir storages are low and daily irrigation demands are at their peak. The most critical draft from the reservoir, considering such demands (commensurate with remaining reservoir storage) together with prior rights and other needed releases, generally determines the minimum irrigation outlet capacity. These requirements are stated in terms of discharge at either a given reservoir content or a given water surface elevation. Evacuation of water stored in an allocated flood control storage space of a reservoir can be accomplished through a gated spillway at the higher reservoir levels or through an outlet at the lower levels.

Flood control releases generally can be combined with the irrigation releases if the outlet empties into a river instead of into a canal. The capacity of a flood control outlet can be determined by the required time of evacuation of the given storage space, considering the inflow into the reservoir during the evacuation. Combined flood control and irrigation releases ordinarily must not exceed the safe channel capacity of the river downstream from the dam and must allow for all anticipated inflows immediately below the dam. These inflows may be natural run-offs, or the results of releases from storage developments along the river or from developments on tributaries emptying into the river.

If an outlet is to serve as a service spillway in releasing surplus inflows from the reservoir, the discharge required for this purpose may determine the outlet capacity. Similarly, the minimum outlet capacity can be determined by the discharge and the time required to empty the reservoir for inspection, maintenance, repair, or emergency drawdown. Here again, the inflow into the reservoir during the emptying period must be considered. The capacity at low reservoir level should be at least equal to the average inflow expected during the maintenance or repair period. It can, of course, be assumed that required repair will be delayed until service demands are light and that repairs will be made during low inflow and during seasons favorable to such construction.

An outlet works cut-and-cover conduit or tunnel is often used to divert the river flow during the construction period, precluding supplementary installations for that purpose. The outlet structure size dictated by this use, rather than the size dictated by ordinary outlet works requirements, may determine the final outlet works capacity. A diversion bypass pipe may be required to satisfy downstream requirements during placement of second-stage concrete and gates in the outlet works.

4.9.4 Position (elevation) of outlet works in relation to reservoir storage levels

The establishment of the intake level and the elevations of the outlet controls and the conveyance passageway, as they relate to the reservoir storage levels, are influenced by many factors. Primarily, to attain the required discharge capacity, the outlet must be placed sufficiently below the minimum reservoir operating level to provide the head required for outlet works flows.

Outlet works for small detention dams are generally constructed near riverbed level because permanent storage space, except for silt retention, is ordinarily not provided. These outlet works may be ungated to retard the outflow while the reservoir temporarily stores the bulk of the flood runoff, or they may be gated to regulate the releases of the temporarily stored waters. If the purpose of the dam is only to raise the reservoir and divert incoming flows at low heads, the main outlet works generally should be an intake or regulating structure at a high level. A sluiceway or small bypass outlet should also be provided to furnish water to the river downstream or to drain the water from behind the dam during off-season periods. Dams that impound water for irrigation, for domestic use, or for other conservation purposes, must have outlet works low enough to draw the reservoir down to the bottom of the allocated storage space; however, the outlet works may be placed above the riverbed, depending on the established minimum reservoir storage level. It is common practice to make an allowance in a storage reservoir for inactive storage to accommodate sediment deposition, for fish and wildlife conservation, and for recreation. The positioning of the intake sill then becomes an important consideration; it must be high enough to prevent interference from the sediment deposits, but at the same time, low enough to permit either a partial or a complete drawdown below the top of the inactive storage.

Where an outlet is placed at riverbed level to accommodate the construction diversion plan or to drain the reservoir, the operating sill may be placed at a higher level to provide a sediment and debris basin and other desired inactive storage space, or the intake may be designed to permit raising the sill as sediment accumulates. During construction, a temporary diversion opening may be formed in the base of the intake to handle diversion flows. Later, this opening may be plugged. For emptying the reservoir, a bypass around the intake may be installed at riverbed level. This bypass may either empty into the lower portion of the conduit or pass under it. Water can be delivered to a canal at a higher level by a pressure riser pipe connecting the conduit to the canal.

4.9.5 Choice of outlet works and their layout

The layout of an outlet work is influenced by many conditions relating to the hydraulic requirements, to the site adaptability, to the interrelation of the outlet works and the construction procedures, and to the other appurtenances of the development. Thus, an outlet work leading to a high-level canal or into a closed pipeline might differ from one emptying into the river. Similarly, a scheme in which the outlet works is used for diversion might vary from one where diversion is effected by other means. In certain instances, the proximity of the spillway may permit combining some of the outlet works and spillway components in a single structure. For example, the spillway and outlet works layout might be arranged so that discharges from both empty into a common stilling basin.

The topography and geology of a site may have a great influence on the layout selection. Some sites may be suited only for a cut-and-cover conduit type of outlet works; whereas, at other sites, either a cut-and-cover conduit or a tunnel may be selected. Unfavorable foundation geology, such as deep over-burdens or inferior foundation rock, precludes the selection of a tunnel scheme. On the other hand, sites in narrow canyons with steep abutments may make a tunnel outlet the only choice. Because of confined working space and excessive costs where hand-construction methods must be used, building a tunnel smaller than about 2 metre in diameter is not practicable. However, a cut-and-cover conduit can be built to almost any size if it is precast or cast-in-place with the inside bore formed by a pre-fabricated liner. Thus, the minimum size dictated by construction conditions, more than the size dictated by hydraulic requirements, influences the choice of either the cut-and-cover conduit or the tunnel scheme. The amount of load to be taken by a conduit will also affect this choice.

The outlet works for a low dam, whether it is to divert water into a canal or release it to the river, often consists of an open-channel or cut-and-cover structure at the dam abutment. The structure may consist of a conventional open flume or rectangular channel with a gate similar to that used for ordinary spillway installations, or it may be regulated by a submerged gate placed to close off openings in a curtain or headwall. Where the outlet is to be placed through a low earthfill embankment, a closed structure may be used. This structure may consist of single or multiple units of buried pipe or box culverts placed through or under the embankment. Flow for such an installation could be controlled by gates placed at the inlet or at an intermediate point along the conduit, such as at the crest of the embankment, where a shaft would be provided for gate operation. Downstream from the control structure, the channel would continue to the canal or to the river where, depending on the exit velocities, a stilling device. Figure I shows typical installations of the arrangements described above.

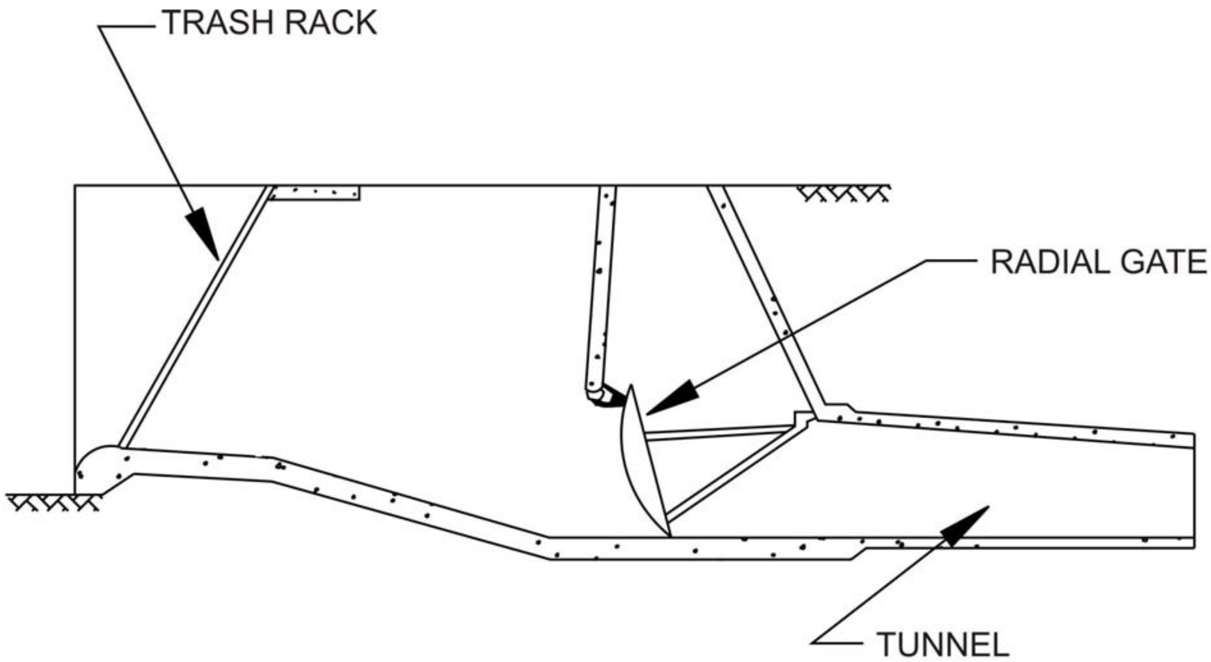


FIGURE 1. OUTLET FOR LOW-HEAD INSTALLATION WITH FREE-FLOW DOWNSTREAM TUNNEL

For higher earthfill dams, where an open-channel outlet structure would not prove feasible, the outlet might be carried through, under, or around the dam as a cut-and-cover conduit or through the abutment as a tunnel. Depending on the position of the control device, the conduit or tunnel may be free flowing, flowing under pressure for a portion of its length, or flowing under pressure for its entire length. Intakes may be arranged to draw water from the bottom of the reservoir, or the inlet sills may be placed at some higher reservoir level. Dissipating devices may be used at the downstream end of the conduit. The outlet works also may discharge into the spillway stilling basin. Depending on the method of control and the flow conditions in the structure, access to the operating gates may be by bridge to an upstream intake tower, by shaft from the crest level of the dam, by walkway within the conduit or tunnel with entrance from the downstream end, or by a separate conduit or tunnel access adit. Arrangements typical of those described above are shown on Figures 2 through 5.

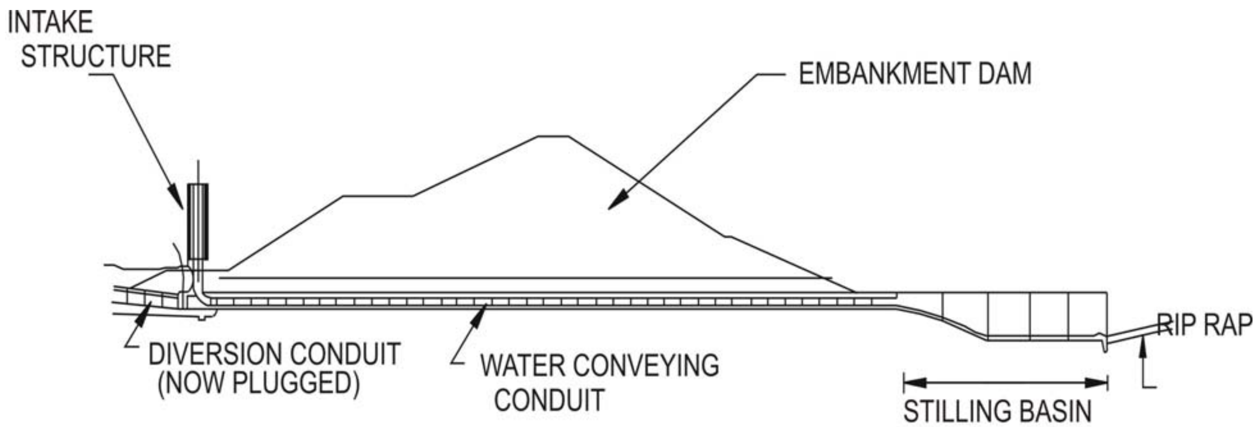


FIG2. OUTLET FOR INTAKES OF MEDIUM HEAD INSTALLATION WITH FREE-FLOW CONDUIT AND HYDRAULIC JUMP STILLING BASIN

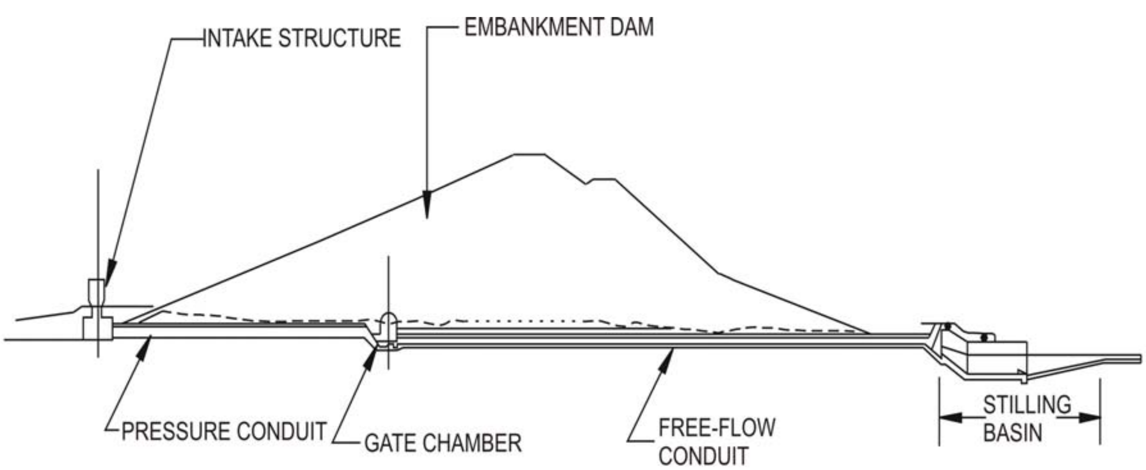


FIG3. OUTLET WIYH A PRESSURE CONDUIT UPSTREAM OF GATE CHAMBER AND FREE-FLOW PIPE IN DOWNSTREAM

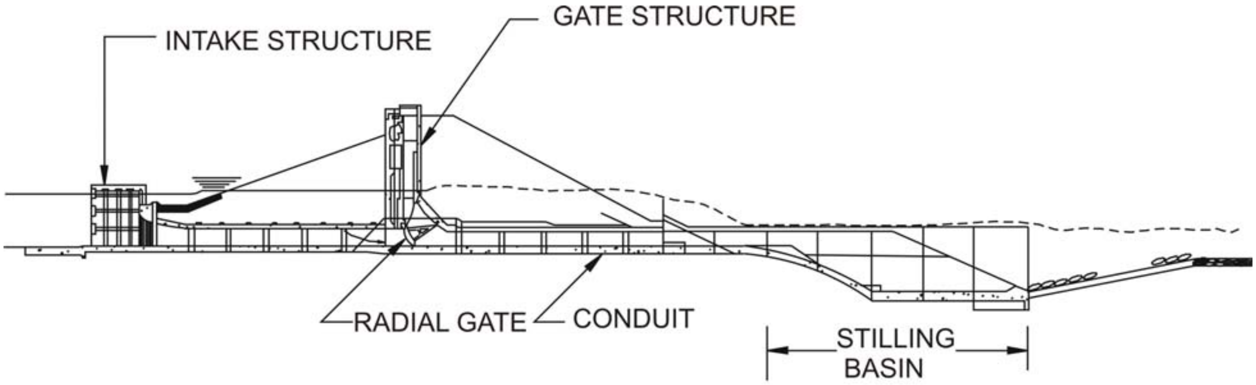


FIG4. OUTLET EITH A TRASH-RACK BOX INTAKE,PRESSURE CONDUIT ON UPSTREAM AND FREE FLOW CONDUIT ON DOWNSTREAM

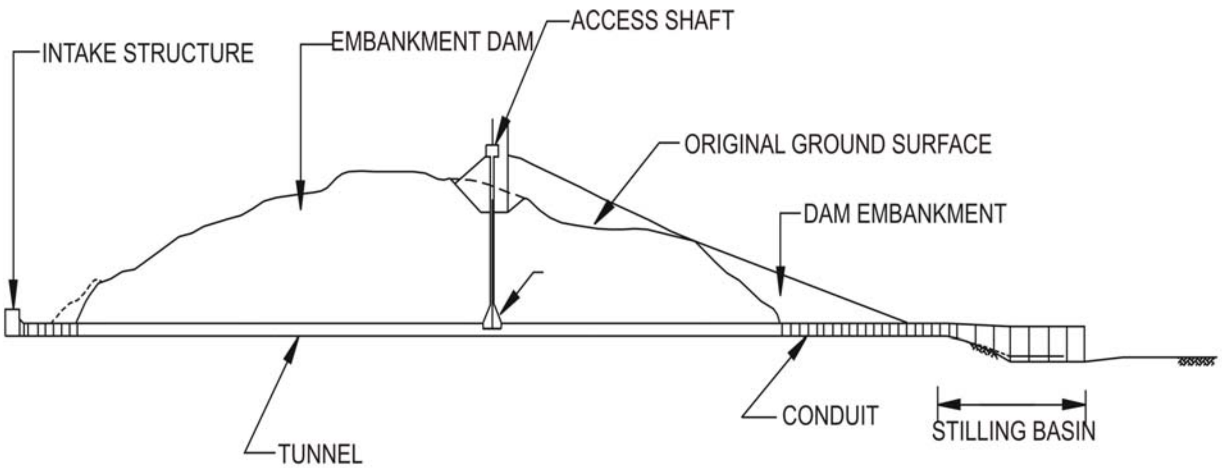


FIG5. OUTLET WITH TRASHRACK INTAKE STRUCTURE,PRESSURE TUNNEL UPSTREAM OF GATE CHAMBER,FREE TUNNEL AND CONDUIT ON DOWNSTREAM

For a concrete dam, the outlet works installation should usually be carried through the dam as a formed conduit or a sluice, or as a pipe embedded in the concrete mass. Intakes and terminal devices may be attached to the upstream and downstream faces of the dam. Often, the outlet is formed through the spillway overflow section using a common stilling basin to dissipate both spillway and outlet works flows. Where an outlet works conduit is installed in the non-overflow section of the dam or where an outlet must empty into a canal, a separate dissipating device will, of course, be necessary. Instead of one large conduit, several smaller conduits may be used in a concrete dam to provide a less expensive and more feasible arrangement for handling the outlet works releases. The multiple conduits may be placed at a single level or, for added flexibility, at several levels. Such an arrangement would reduce the cost of the control gates because of the lower heads on the upper-level gates. Details of typical outlet works installations for the concrete dam at Bhakra are shown in Figures 6 and 7.

A diversion tunnel used during the construction of a concrete dam can often be converted into a permanent outlet works by providing outlet sluices or conduits through the tunnel plug. Ordinarily, the diversion tunnel for a concrete dam will be in good quality rock and will therefore require little lining protection. Furthermore, the outlet portal of the tunnel will generally be located far enough downstream from the dam so that no dissipating structure will be needed or, at most, only a deflector will be required to direct the flow to the downstream river channel.

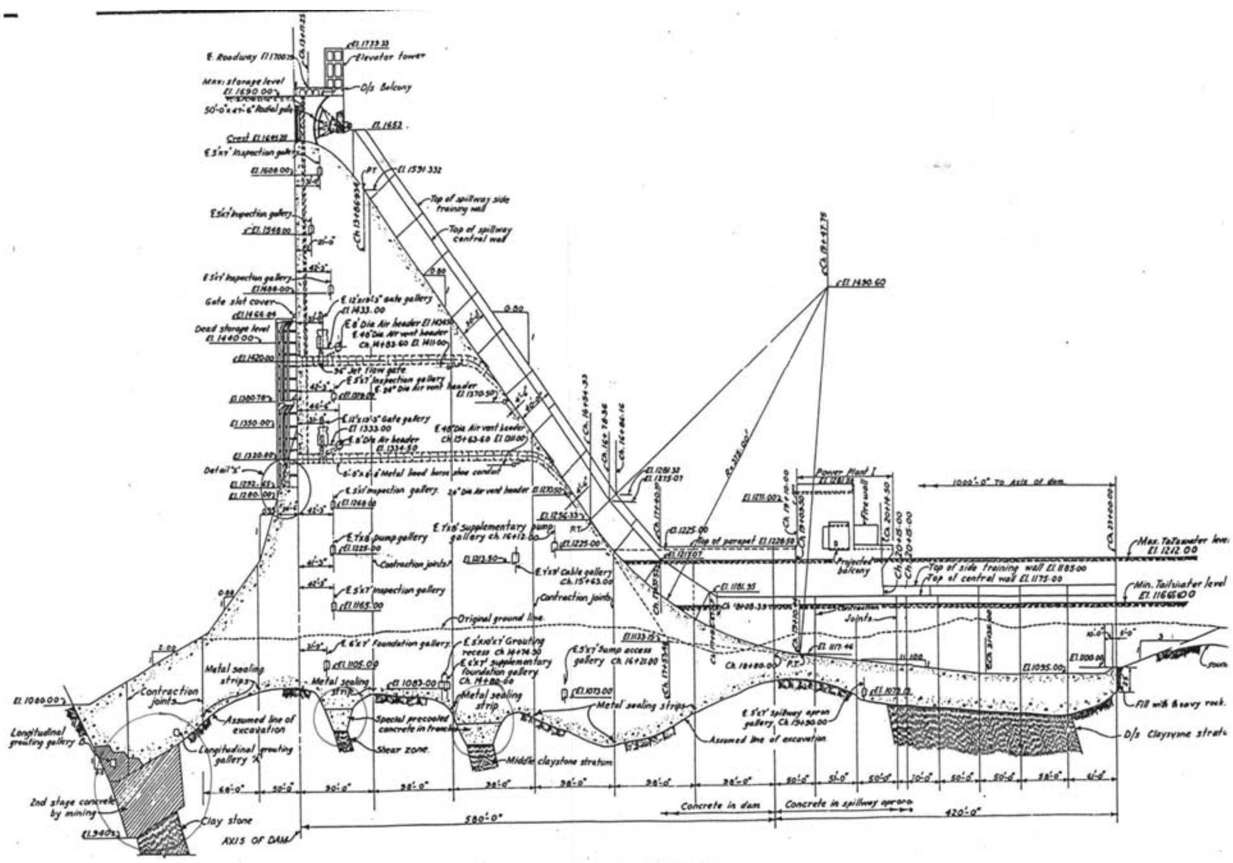


FIGURE 6. MAXIMUM SPILLWAY SECTION OF BHAKRA DAM SHOWING RIVER OUTLETS

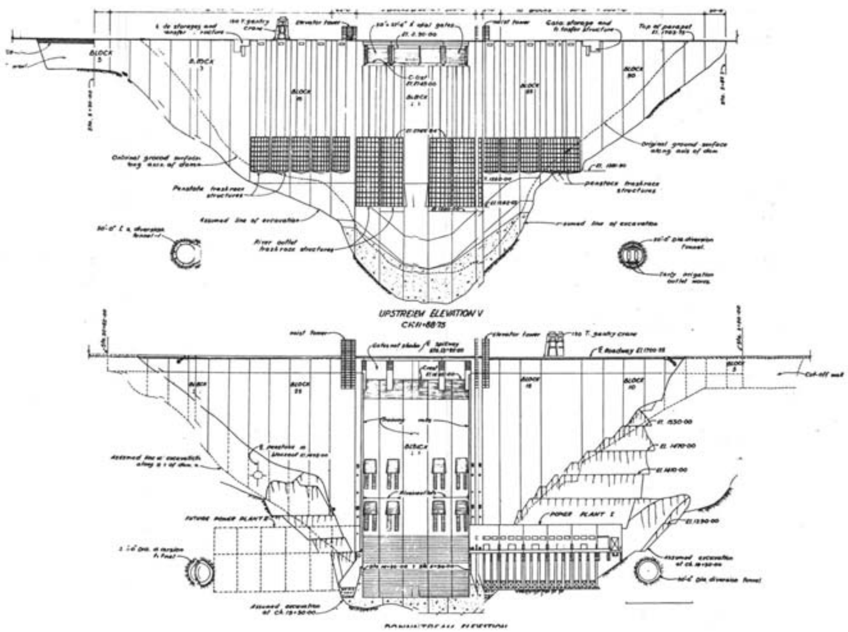


FIGURE 7. UPSTREAM AND DOWNSTREAM ELEVATION OF BHAKRA DAM SHEARING POSITION OF RIVERS OUTLETS

4.9.6 Components of outlet works

For an open-channel outlet works or for a conduit-type outlet where partial full flow prevails, the control gates or valves should determine the outlet works capacity. Where an outlet works operates as a pressure pipe, the size of the waterway and that of the control device should determine the capacity. The overall size of an outlet works is determined by its hydraulic head and the required discharge. The selection of the size of some of the component parts of the structure, such as the tunnel, is dictated by practical considerations or by interrelated requirements such as diversion, reservoir evacuation, and initial filling.

When the type of waterway has been chosen and the method of control established, the associated structures to complete the layout can be selected. The type of intake structure depends on its location and function and on the various appurtenances, such as fish-screens, trash racks, stop log arrangements, or operating platforms that must be furnished. A means for dissipating the energy of flow before returning the discharge to the river should normally be provided. This can be accomplished by a flip bucket, a stilling basin, a baffled apron drop, a stilling well, or a similar dissipation device. Gate chambers, control platforms, or enclosures may be required to provide operating space and protective housing for the control devices. An outlet works may also require an outlet channel to return releases to the river and an entrance channel to lead diversion flows or low-reservoir flows to the intake structure.

Tunnels

Because of its inherent advantages, a tunnel outlet works is preferred where abutment and foundation conditions permit its use and it is more economical than the other types of outlet works. A tunnel is not in direct contact with the dam embankment and, therefore, provides a much safer and more durable layout than can be achieved with a cut-and-cover conduit. Little foundation settlement, differential movement, and structural displacement is experienced with a tunnel that has been bored through competent abutment material, and seepage along the outer surfaces of the tunnel lining or leakage into the material surrounding the tunnel is less serious. Furthermore, it is less likely that failure of some portion of a tunnel would cause failure of the dam than the failure of a cut-and-cover conduit that passes under or through the dam.

Cut-and-Cover Conduits

If a closed conduit is to be provided and foundation conditions are not suitable for a tunnel, or if the required size of the waterway is too small to justify the minimum sized tunnel, a cut-and-cover conduit should be used. Because this type of conduit passes through or under the dam, conservative and safe designs must be used. Numerous failures of earthfill dams caused by improperly designed or constructed cut-and-cover outlet conduits have demonstrated the need for conservative procedures.

Control Devices

Selection of the outlet works arrangement should be based on the use of commercially available gates and valves or relatively simple gate designs where possible. The use of special devices that involve expensive design and fabrication costs should be avoided. Cast iron slide gates, which may be used for control and guard gates, are available for both rectangular and circular openings and for design heads up to about 15 metre. However, higher head installations require special gate designs. Simple radial gates are available for ordinary surface installations, and top-seal radial gates can be secured from manufacturers on the basis of simple designs and specifications. For low heads up to about 15 metre, commercial gate and butterfly valves are suitable for control at the downstream end of pressure pipes if they are designed to operate under free discharge conditions with the jet well aerated all around. Gate and butterfly valves are also suitable for use as inline guard valves and can be adapted for inline control valves if air venting and adequate aeration of the discharge jet are provided immediately downstream from the valve.

The control gate for an outlet works may be placed at the upstream end of the conduit, at an intermediate point along its length, or at the lower end of the structure. Where flow from a control gate is released directly into the open as free discharge, only that portion of the conduit upstream from the gate is under pressure. Where a control gate or valve is placed at the lower end of the structure, full internal pressure should be considered in the design of the conduit tunnel or pipe. However, when a control discharges into a free-flow conduit, the location of the control gate becomes important in the design of the outlet. Upstream gate controls for conduits are generally placed in a tower structure with the gate hoists mounted on the operating deck (Figure 2). With this arrangement, the tower must extend above the maximum water surface. If controls are to be located at some intermediate point along the conduit, high-pressure gates, slide gates, and top-seal radial gates may be used. These controls may be located in a wet-well shaft that extends vertically from the conduit level to the crest of the dam. Typical arrangements of these installations are shown in Figures 1 to 4.

Intake Structures

In addition to forming the entrance to the outlet works, an intake structure may accommodate control devices. It also supports necessary auxiliary appurtenances (such as trashracks, fishscreens, and bypass devices), and it may include temporary diversion openings and provisions for installation of bulkhead or stoplog closure devices. Intake structures may appear in many forms. The type of intake structure selected should be based on several factors: the functions it must serve, the range in reservoir head under which it must operate, the discharge it must handle, the frequency of reservoir drawdown, the trash conditions in the reservoir (which will determine the need for or the frequency of cleaning of the trashracks), reservoir wave action that could affect the stability, and other similar considerations. Depending on its function, an intake structure may be either submerged or extended in the form of a tower above the maximum reservoir water surface. A tower must be provided if the controls are placed at the intake, or if an operating platform is needed for trash removal, maintaining and cleaning fish-screens, or installing stoplogs. Where the structure serves only as an entrance to the outlet conduit and where trash cleaning is ordinarily not required, a submerged structure may be adopted.

The necessity for trashracks on an outlet works depends on the size of the sluice or conduit, the type of control device used, the nature of the trash burden in the reservoir, the use of the water, the need for excluding small trash from the outflow, and other factors. These factors determine the type of trashracks and the size of the openings. Where an outlet consists of a small conduit with valve controls, closely spaced trash bars are needed to exclude small trash. Where an outlet involves a large conduit with large slide-gate controls, the racks can be more widely spaced. If there is no danger of clogging or damage from small trash, a trashrack may consist simply of struts and beams placed to exclude only larger trees and similarly sized floating debris. The rack arrangement should also be based on the accessibility for removing accumulated trash. Thus, a submerged rack that seldom will be dewatered must be more substantial than one at or near the surface. Similarly, an outlet with controls at the entrance, where the gates can be jammed by trash protruding through the rack bars, must have a more substantial rack arrangement than one whose controls are not at the entrance.

Energy Dissipating Arrangements

The discharge from an outlet, whether of a gate valve, or free flow conduit, will emerge at a high velocity, usually in a nearly horizontal direction. If erosion-resistant bedrock exists at shallow depths, the flow may be discharged directly into the river. Otherwise, it should be directed away from the toe of the dam by a deflector. Where erosion is to be minimized, a plunge basin may be excavated and lined with riprap or concrete. When more energy dissipation is required for free flow conduits, the terminal structures described for spillways may be used. The hydraulic-jump basin is most often used for energy dissipation of outlet works discharges. However, flow that emerges from the outlet in the form of a free jet, as is the case for valve-controlled outlets of pressure conduits, must be directed onto the transition floor approaching the basin so it will become uniformly distributed before entering the basin. Otherwise, proper energy dissipation will not be obtained.

Entrance and Outlet Channels

An entrance channel and an outlet channel are often required for a tunnel or cut-and-cover conduit layout. An entrance channel may be required to convey diversion flows to a conduit in an abutment or to deliver water to the outlet works intake during low reservoir stage. And an outlet channel may be required to convey discharges from the end of the outlet works to the river downstream or to a canal.

4.9.7 Hydraulic design of outlets

The hydraulics of outlet works usually involves either open-channel (free) flow or full conduit (pressure) flow. Analysis of open-channel flow in outlet works, either in an open waterway or in a partly-full conduit, is based on the principle of steady nonuniform flow conforming to the law of conservation of energy. Full-pipe flow in closed conduits is based on pressure flow, which involves a study of hydraulic losses to determine the total heads needed to produce the required discharges.

Three types of outlet works are briefly discussed below which are commonly provided in river valley projects.

Sluices in concrete dams

Sluices are provided in the body of the dam to release regulated supplies of water for a variety of purposes which are briefly listed below:

1. River diversion
2. Irrigation
3. Generation of hydro-electric power
4. Water supply for municipal or industrial uses
5. To pass the flood discharge in conjunction with the spillway
6. Flood control regulation to release water temporarily stored in flood control storage space or to evacuate the storage in anticipation of flood inflows
7. Depletion of the reservoir in order to facilitate inspection of the reservoir rim and the upstream face of the dam for carrying out remedial measures, if necessary
8. To furnish necessary flows for satisfying prior right uses downstream
9. For maintenance of a live stream for abatement of stream polation, preservation of aquatic life, etc.

The flow through a sluice may be either pressure flow or free flow along its entire length or a combination of pressure flow in part length and free flow in the remainder part. Sluices are also classified based upon their alignment as:

1. *Straight Barrel Sluice* - The barrel of this sluice is kept nearly horizontal between the entry and exit transitions (Figure 8). This sluice has the advantage of having minimum length due to which lesser friction losses take place. Horizontal sluices are generally used under the following conditions:
    a) When the sluices are drowned at the exit; and
    b) When they have to be located at or near the river bed level, for example, in construction sluices for river diversion.

    The width of the sluice barrel is generally kept uniform throughout the length except in the entry transaction. If the sluice is designed for pressure flow conditions then the top profile of the sluice may be given a slight constriction. On the other hand, if free flow conditions prevail then no such constriction is required.

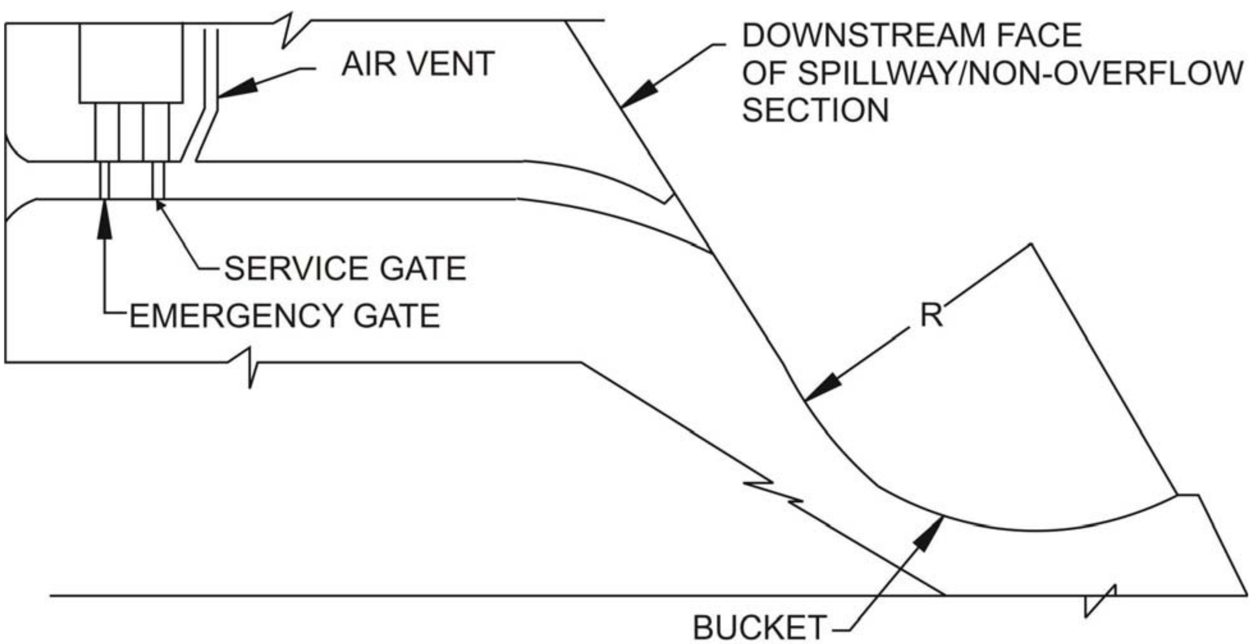


FIGURE 8. STRAIGHT BARREL LUICE

2. Trajectory Type Sluice - The barrel of this sluice is generally kept horizontal downstream of the entry transition up to the service gate to facilitate resting of the latter. Beyond the service gate the bottom of the sluice conforms to the parabolic path of the trajectory and meets the downstream face of the dam section tangentially (Figure 9).

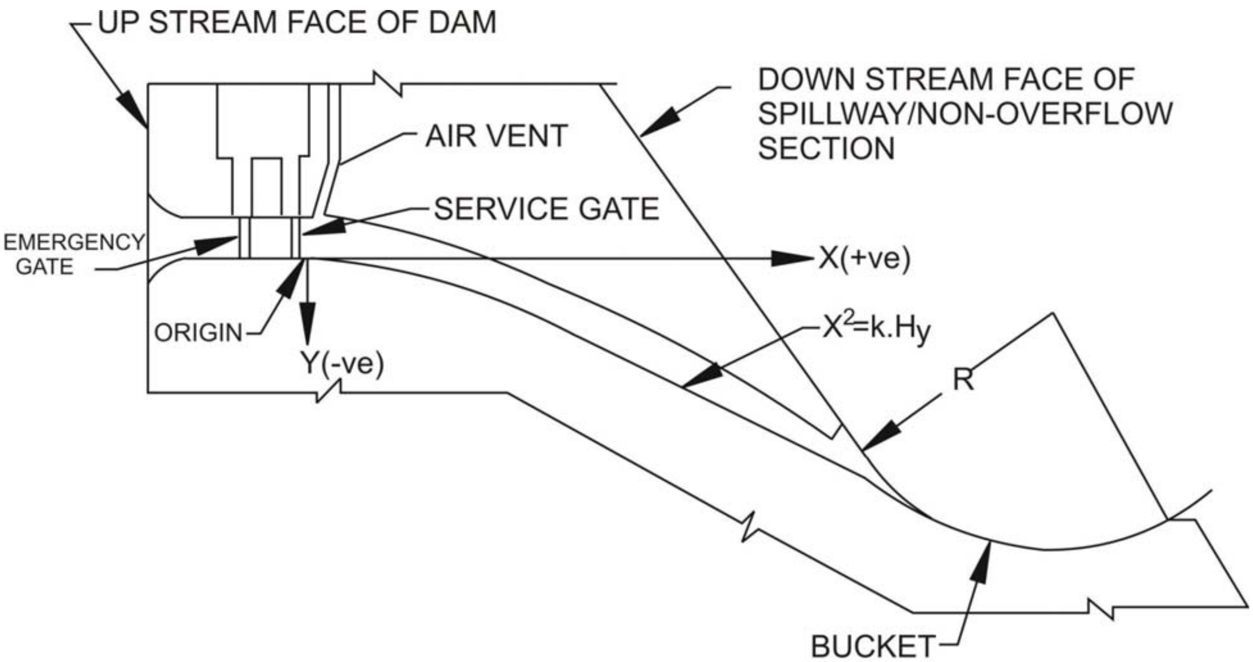


FIGURE 9 . TRAJECTORY TYPE SLUICE

For deciding upon the number and size of sluices, one has to consider the design discharge at a predetermined reservoir elevation. Details of this may be had from the Bureau of Indian Standards code IS: 11485-1985 "Criteria for hydraulic design of sluices in concrete and masonry dams".

Intakes for irrigation

Two types of intakes are generally used for outlet works meant for releasing irrigation water. These are:

1. Run-of-the-river type intakes, and
2. Reservoir type intakes.

Run-of-the-river type intakes are those which draw water from the fresh continuous river inflows without any appreciable storage upstream of the diversion structure. A typical sketch of intake to meet special characteristics, such as steep slopes, high peaks and short duration flood flows and high sediment loads, is shown in Figure 10. A canal head regulator (discussed in Lesson 4.3) is a typical example of this type of intake constructed upstream of a barrage.

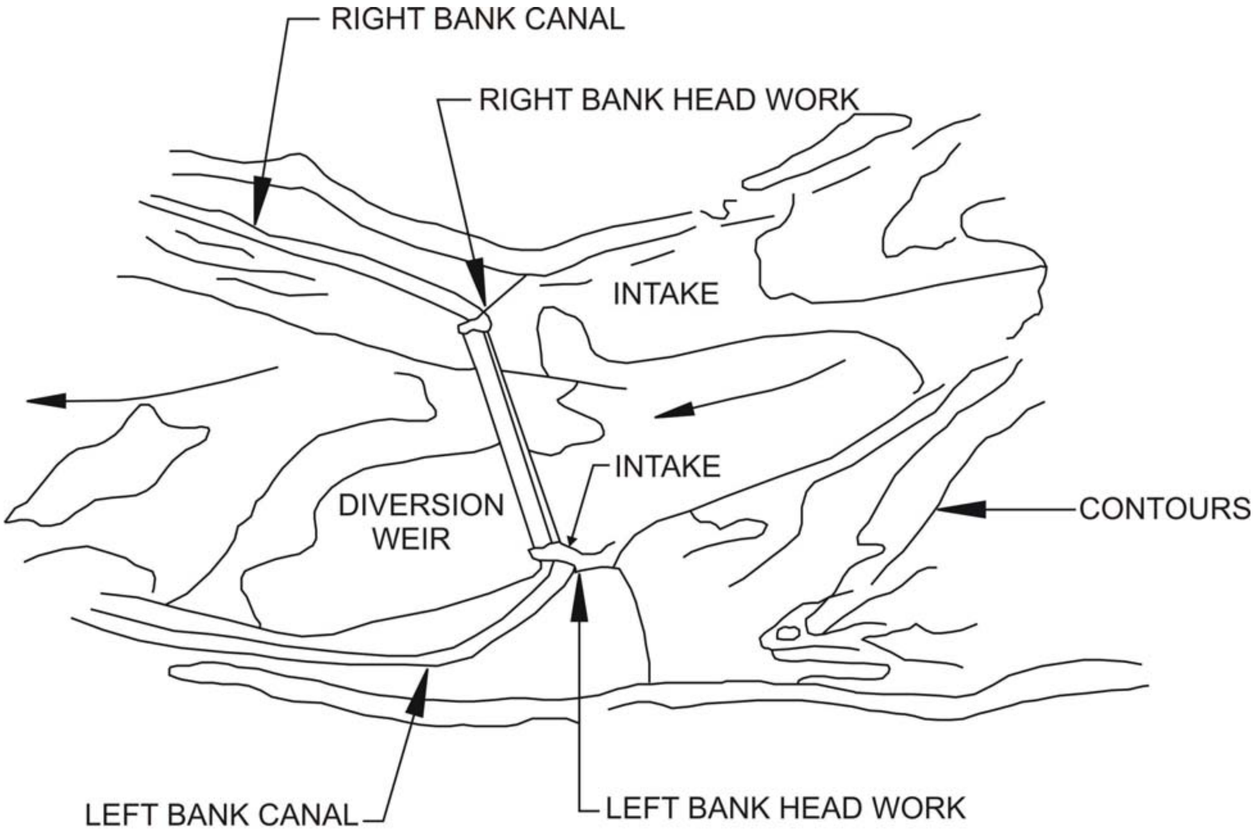


FIG10 a. RUN-OFF RIVER TYPE INTAKE (GENERAL LAYOUT)

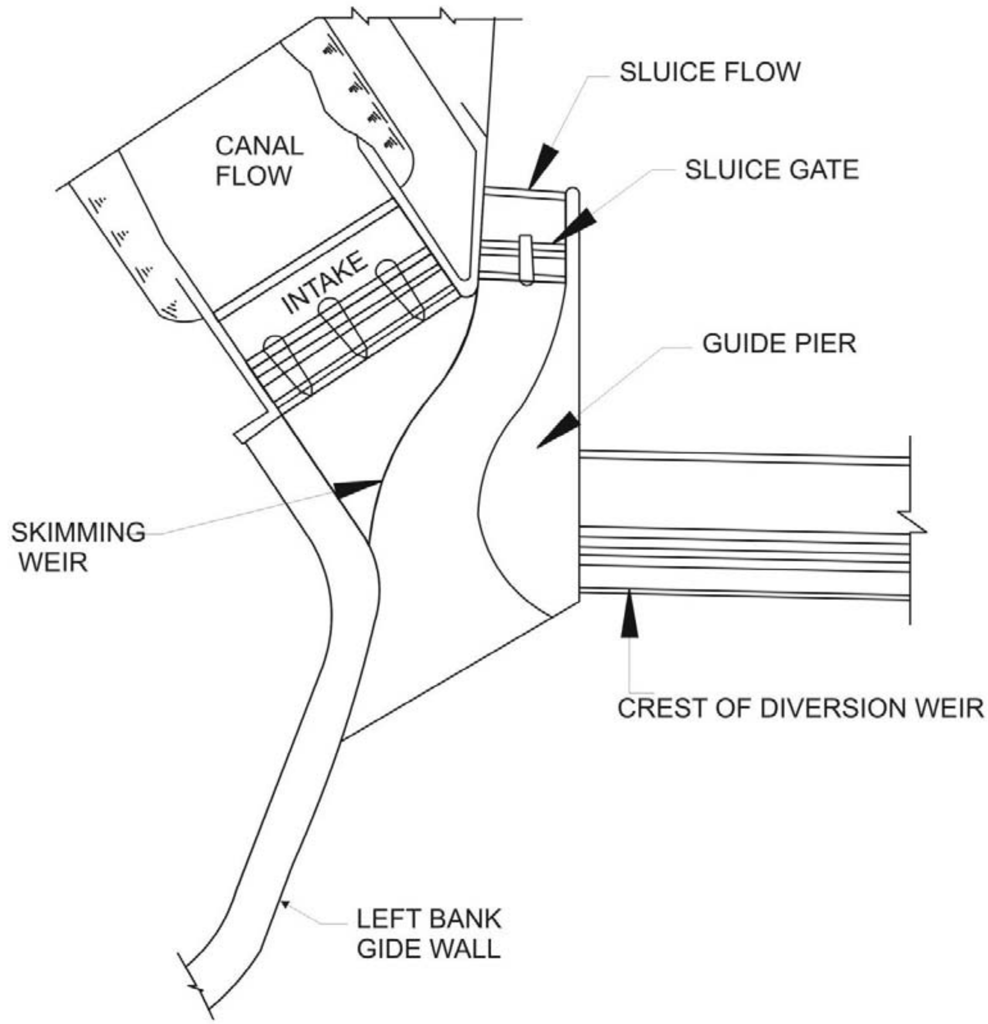


FIGURE 10(b) DETAILS OF INTAKE AND SEDIMENT EXCLUSION DEVICE THROUGH SLUICE

Reservoir type intake is provided where discharges for irrigation are drawn from storage built up for this purpose. Depending on the head, this is further categorized as under:

1. Low head (up to 15 m),
2. Medium head (15 to 30 m), and
3. High head (above 30 m).

Intake in concrete or masonry dams: In the case of concrete dams, irrigation intake structure can be located either at the toe when operating head is low or in the body of the dam itself when operating head is medium or high. Typical section of such an intake is shown in Figure 11.

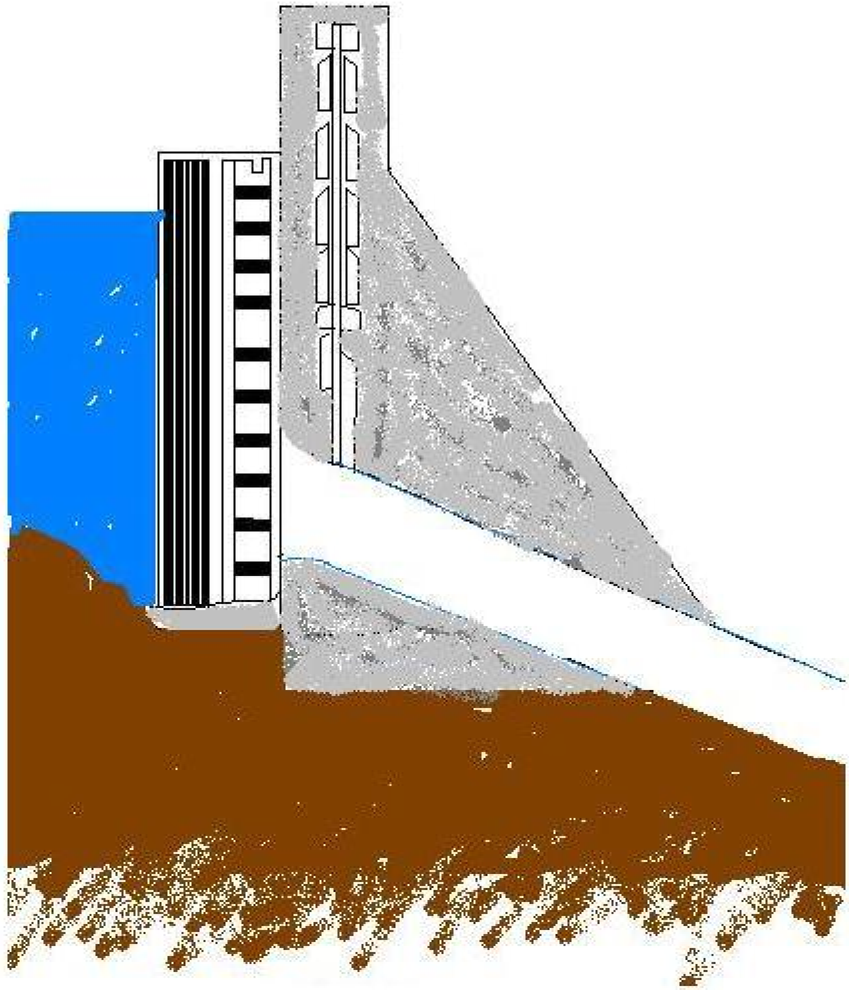


FIGURE 11. Typical intake through a concrete dam

Intake in earthen dams: When the reservoir is formed by an earthen dam, the irrigation tunnel is laid below it or in the abutment. The intake structure for such situations will be a sloping intake or tower type of intake. Typical layouts for sloping and tower type intakes are shown in Figure 12 and 13 respectively. As far as possible, reinforced cement concrete pressurized system should be avoided in the body of the earth dam. Measures like provision of steel liners and suitable drainage downstream of core, provisions of joints for differential settlements when not founded on rock should be considered in case pressure conduits are provided under earth dams.

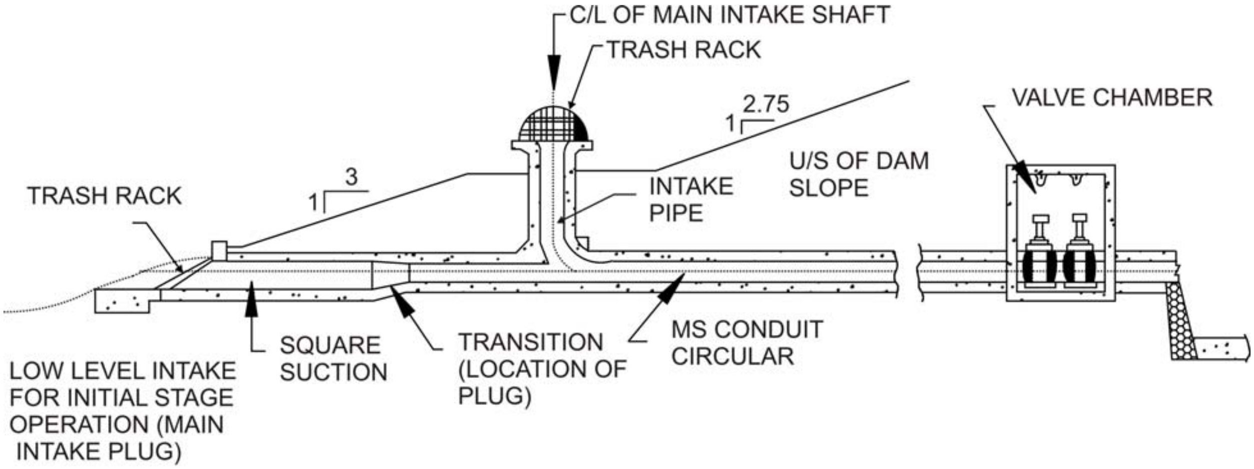


FIGURE12. TYPICAL INSTALLATION IN AN EARTH DAM-SLOPING INTAKE

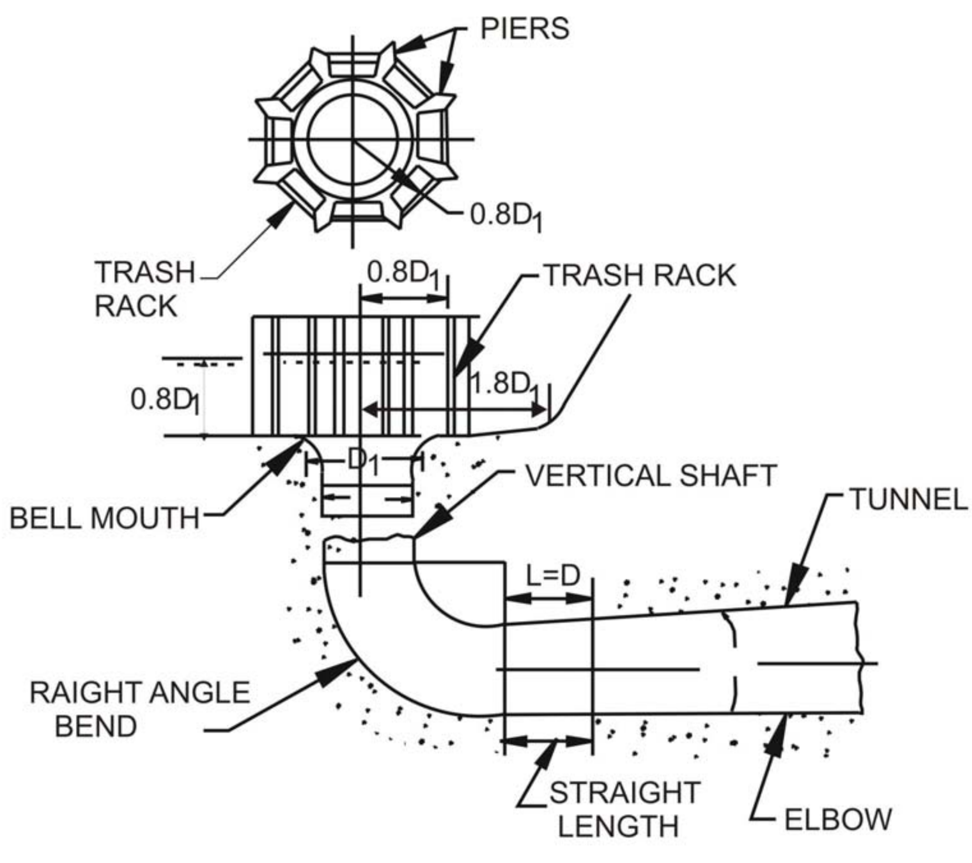


FIGURE 13 TYPICAL INSTALLATION IN AN EARTH DAM OF TOWER TYPE INTAKE

Intakes directly from the reservoir through abutments: In this case, the dam is separated from the intake structure. The dam may be of any material (concrete, earth, etc.) but the intake is constructed by grading the left or right abutments and leading the outlet pipes through the surrounding hills. Figures 14 (a) through (c) show the typical layout, section and plan of such an intake.

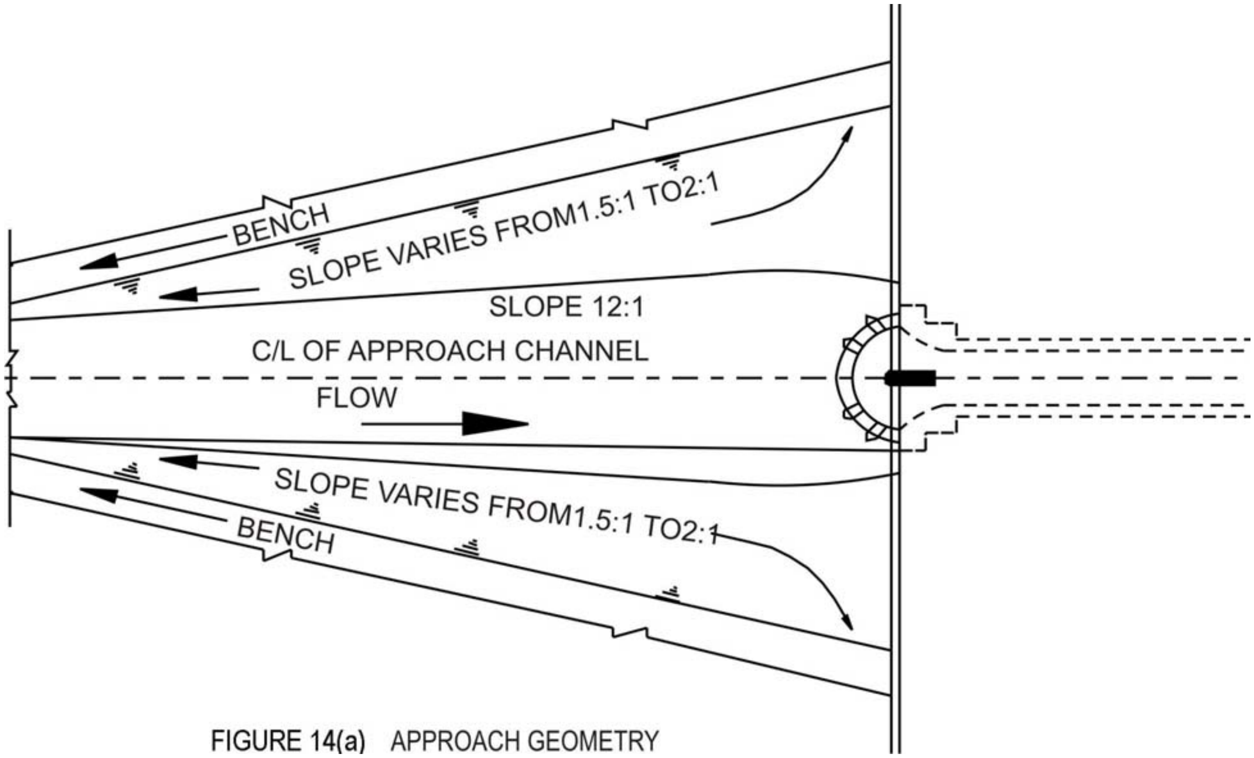


FIGURE 14(a) APPROACH GEOMETRY

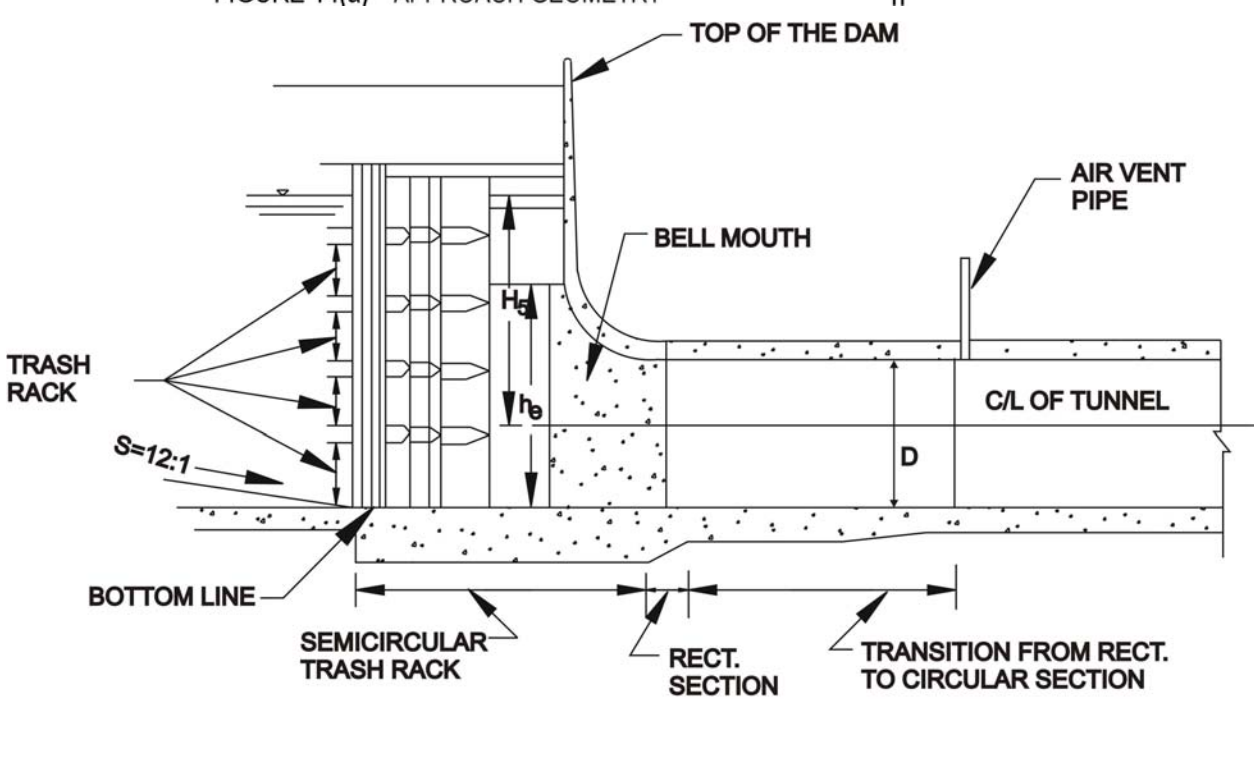


FIGURE 14 (b) SEMICIRCULER TYPE INTAKE STRUCTURE - ELEVATION

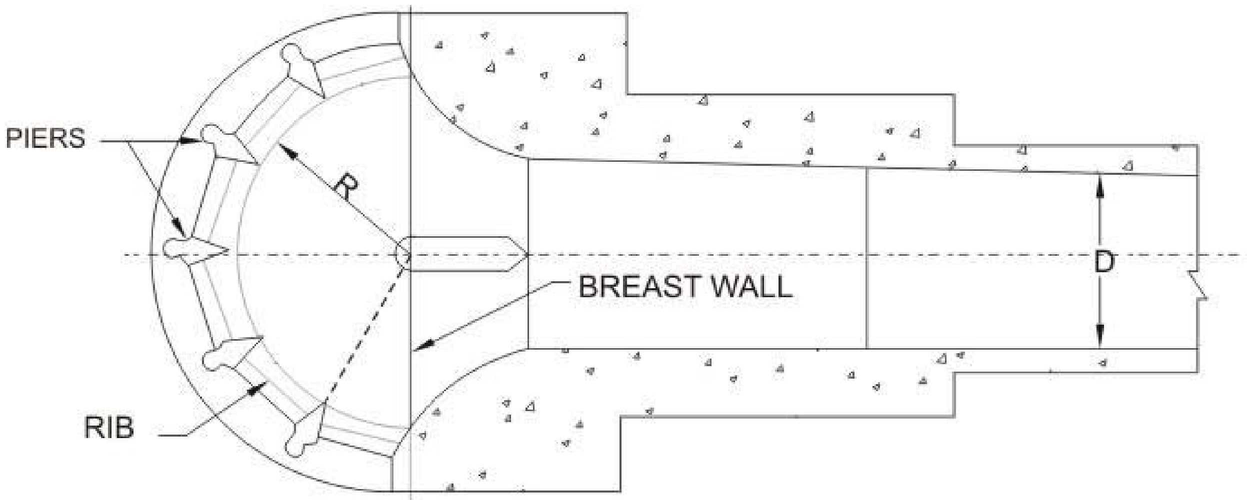


FIGURE 14(c) SEMICIRCULAR TYPE INTAKE STRUCTURE-PLAN

The hydraulic design of the various components of an intake, like the elevation of the centre-line, trash rack structure, bell mouth shape and transition details, etc. may be found from the Bureau of Indian Standards code IS: 11570-1985 “Criteria for hydraulic design of irrigation intake structures”.

Intakes for hydropower

Similar to the irrigation intakes, the hydropower intakes are also of two types:

1. Run-of-the-river type intakes, and
2. Reservoir type intakes.

Both these types of intakes have been discussed in detail in Lesson 5.2. However, it may be mentioned that on the whole, these intakes are quite similar to those constructed for serving irrigation requirements. The Bureau of Indian Standards code IS: 9761-1995 “Hydropower intakes – criteria for hydraulics design” may be referred to finalise details about the hydraulics design of hydropower intakes.

Module 4

Hydraulic Structures for Flow Diversion and Storage

Lesson 10

Gates and Valves for Flow Control

Instructional objectives

On completion of this lesson, the student shall learn:

1. The different types of gates and valves used in water resources engineering
2. Difference between crest gates and deep seated gates
3. Classification of gates
4. Design criteria for important gates
5. Common hoists for gate lifting
6. Different types of valves for flow control

4.10.0 Introduction

Almost every water resources project has a reservoir or diversion work for the control of floods or to store water for irrigation or power generation, domestic or industrial water supply. A spillway with control mechanism is almost invariably provided for release of waters during excess flood inflows. Releases of water may also be carried out by control devices provided in conduits in the body of the dam and tunnels. In order to achieve flow control, a gate or a shutter is provided in which a leaf or a closure member is placed across the waterway from an external position to control the flow of water. Control of flow in closed pipes such as penstocks conveying water for hydropower is also done by valves, which are different from gates in the sense that they come together with the driving equipment, whereas gates require a separate drive or hoisting equipment.

Different types of hydraulic gates and hoists, working on different principles and mechanism are in use for controlled release of water through spillways, sluices, intakes, regulators, ducts, tunnels, etc. Right selection of gates and their hoisting arrangement is very important to ensure safety of the structure and effective control. A designer has to plan a gate and its hoisting arrangement together. Separate planning of gates or hoists, sometimes results in unsatisfactory installation. Though the choice for the gates and hoists depends on several factors, primarily safety, ease in operation as well as maintenance and economy are the governing requirements in the same order. It is essential for the water resources engineer to be aware of the different factors, which would largely affect the choice of gates and hoists and would help in selection of the same. In this lesson, an introduction is provided on different gates, specific purposes for which they may be used, possible locations in which to install, and suitable hoists with which to operate. A brief outline is also provided on the common types valves used to regulate flow in penstocks.

The Bureau of Indian Standards code IS 13623: 1993 “Criteria for choice of gates and hoists” provides the basic classification of gates, which may be done according to the following criteria.

1. Location of the gate with respect to reservoir water surface

2. Head of water over sill of gate
3. Operational requirement
4. Material used in fabrication
5. Mode of operation
6. Shape of gate
7. Discharge through gate
8. Type of flow passage with which connected and its location
9. Location of seal
10. Location of skin plate
11. Closing characteristics
12. Drive to operate

Hoists for raising gates are also classified based on certain characteristics, such as:

1. Drive operating mechanism
2. Mounting

Some of the important terminologies associated with gates are given below, which would help one to understand the operation of gates more closely.

1. **Counter weight**
   A weight used for opposing the dead weight of a gate so as to reduce the hoisting capacity. A counter weight may also be used for making the gate 'Self closing'.

2. **Frame**
   A structural member embedded in the surrounding supporting structure of a gate, which is required to enable the gate to perform the desired function.

3. **Hanger**
   A device meant for suspending or supporting a gate in the open position when disconnected from its hoisting mechanism.

4. **Gate groove or gate slot**
   A groove or slot is a recess provided in the surrounding structure in which the gate moves rests or seats.

5. **Leaf**
   The main body of a gate consisting of skin plate, stiffeners, horizontal girders and end girders.

6. **Lip**
   The lower most segment of a gate which is suitably shaped from hydraulic consideration.

7. **Seal (Bottom, side and top)**
   A seal is a device for preventing the leakage of water around the periphery of a gate. A bottom seal is one that is provided at the bottom of the gate leaf. Side seals are those that are fixed to the vertical ends of gate leaf. A top seal is one that is provided at the top of a gate leaf or gate frame.

8. **Sill**
   This is the top of an embedded structural member on which a gate rests when in closed position.

9. **Guide**
   That portion of a gate frame which restricts the movement of a gate in the direction normal to the water thrust.

10. **Guide rollers**
    Rollers provided on the sides of a gate to restrict its lateral and/or transverse movements.

11. **Guide shoe**
    A device mounted on a gate to restrict its movement in a direction normal to the water thrust.

12. **Horizontal and vertical girders**
    Horizontal girders are the main structural members of a gate, spanning horizontally to transfer the water pressure from the skin plate and vertical stiffeners (if any) to the end girders or end arms of the gate. Vertical girders (also called vertical stiffeners) are the structural members spanning vertically across horizontal girders to support the skin plate.

13. **Hydraulic down-pull**
    The net force acting on a gate in vertically downward direction under hydrodynamic condition.

14. **Hydraulic uplift**
    The net force acting on a gate in vertically upward direction under hydrodynamic condition.

15. **Lift of a gate**
    The maximum vertical travel of a gate above the gate sill.

16. **Lifting beam**
    A beam (with a gripping mechanism) suspended from a gantry crane or a traveling hoist and moves vertically in a gate groove for lifting or lowering a gate or a stop-log.

17. Lifting lugs

Structural members provided on a gate to facilitate handling of the gate during erection, installation or operation.

18. Air vent

A passage of suitable size provided on the downstream of the gate for venting / admitting air during filling / draining a conduit or for delivering a continuous supply of air to the flow of water from a gate.

19. Anchorage

An embedded structural member, transferring load from gate to its surrounding structure.

20. Bearing plate

A metal plate fixed to the surrounding surface of the frame to transfer water pressure to gate frame.

21. Gate Frame or Embedded Part of Embedment

A structural member embedded in the surrounding supporting structure of a gate, which is required to enable the gate to perform the desired function.

22. Thrust Pad or Thrust Block

A structural member provided on a gate leaf to transfer water load from the gate to a bearing plate. It could also be a structural member designed to transfer to the pier or abutment that component of water thrust on a radial gate, which is normal to the direction.

23. Skin plate

A membrane which transfers the water load on a gate to the other components.

24. Track Plate

A structural member on which the wheels of a gate move.

25. Trunnion axis

The axis about which a radial gate rotates.

26. Trunnion Pin

A horizontal axle about which the trunnion hub rotates.

27. Trunnion Tie

A structural tension member connecting two trunnion assemblies of a radial gate to cater to the effect of lateral force (normal to the direction of flow)

28. **Block out**
    A temporary recess/opening left in the surrounding structure of a gate for installing the embedded parts of a gate.

29. **Liner**
    Steel lining generally provided in the gate groove and its vicinity for a medium or high head installation.

30. **Filling Valve**
    A valve fixed over a gate to create balanced water head conditions for gate operation.

4.10.1 Classification of gates based on location of opening with respect to water head

The different types of gates used in water resources projects may be broadly classified as either the Crest or Surface type, which are intended to close over the flowing water and the Deep-seated or Submerged type, which are subjected to submergence of water on both sides during its operation. The different types of gates falling under these categories are as follows:

Crest type gates

1. ***Stop-logs/flash boards***
   A log, plank cut timber, steel or concrete beam fitting into end grooves between walls or piers to close an opening under unbalanced conditions, usually handled or placed one at a time (Figure 1). Modern day stop-logs consist of steel frames that may be inserted into grooves etched into piers and used during repair / maintenance of a regular gate (Figure 2). The stop logs are inserted or lifted through the grooves using special cranes that move over the bridge.

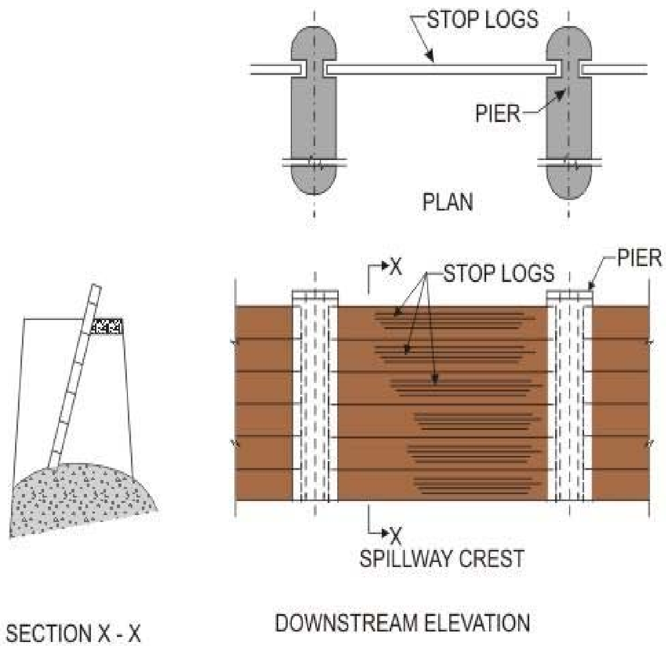


FIGURE - 1. STOPLOGS (NEEDLES)
INSERTED WITHIN GROOVES OF ADJACENT PIERS

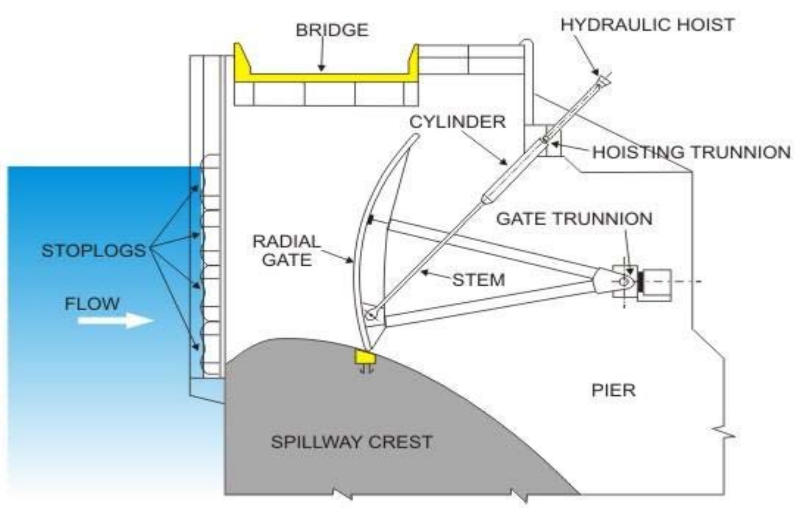


FIGURE - 2. MODERN DAY STOPLOGS SHOWN IN
COMBINATION WITH A RADIAL GATE
AND HYDRAULIC HOIST

2. *Vertical lift gates*

These are gates that moves within a vertical groove incised between two piers (Figure 3). The vertical lift gates used for controlling flow over the crest of a hydraulic structure are usually equipped with wheels, This type of gate is commonly used for barrages but is nowadays rarely used for dam spillways. Instead, the radial gates (discussed next) are used for dams. This is mostly due to the fact that in barrage spillways, the downstream tailwater is usually quite high during floods that may submerge the trunnion of a radial gate.

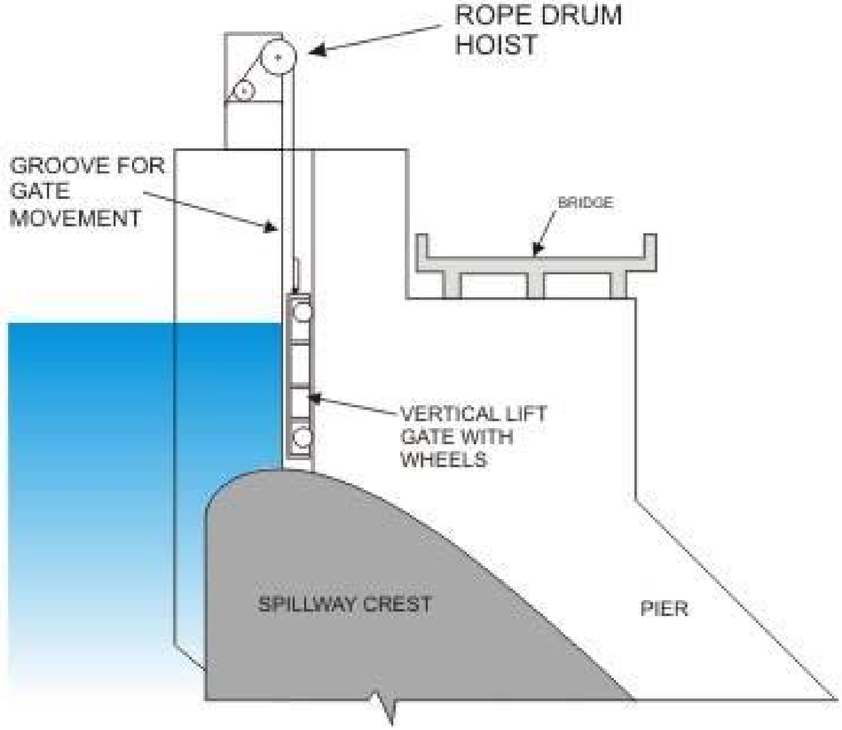


FIGURE 3 : VERTICAL LIFT GATE ARRANGEMENT FOR DAM SPILLWAY

3. *Radial gates*

These are hinged gates, with the leaf (or skin) in the form of a circular arc with the centre of curvature at the hinge or trunnion (Figure 4). The hoisting mechanism shown is that using a cable that is winched up by a motor placed on a bridge situated above the piers. Another example of radial gate may be seen in Figure 2, where a hydraulic hoisting mechanism is shown.

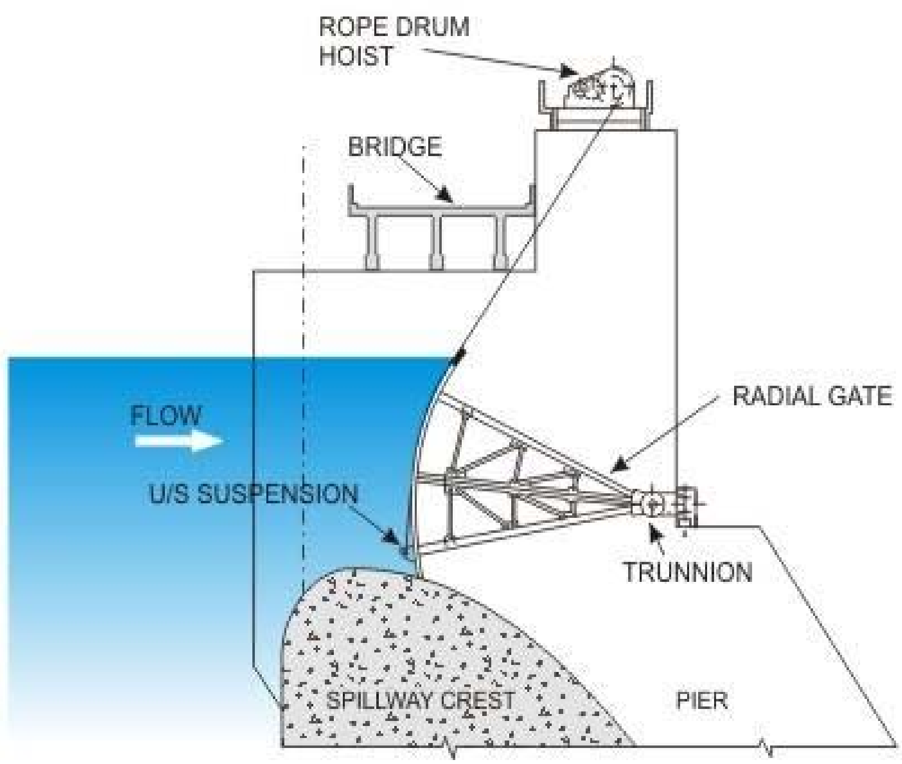


FIGURE 4 : RADIAL GATE SHOWN WITH ROPE DRUM HOISTING MECHANISM

4. ***Ring gates***

A cylindrical drum which moves vertically in an annular hydraulic chamber so as to control the peripheral flow of water from reservoir through a vertical shaft (Figure 5).

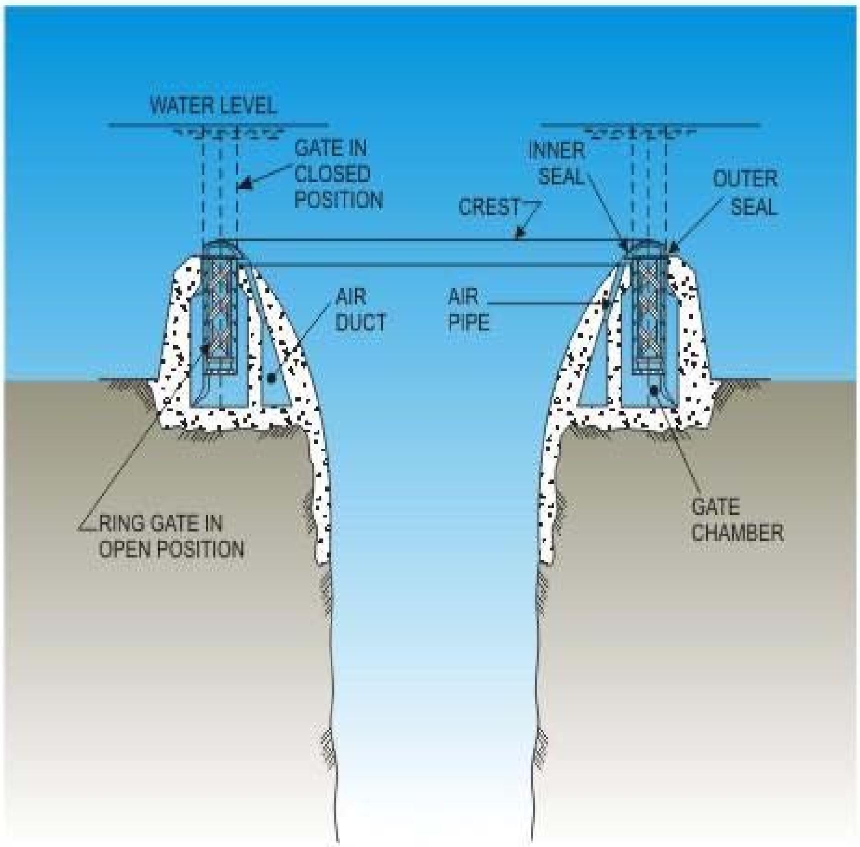


FIGURE 5 : RING GATE ARRANGEMENT
FOR A MORNING GLORY SPILLWAY

5. ***Stoney gate***

A gate which bears on roller trains which are not attached to the gate but in turn move on fixed tracks. The roller train travels only half as far as the gate (Figure 6). This type of gate is not much in use now.

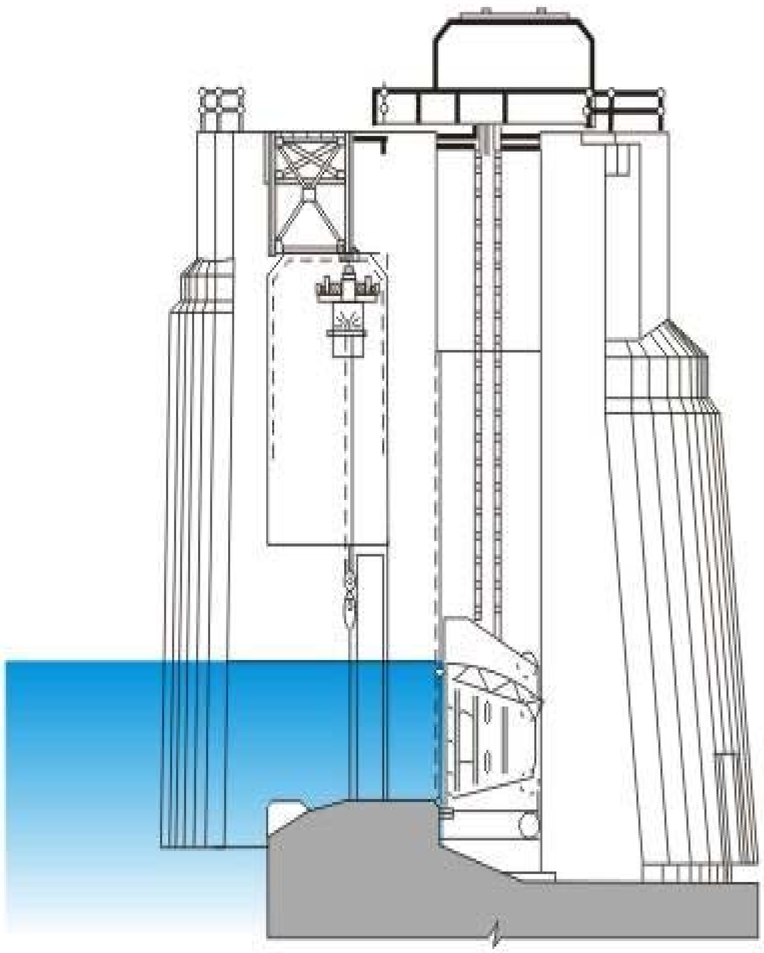


FIGURE 6 : TYPICAL INSTALLATION OF A STONEY GATE

6. Sector gates

A pair of circular arc gates which are hinged on vertical axis in a lock (Figure 7). These gates are used in navigation locks where ships pass from a reservoir with a higher elevation to one with a lower elevation.

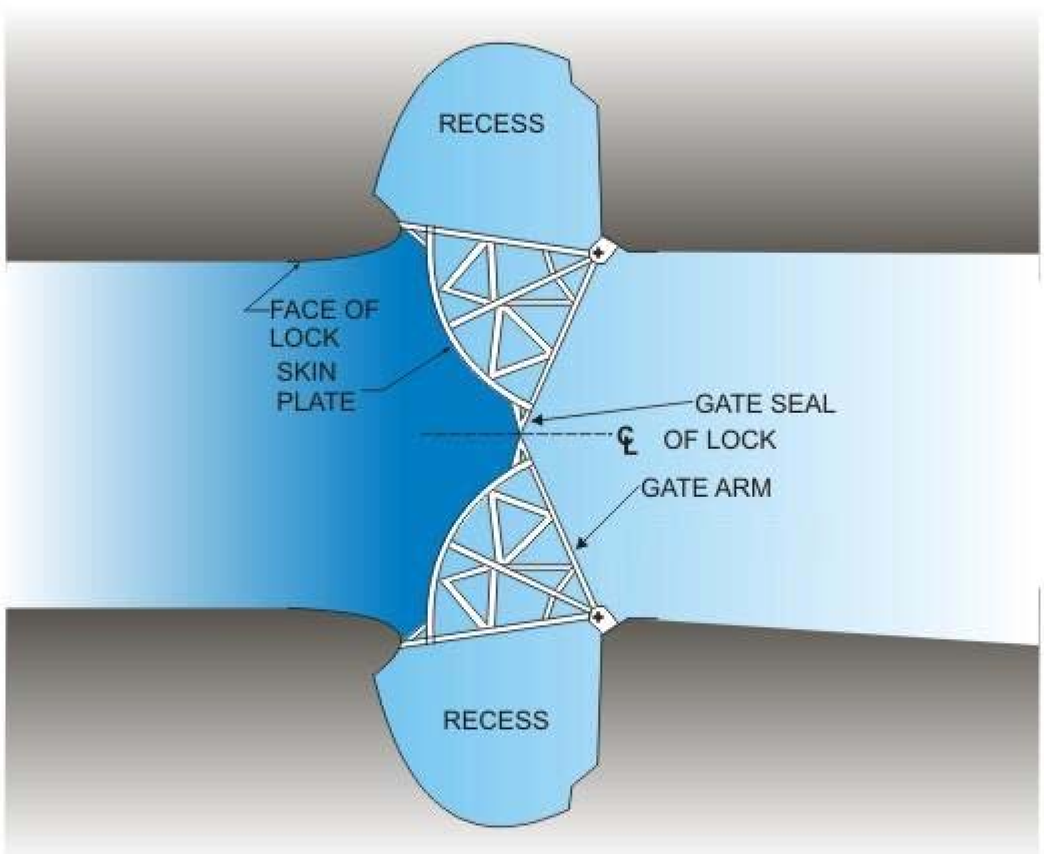


FIGURE 7 : PLAN OF A SECTOR GATE. LEFT SIDE CONTAINS WATER AT HIGHER ELEVATION

7. Inflatable gates

These are gates which has expandable cavities. When inflated either with air or water it expands and forms an obstruction to flow thus effecting control (Figure 8). Though these gates have not been commonly used in our country, it is used quite often in many other countries because of its simplicity in operation – However, they suffer from possible vulnerability from man-made damages.

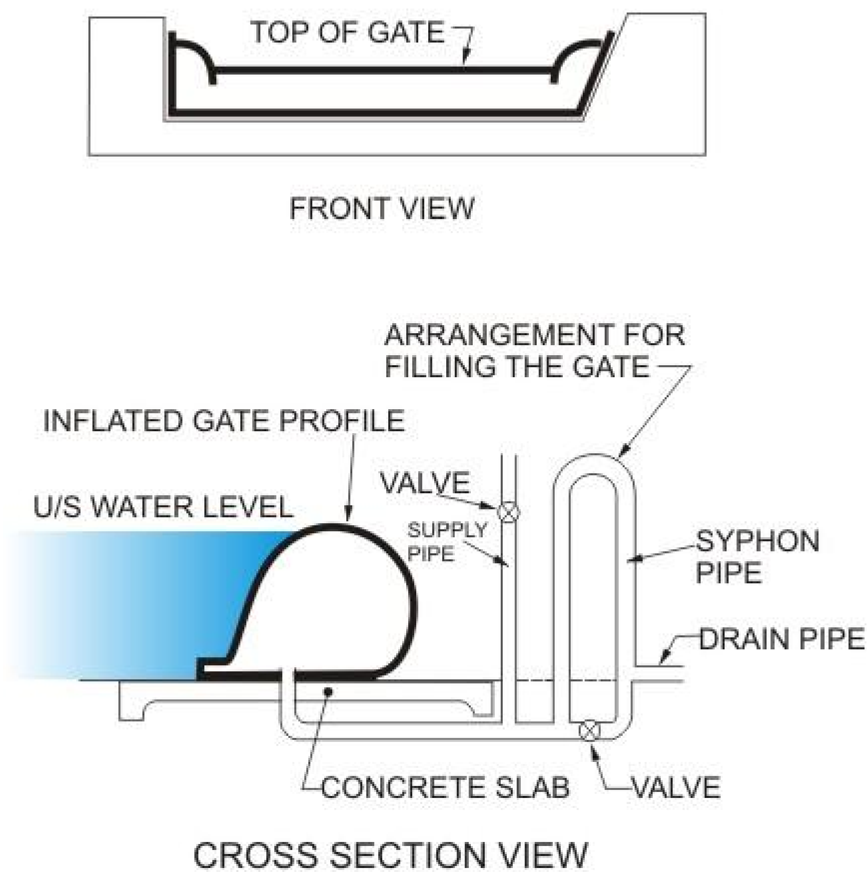


FIGURE 8 : INFLATABLE GATE

8. *Falling shutters*

Low head gates installed on the crest of dams, barrages or weirs (Figure 9) which fall at a predetermined water level. Generally these are fully closed or fully open, that is, fallen flat, which are shown to operate using a hoist. However, in some weirs, falling shutters have been provided earlier that are manually operated. In many of the older weir installations constructed during the pre-independence period were equipped with falling shutters, some of which are still in use today (Figure 10).

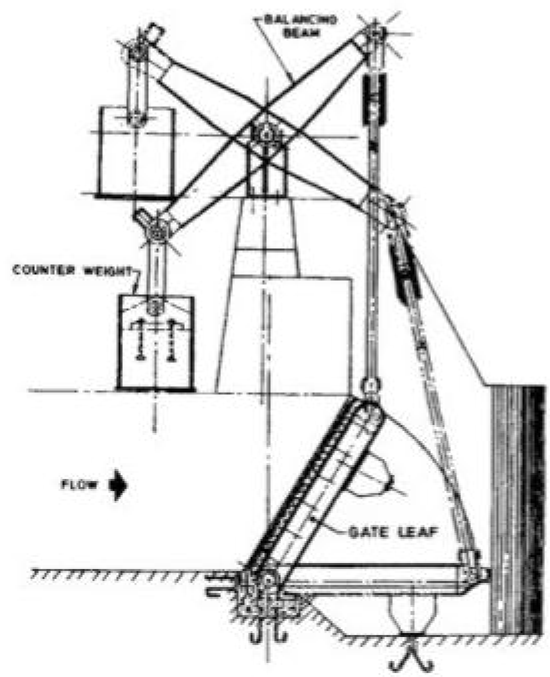


Figure 9. Automatic falling shutter

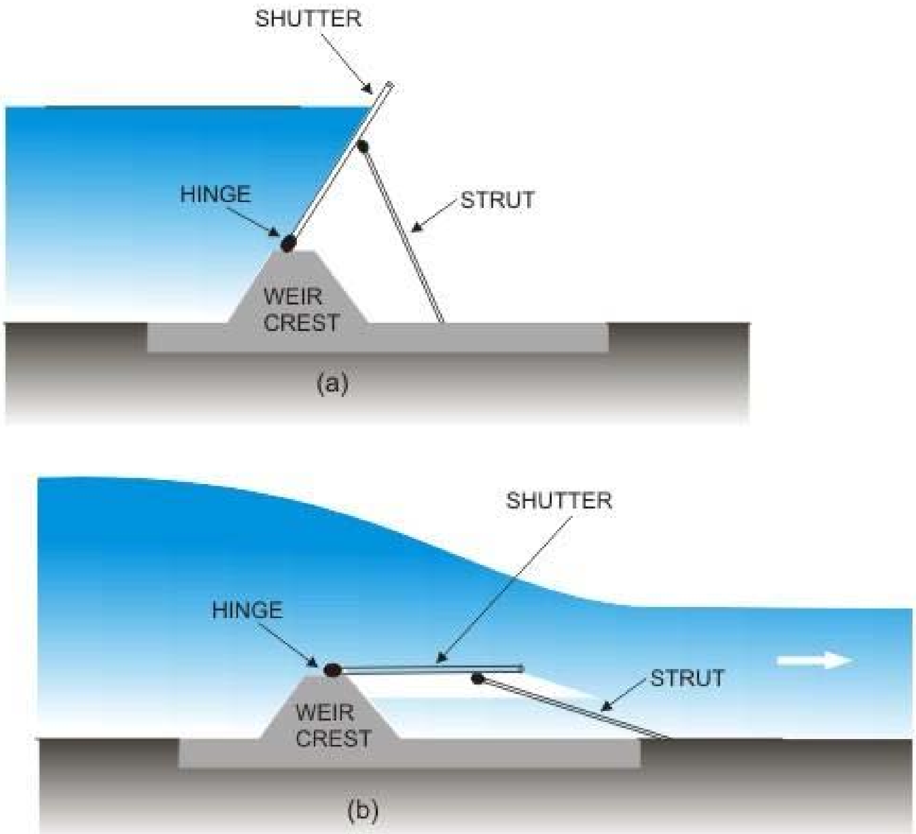


Fig.10: Manually operated falling shutter (a) Closed Position, (b) Open Position

9. ***Float operated gates***

A gate in which the operating mechanism is actuated by a float that is pre-set to a predetermined water level (Figure 11). These may be used as escape in canals or even in dams to release water if it goes above a certain level considered dangerous for the overall safety of the project.

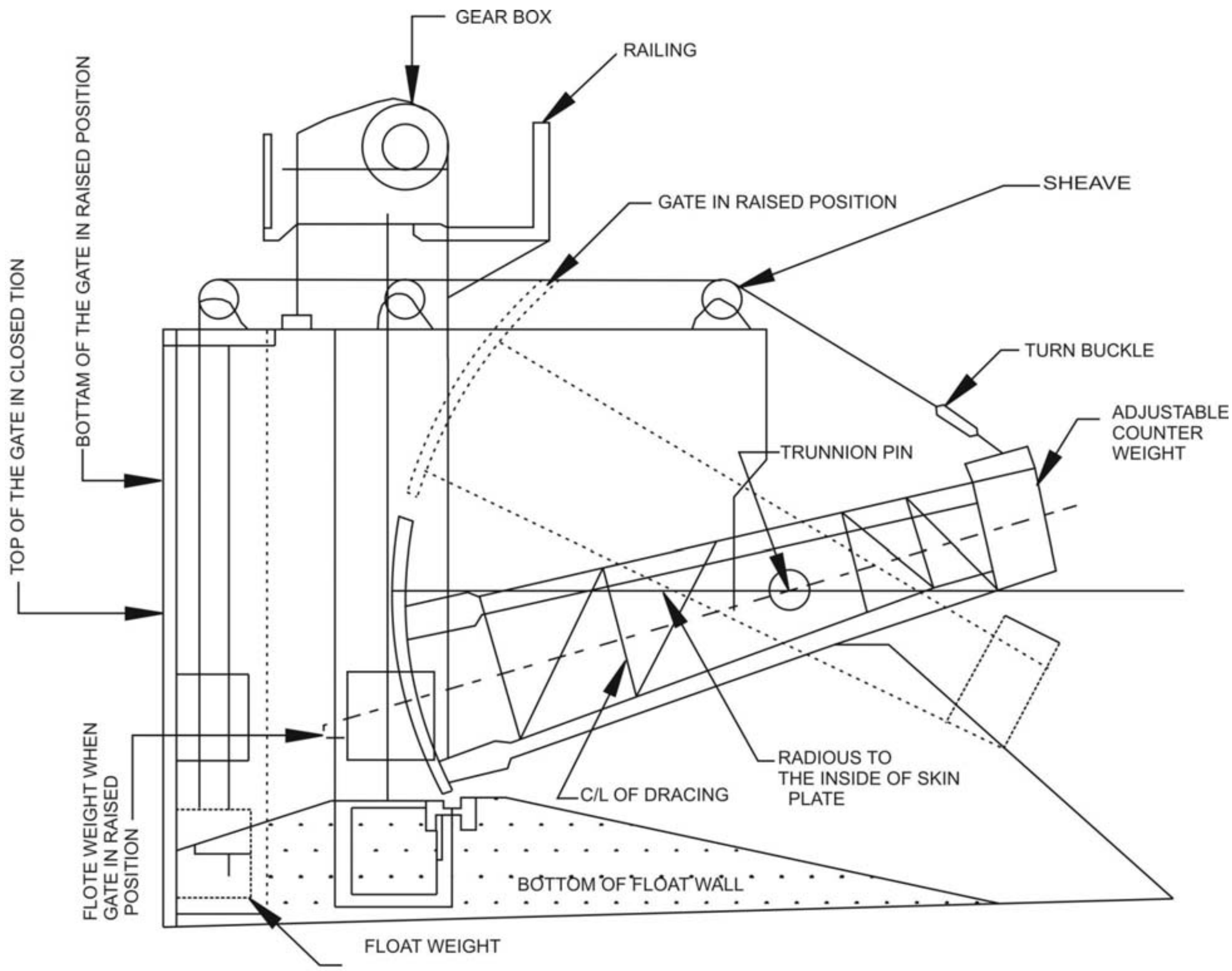


FIGURE 11. AUTOMATIC FLOAT OPERATED RADIAL GATE

10. Two-tier gates

A gate used in two leaves or tiers which can be operated separately, but when fully closed act as one gate. These types of gates are used to reduce the hoist capacity or the lift of the gate (Figure 12). Such a gate has been installed in the canal head regulator of the Farakka barrage.

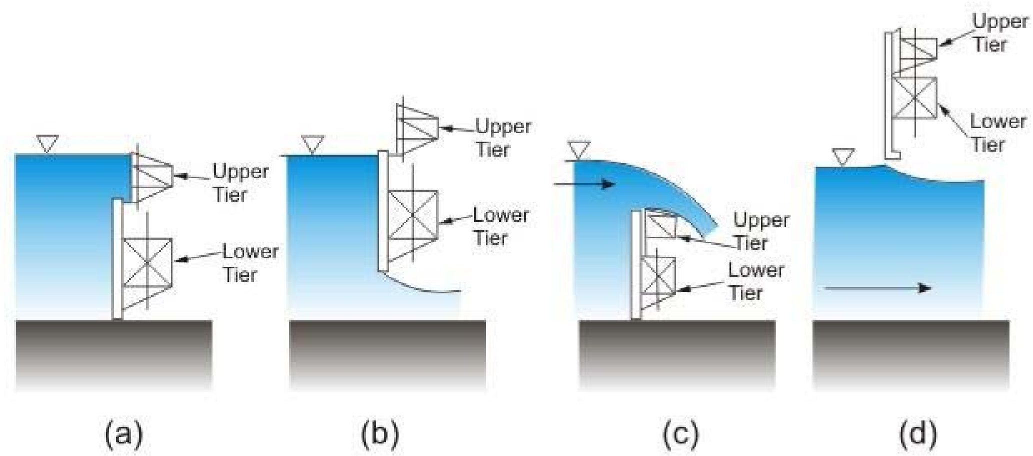


FIGURE 12. Two tier gate at different positions
(a) Closed: (b) Underflow
(c) Overflow; (d) Freeflow

Deep seated gates

1. Vertical gate

Similar to that used for crest type gates (Figure 1), but usually for deep-seated purposes like controlling flow to hydropower intake either the ones with roller wheels (Figure 13), or the sliding-type without any wheels (Figure 14), are used.

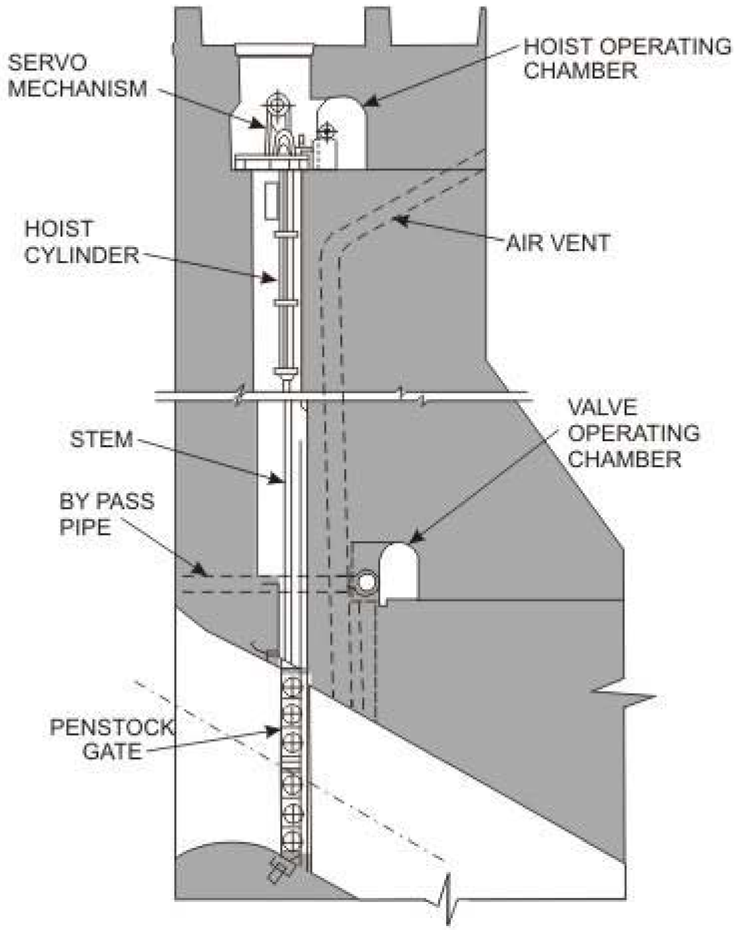


FIGURE 13. TYPICAL ARRANGEMENT OF VERTICAL LIFTGATE WITH WHEELS WITH HYDRAULIC HOIST

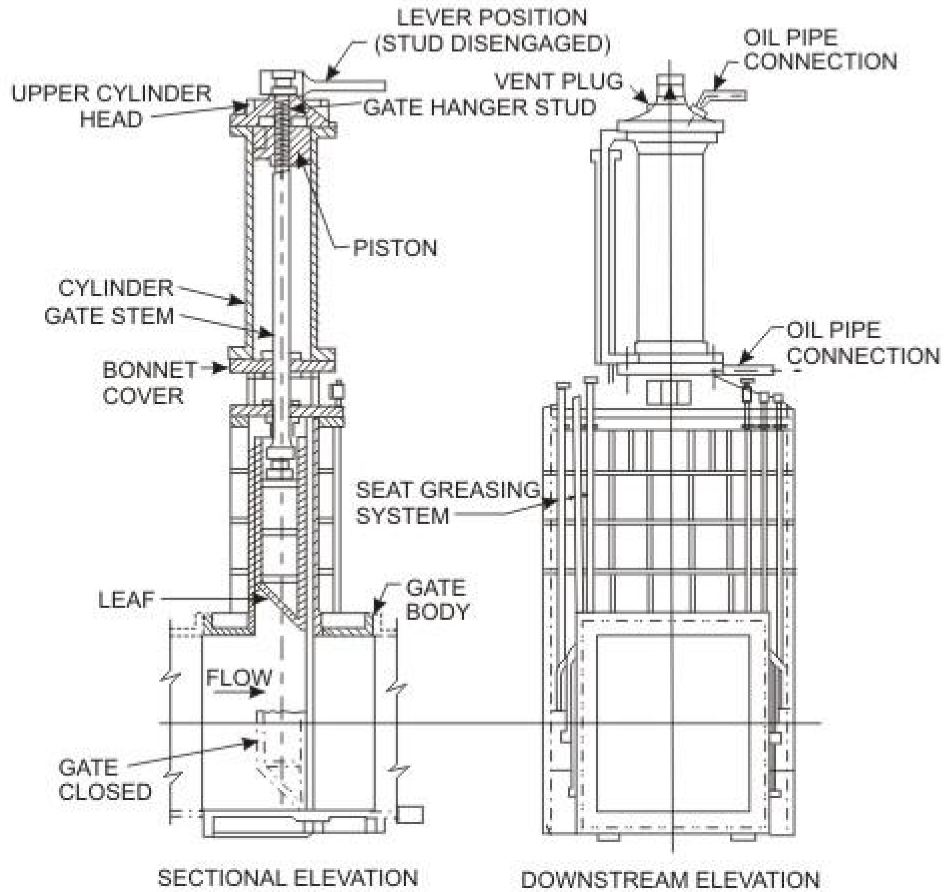


FIGURE 14. SLIDE GATE (VERTICAL LIFT) WITH HYDRAULIC HOIST MECHANISM

According to the Bureau of Indian Standards code IS: 5620 “Recommendations for structural design criteria for low head slide gates”, slide gates may be classified into the following three types depending upon their service conditions.

(i). Bulk head or stop-logs

These are usually located at the upstream end of river outlet conduits or penstocks where in addition some other equipment is used to cut off flow and are subjected to relatively high heads.

(ii). Emergency or guard gates

These are designed to be operated under unbalanced head, that is, with water flowing through the conduit or sluice but are not meant for regulation. These are kept either fully opened or fully closed and are not operated at part gate opening.

(iii). Regulating gates

These are used for regulating flow of water. These are also operated under unbalanced head condition and are designed to be operated at any gate opening.

2. Deep-seated radial gates

These are low level radial outlet gates. These gates have sealing on top apart from on all sides. They are located at sluices in the bottom portion of dam (Figure 15). The hoisting arrangement is usually at the top but could also be provided near the elevation of top seal to reduce hoist stroke.

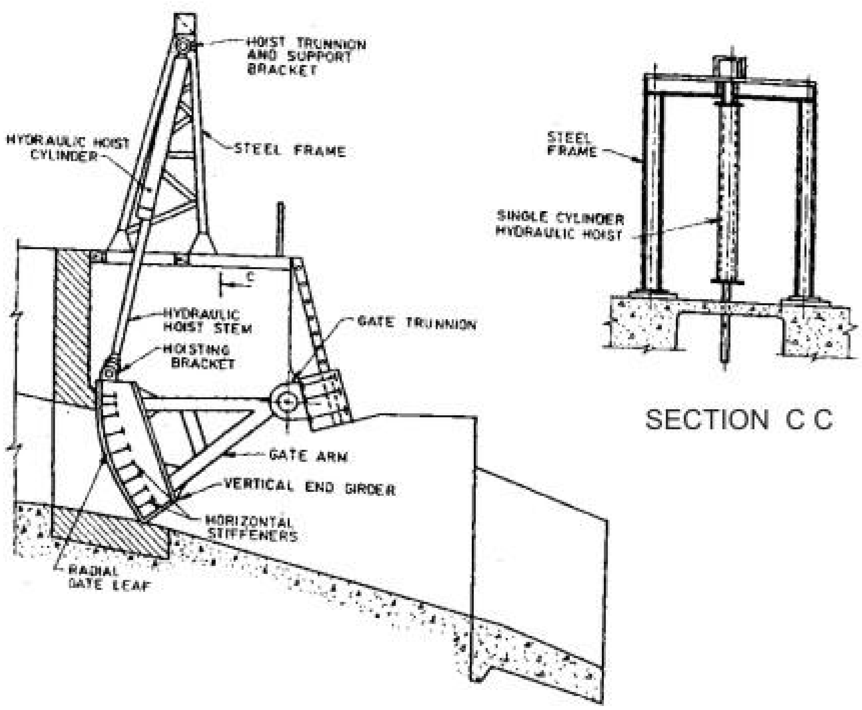


FIGURE 15. DEEP SEATED RADIAL GATE WITH HYDRAULIC HOIST

3. Disc gates

A gate, which is in the form of disc, and rotates about an axis of its plane to control the flow of water.

4. Cylindrical gates

A gate in the form of a hollow cylinder placed in a vertical shaft. These gates are used usually for intake towers, upstream of dams for shutting off the water to penstocks and control values. These may also be used in outlet works (Figure 16).

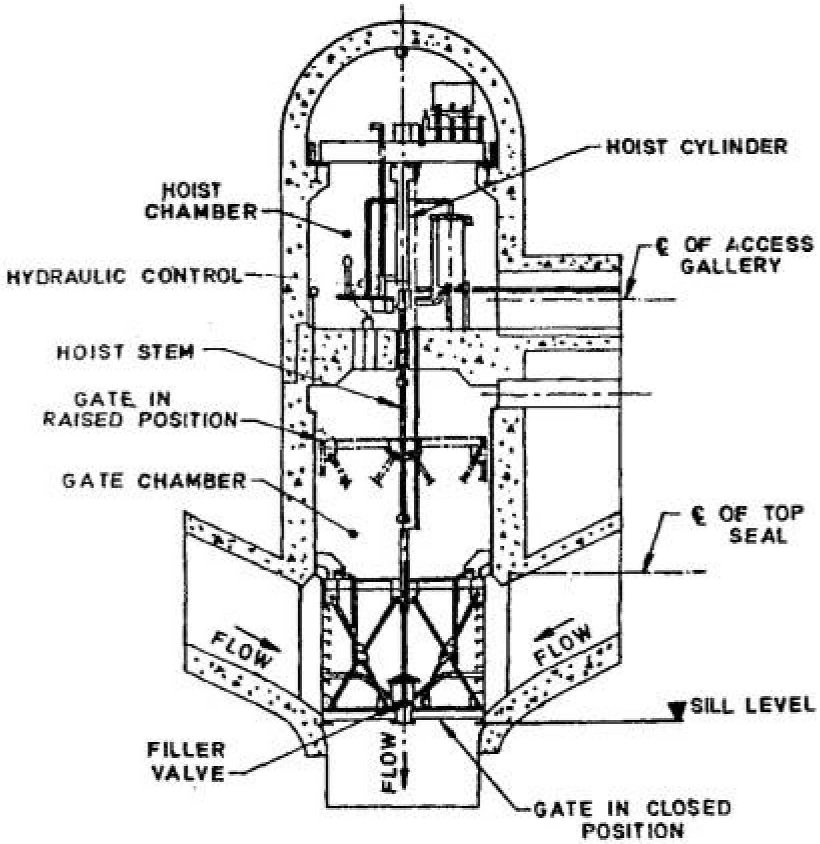


FIGURE 16. CYLINDRICAL GATE WITH HYDRAULIC HOIST

5. Ring follower gates

These are gates with a slide gate with a circular ring (a leaf with a circular hole) extending below the gate leaf. The diameter of the circular hole is equal to the diameter of the conduit. When the gate leaf is raised above the conduit, the circular hole forms an unobstructed passage for the flow of water in the conduit. When the gate is lowered to shutoff the flow, the circular ring fits into a recess below the invert of the conduit. It is used as emergency gate upstream of a regulating or service gate and is operated either in fully closed or fully open position (Figure 17).

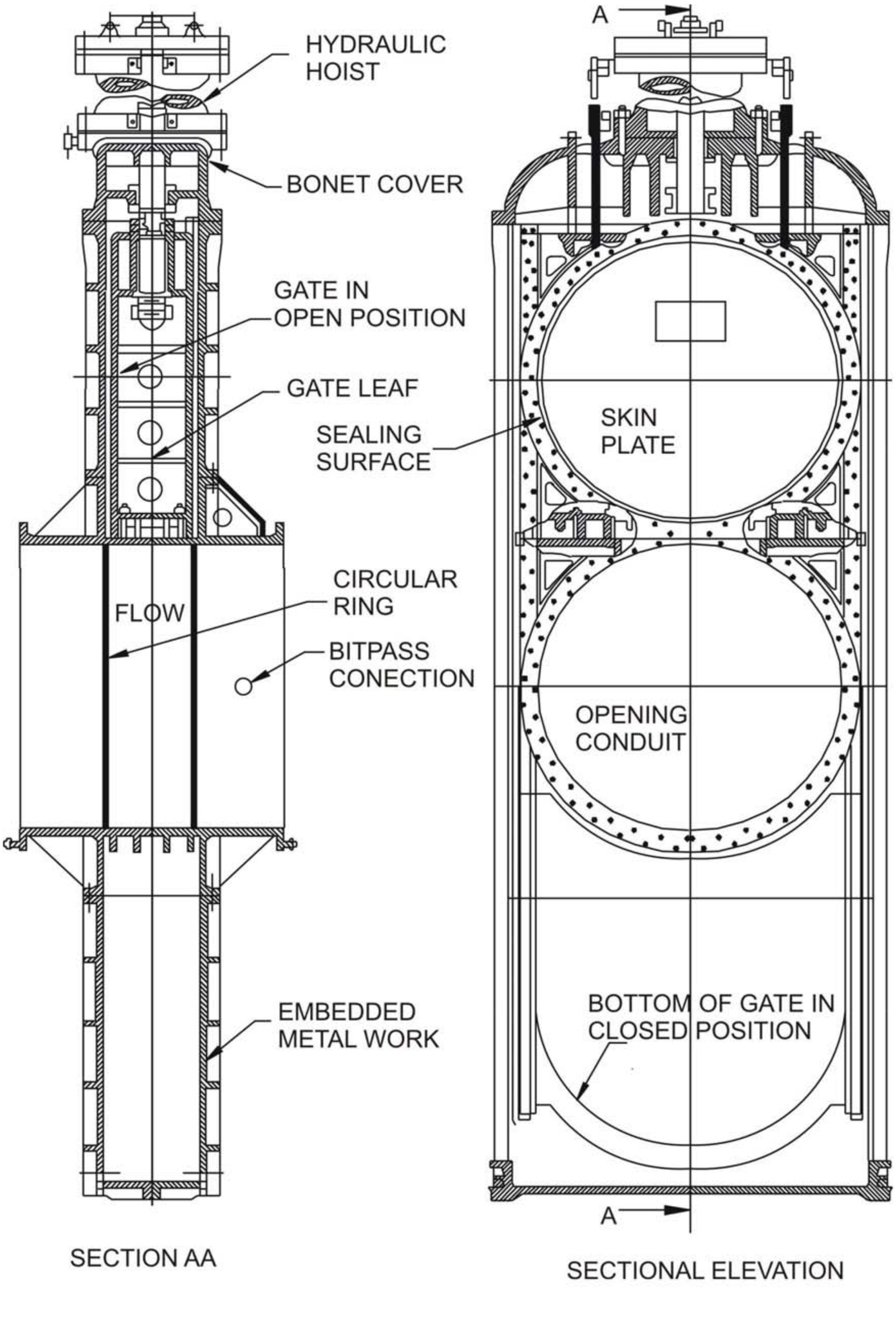


FIGURE17. RING FOLLOWER GATE

6. **Jet flow gates**

A high pressure regulating gate in which the leaf and the housing are so shaped as to make the water issue from the orifice in the form of a jet which skips over the gate slot without touching the downstream edge of the slot (Figure 18). They are adopted when very fine control of discharge is desired.

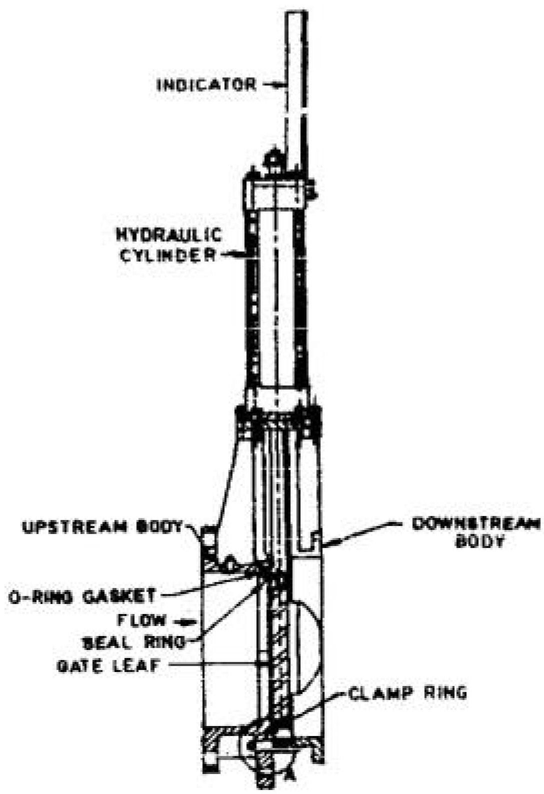


FIGURE 18. JET FLOW GATE

7. **Ring seal gates**

A roller or wheel mounted gate in which the upper portion of the gate leaf forms a bulkhead section to stop the flow of water and the lower portion forms a circular opening of the same size as the conduit so as to produce as unobstructed water passage with the leaf in the open position. Complete closure of the leaf in the lower position is made by extending a movable ring seal actuated hydraulically from the water pressure in the conduit to contact a seat on the leaf. This type of gate is usually used as either service or emergency gates in the penstocks or other conduits (Figure 19).

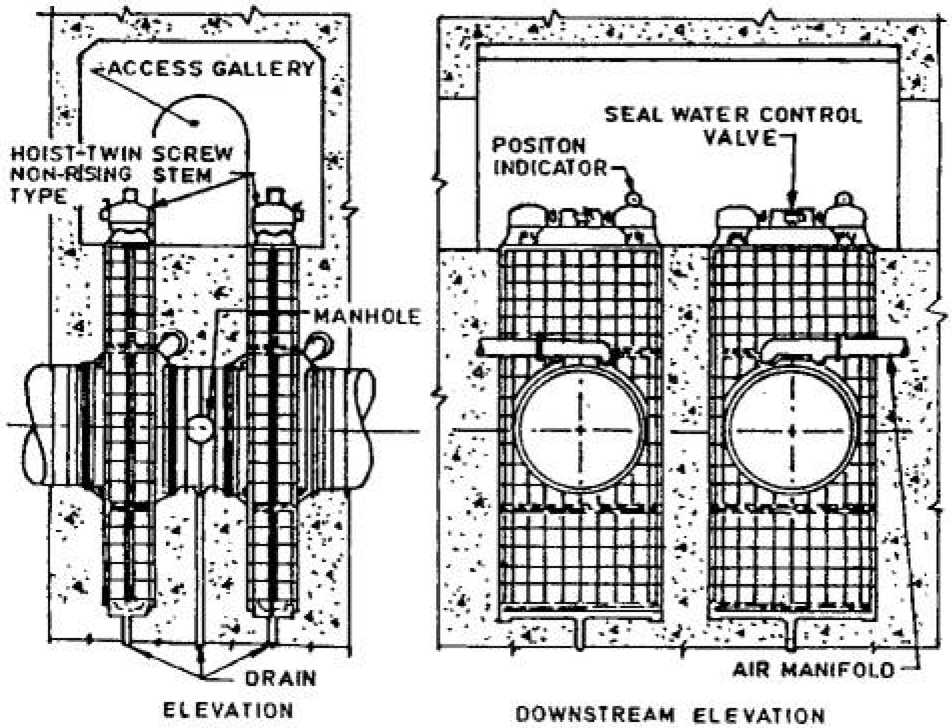


FIGURE 19. RING SEAL GATE (INSTALLATION OF 2 GATES)

4.10.2 Classification of gates based on the type of flow passage with which connected and its location

Gates are a part of most of the openings provided in any water resources project. They may be used to regulate flow through spillways, sluices, intakes, regulators, ducts, tunnels, etc., to name a few. The following list provides classification of gates based on its association with a particular water passage. The gates associated with hydropower have only been briefly described here. They are described in more detail in the next module.

1. **Crest gates**
   A gate mounted on a crest for the purpose of controlling the discharge flowing over the crest of the spillway of a dam or a barrage (Figure 20). As mentioned in Section 4.10.1, it is common to find radial gates to regulate flow over dam crests and vertical lift gates for barrage spillways.

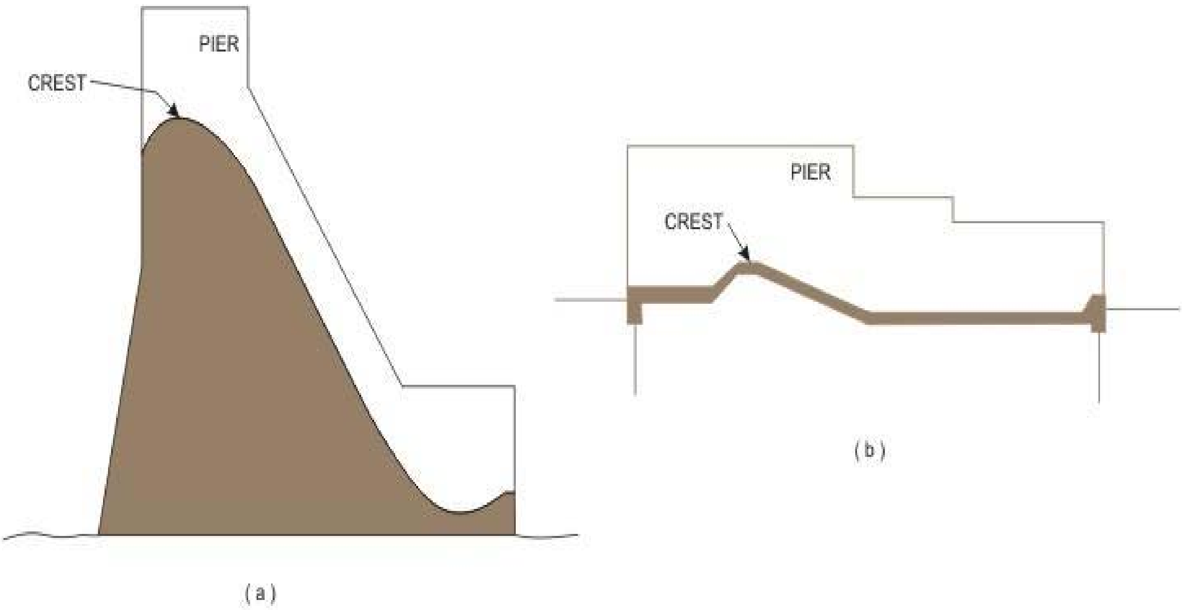


FIGURE 20. SPILLWAY BAYS SHOWING CRESTS OF (a) DAM AND (b) BARRAGE SHOWING CRESTS

2. **Sluice gates**
   These are gates which controls or regulates flow through an opening or sluice in the body of the dam where the upstream water level is above the top of opening as shown for gates at the entry to the penstock of a hydropower intake in Figure 21.

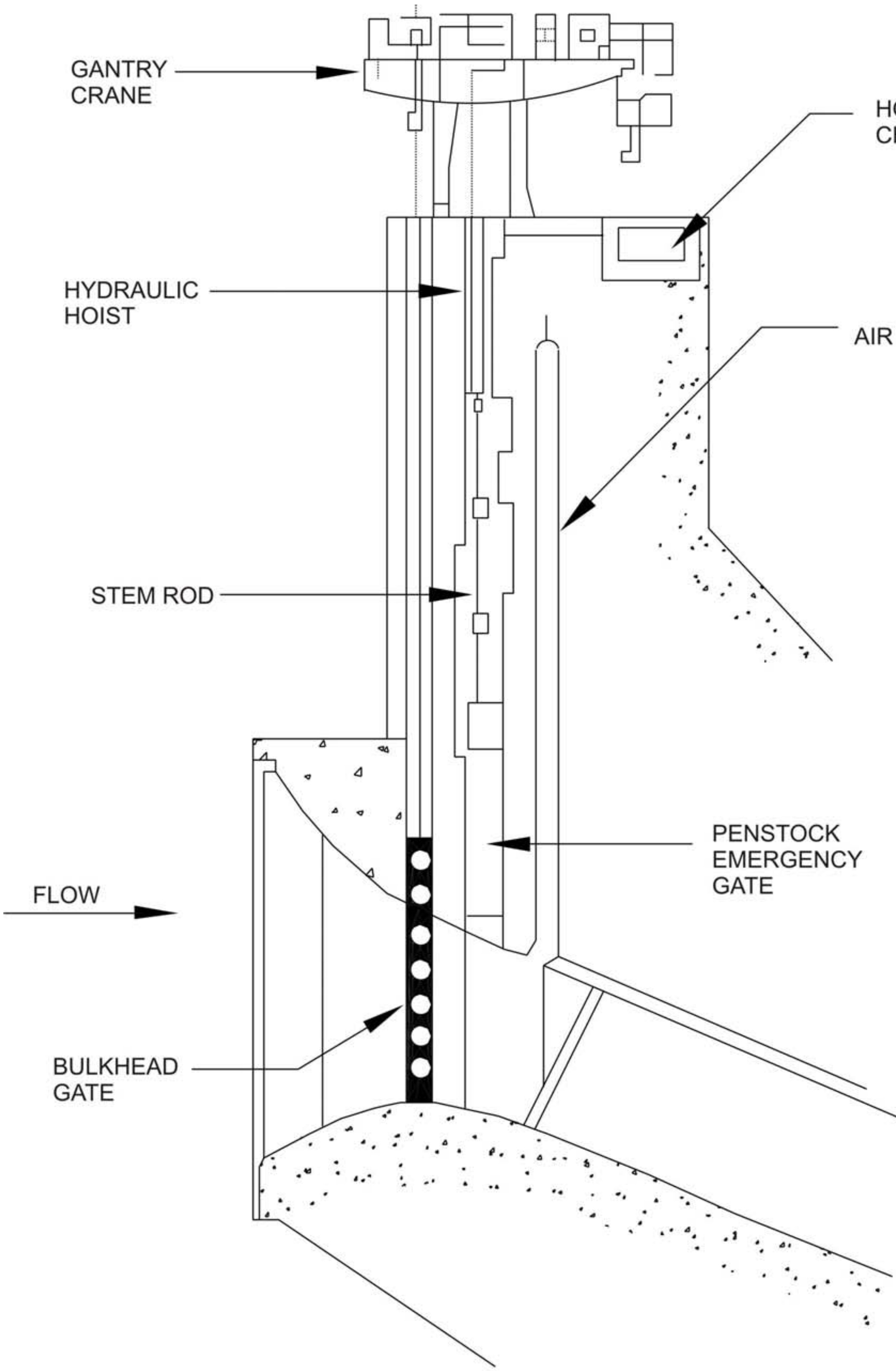


FIGURE 21. PENSTOCK EMERGENCY GATE

3. **Depletion sluice gates**
A gate located at lowest level in the body of the dam to deplete the reservoir in the event of distress. It may be either wheel mounted type or sliding type.

4. **Construction sluice gates**
This gate is meant for closing a construction sluice which is normally plugged after construction.

5. **Diversion tunnel gates**
This gate is meant for making diversion tunnel dry, when it has to be plugged after construction (Figure 22). Service gates are lowered for plugging the diversion tunnel and emergency gates are provided to take care of any eventuality resulting from malfunctioning of the service gates. Usually, such gates are meant for one time operation while plugging the tunnel.

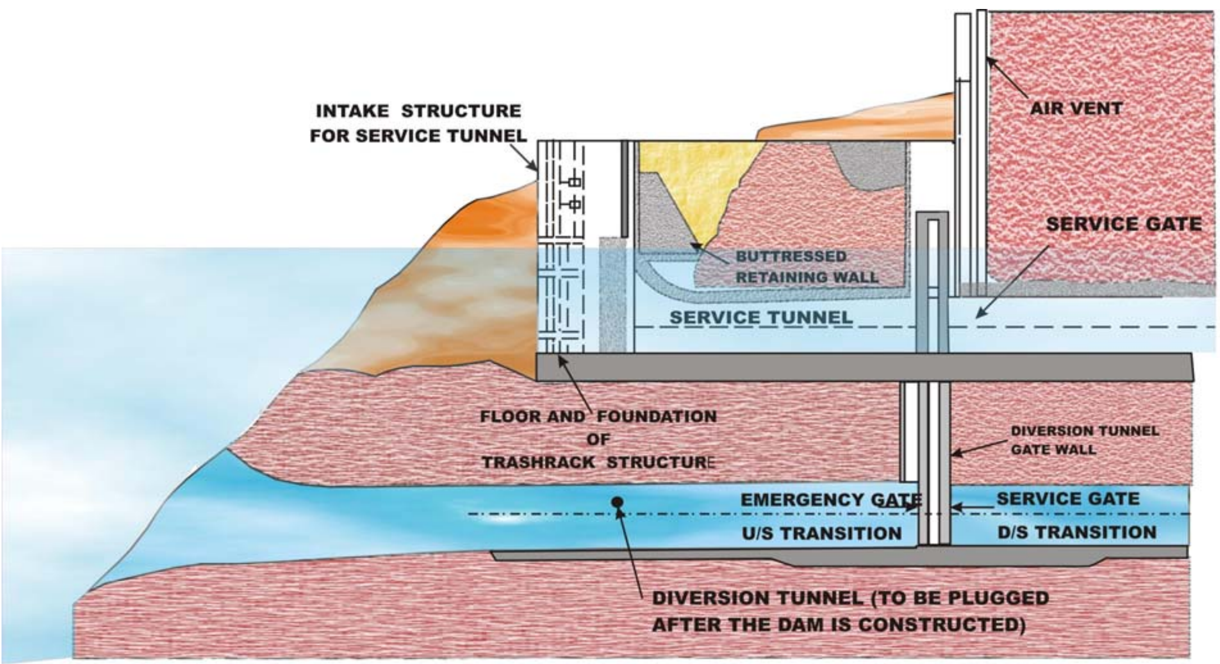


FIGURE 22. DIVERSION ON TUNNEL AND GATE , WHICH CONTROL, FLOW DURING DIVERSION

6. **Head regulator and Cross regulator gates**
The Head regulator gates are used for regulating water from reservoir to main canal. These are generally wheel mounted vertical lift gates. The Cross regulator gates are used in an irrigation channel for the purpose of raising the water level. Usually, vertical lift gates are commonly used, but radial gates are also being adopted.

7. **Desilting chamber gates / Silt flushing gates**
   These gates are located at the exit of desilting chamber of a hydroelectric plant to flush out accumulated silt.

8. **Head race tunnel gates**
   A gate installed at the entrance of head race tunnel of hydroelectric project. It is generally a wheel mounted gate.

9. **Surge shaft gates**
   Surge shaft gate is used for inspection of tunnel / penstock and is located in the vicinity of surge shaft and tunnel junctions.

10. **Penstock gates / Intake gates**
    A gate provided at the upstream end of the penstock.

11. **Draft tube / Tail race gates**
    A bulkhead gate used to permit dewatering of the draft tubes for inspection and repair of turbine parts and draft tubes.

12. **Navigation lock gates**
    These are gates provided on navigation locks. Commonly used in India is the Mitre gate, which is a lock gate comprising of two hinged symmetrical leaves which meet at the centre of the lock channel when in the closed position and fit into recesses in the side walls of the channel when open (Figure 23).

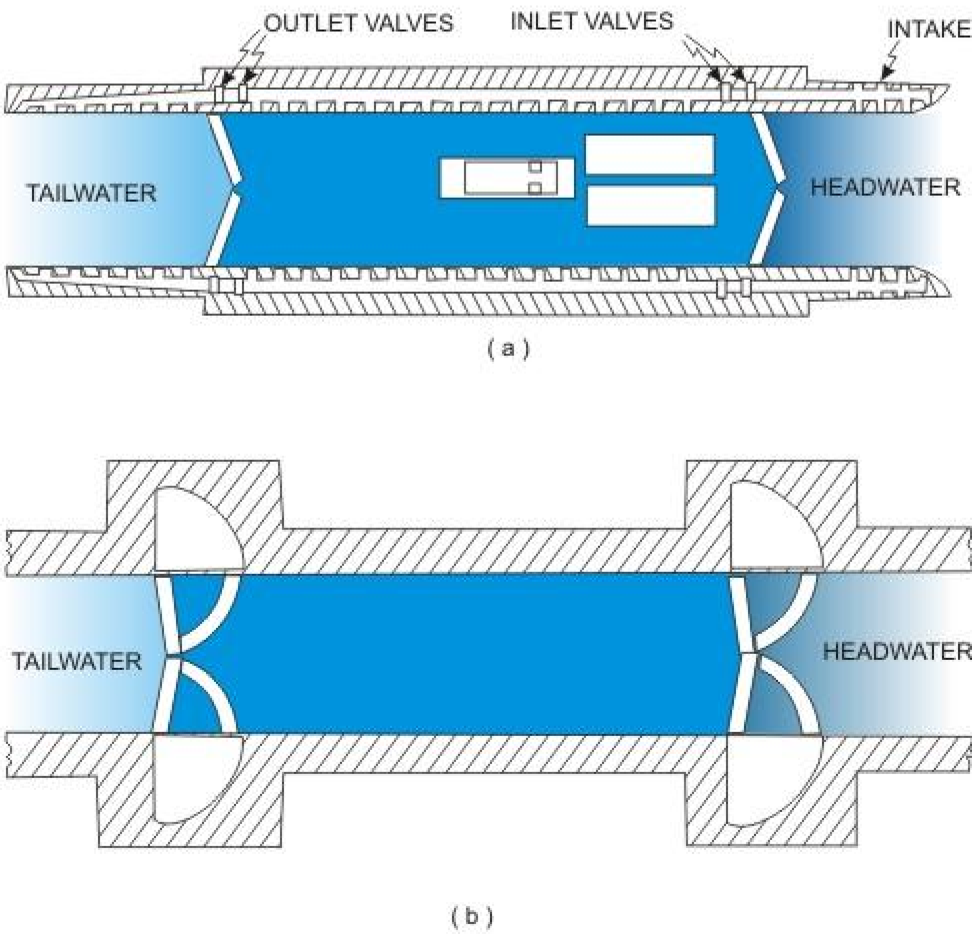


FIGURE 23. PLAN OF NAVIGATION LOCK

13. Balancing gates

A gate used for the purpose of balancing water levels on either side.

4.10.3 Classification of gates based on other criteria

The Bureau of Indian Standards code IS: 13623-1993 "Criteria for choice of gates and hoists" has recommended certain selection criteria for gates under specific conditions, since this has a great impact on the safety of the structure and effective control of water flow. Further, a designer has to plan a gate and its hoisting arrangement together. Separate planning may sometimes lead to unsatisfactory installation. Though the choice for the gates and hoists depends upon several factors, primarily safety, ease in operation as well as maintenance and economy are the governing requirements. Some of the salient points, taken from IS: 13623 – 1993 are presented below.

Classification based on head over Sill

1. Low head gate: head less than 15 m
2. Medium head gate: head between 15 m and 30 m
3. High head gate: head more than 30 m

Classification based on operational requirements

1. Service gates (main gate): To be used for regulation and routine operation such as main gate for regulation of flow through spillway sluices, outlets, etc.
2. Emergency closure gates: To close the opening in flowing water condition in case of emergency such as emergency penstock gate.
3. Maintenance gate: Bulkhead gate, emergency gate, stop-logs, which are used for maintenance of service gates.
4. Construction gates: Required to shut off the opening during construction or to finally close the opening after construction such as construction sluice gates, diversion tunnel gates, etc.

Classification based on material used in fabrication

1. Steel gates
2. Wooden gates
3. Reinforced concrete gates
4. Aluminium gates
5. Fabric (plastic) gates/Rubber gate
6. Cast iron gates.

Classification based on mode of operation

1. Regulating gates: Operated under partial openings. Generally the main regulating gates are the service gates.
2. Non-regulating gate: Gates not suitable as well as not intended for operation under partial gate openings.

Classification based on shape

1. Hinged gates: Such as radial gates, Sector gates, hinged leaf gates, falling shutters.
2. Translatory gates: Rolling gates such as fixed wheel gate, Stoney gate, slide type gate, etc.

Classification based on discharge through the gate

1. Free discharging gate: Flow past the gate is in open air that is the tail water level is below the sill level of the gate and there is no submergence.

2. Gates for submerged flow: Where the tail water level is above the sill level of the gate such as deep radial gate.

Classification based on location of seal

1. Upstream seal gate,
2. Downstream seal gate, and
3. Seals both upstream and downstream.

Classification based on the location of skin plate

1. Gates with upstream skin plate, and
2. Gates with downstream skin plate.

Classification based on closing characteristics

1. Self closing gates
2. Gate requiring positive thrust for closure

Classification based on drive to operate

1. Manually operated gates
2. Electrically operated gates
3. Semi automatic gates
4. Automatic gates, like:
   i) Float operated gate,
   ii) Water powered automatic gates
   iii) Solar powered gate
   iv) Computer controlled gates

4.10.4 Design of important gates

The important types of gates used for water resources projects are the following:

1. Fixed wheel type vertical lift gates
2. Radial gates
3. Sliding gates

The following paragraphs mention the salient features of these gates, the detailed design of which are available in the respective Bureau of Indian Standards codes as mentioned.

Fixed wheel type vertical lift gates

The fixed-wheel vertical lift gates comprise of, in general, a structural steel frame consisting of end vertical girders with properly spaced horizontal girders between them. The spacing depends on the design water pressure and on dimensions of the gate. The frame is held a piece by secure welding or riveting. Skin plate protects the structural framework from damage due to ice and heavy debris, minimizes downpull, reduces corrosion and facilitates maintenance. However, in some cases as in the case of fixed wheel gates moving on track provided on the face of the dam, skin plate is provided on the downstream side. In exceptional cases, skin plate is provided on both downstream side and upstream sides, if the down stream water is above sill. In such cases the gates maybe fully or partially buoyant. In case of fully buoyant gates, buoyancy shall be taken into account in determining the net balance of vertical forces and addition of ballast may be necessary to ensure lowering without difficulty. This problem is absent in the case of flooded gates but greater care against corrosion becomes necessary. The wheels are mounted on the end girders. The bottom of gate should be so shaped that satisfactory performance and freedom from harmful vibrations are attained under all conditions of operation apart from minimizing downpull. A typical arrangement of various components of gate is shown in Figure 24. Detailed design of this type of gates has been published by the Bureau of Indian Standards code IS: 4622-2003 “Recommendations for structural design of fixed wheel gates”.

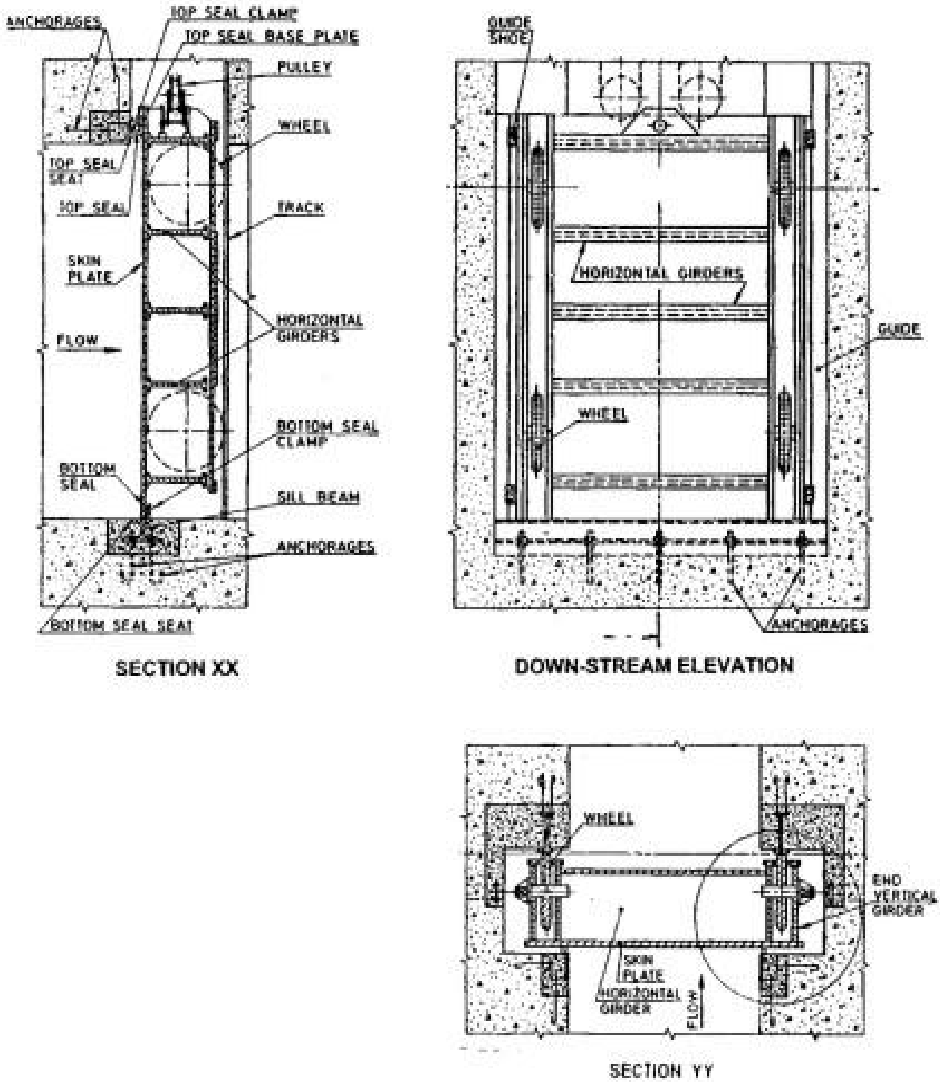


FIGURE 24. VERTICAL LIFT FIXED WHEEL GATE

Radial gates

Normally, the radial gate has an upstream skin plate bent to an arc with convex surface of the arc on the upstream" side (Figure 25 and 26). The centre of the arc is at the centre of the trunnion pins, about which the gate rotates. The skin plate is supported by suitably spaced stiffeners either horizontal or vertical or both. If horizontal stiffeners are used, these are supported by suitably spaced vertical diaphragms which are connected together by horizontal girders transferring the load to the two end vertical diaphragms. The end beams are supported by radial arms, emanating from the trunnion hubs located at the axis of the skin plate cylinder. If vertical stiffeners are used, these are supported by suitably spaced horizontal girders which are supported by radial arms. The arms transmit the water load to the trunnion/yoke girder. Suitable seals are provided along the curved ends of the gate and along the bottom. If used as a regulating gate in tunnels or conduits, a horizontal seal fixed to the civil structure, seals with the top horizontal edge of the gate, in the closed position. The upstream face of the gate rubs against the top seal as the gate is raised or lowered. Guide rollers are also provided to limit the sway of the gate during raising or lowering.

The trunnion anchorage comprises essentially of a trunnion yoke girder, held to the concrete of the spillway piers or the abutments by anchor rods or plate sections designed to resist the total water thrust on the gate. The trunnion or yoke girder is usually a built-up section to which the anchors are fixed.

The thrust may be distributed in the concrete either as bond stresses along the length of the anchors (Figure 27) or as a bearing stress through the medium of an embedded anchor girder at the up stream end of the anchors. In the latter case the anchors are insulated from the surrounding concrete.

Alternatively, anchorages of radial gates could also comprise pre-stressed anchorage arrangement. This system is especially advantageous in the case of large sized gates where very high loads are required to be transferred to the piers and the system of anchorages mentioned above is cumbersome and tedious. In this case pre-stressed anchorages post tensioned steel cables or rods are used which when subjected to water thrust will release pressure from concrete due to higher tensile stresses carried by anchorages.

The Bureau of Indian Standards code IS: 4623-2003 “Recommendations for structural design of radial gates” may be referred to for further details on radial gates design.

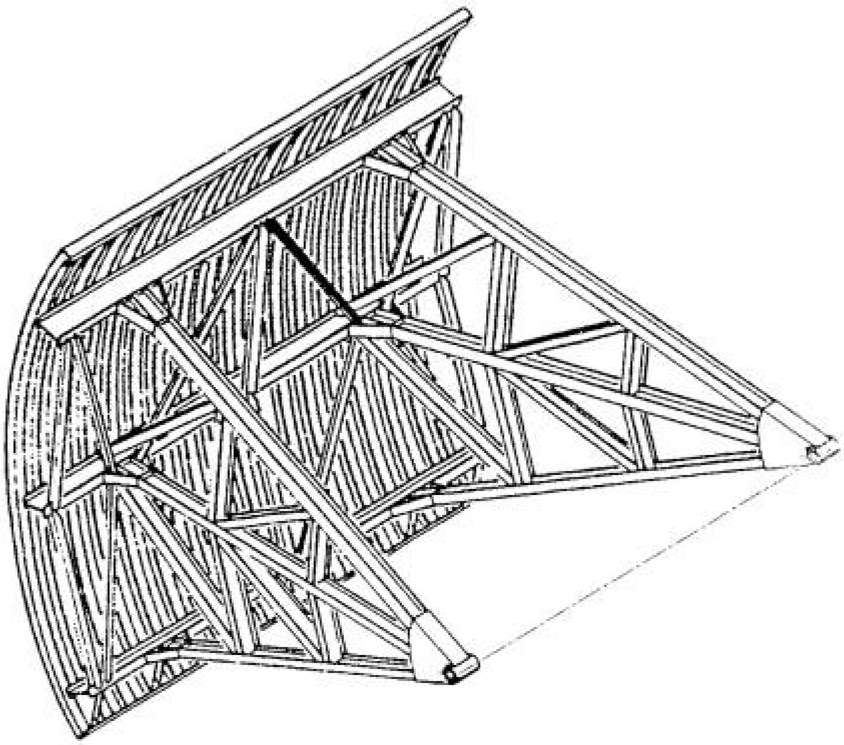


FIGURE 25. DOWNSTREAM VIEW OF A TYPICAL TAINTER GATE

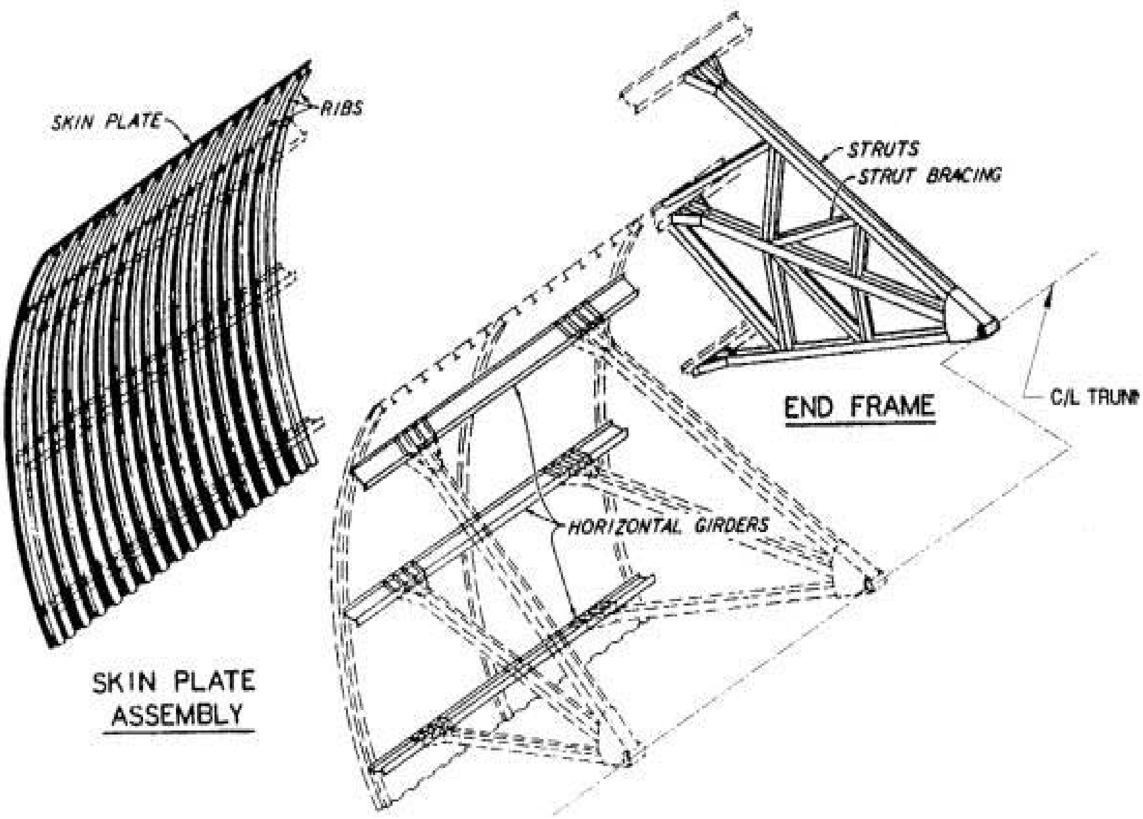


FIGURE 26. PRIMARY TAINTER GATE COMPONENTS

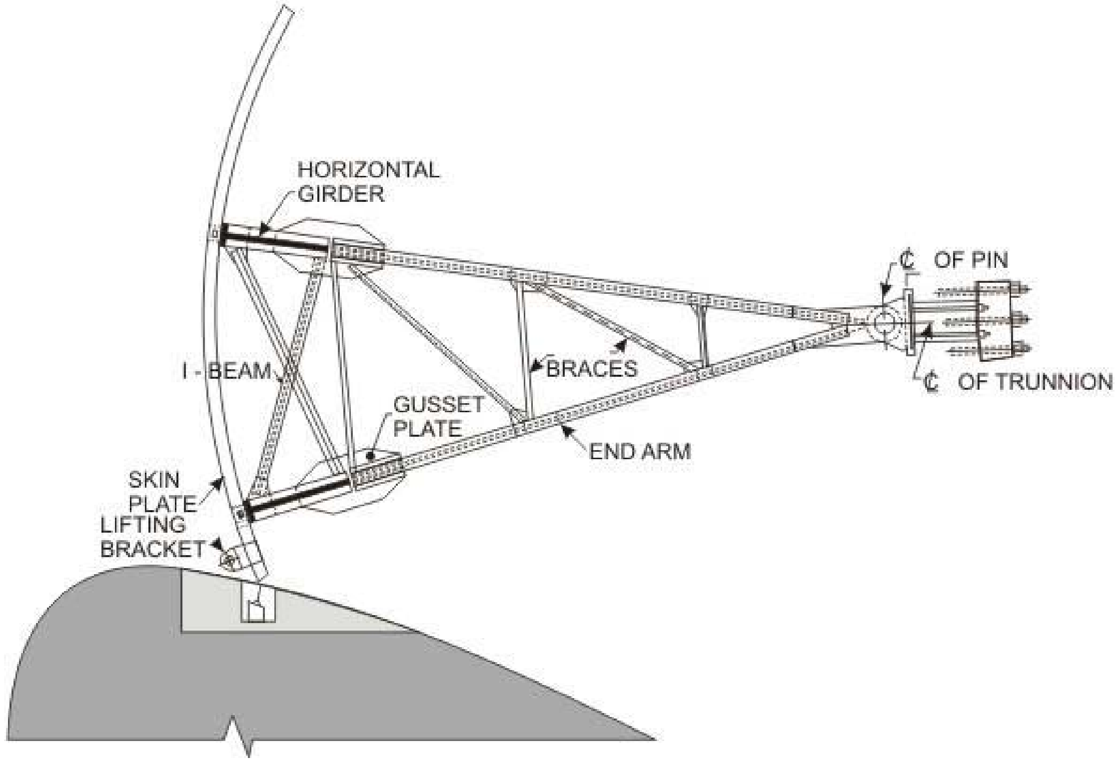


FIGURE 27. SIDE VIEW OF A RADIAL GATE

Sliding gates

Slide gates, as the name implies, are the gates in which the operating member (that is, gate leaf) slides on the sealing surfaces provided on the frame. In most cases, the sealing surfaces are also the load-bearing surface. Slide gates may be with or without top seal depending whether these are used in a close conduit or as crest gate. A typical installation of a slide gate is shown in Figure 14. These consist of a gate leaf and embedded parts. These embedded parts serve the following purposes:

a) Transmit water load on the gate leaf to the supporting concrete (structure),
b) Guide the gate leaf during operation, and
c) Provide sealing surface.

The following Bureau of Indian Standards codes may be referred to while designing slide gates:

IS: 5620-1985 “Recommendations for structural design criteria for low head slide gates”.

IS: 9349-1986 “Recommendations for structural design of medium and high head slide gates”.

4.10.5 Commonly used hoists for gate operation

The mechanical arrangements used for operating the gates are called Hoists, which are classified as follows:

- **Mechanical hoist:**
    1. Rope-drum type like winches, chain-pulley block, monorail crane, gantry crane, etc.
    2. Screw operated type
    3. Chain and sprocket type

- **Hydraulics hoist**

The Bureau of Indian Standards code IS 6938 – 1989 "Design of rope drum and chain hoists for hydraulic gates – code of practice" lays down the guiding principles for design of rope drum and chain hoists. The general principle of a rope drum and chain hoist for vertical lift gates is shown in Figure 28. The rope drum arrangement for radial gate is shown in Figure 29.

The Bureau of Indian Standards code IS 10210 – 1993 "Criteria for design pf hydraulics hoists for gates" provides guidelines for typical hydraulic hoists for gates. A typical arrangement for hydraulic hoist for radial gates is shown in Figure 30 showing the position of the hoist and the gate in open and closed positions.

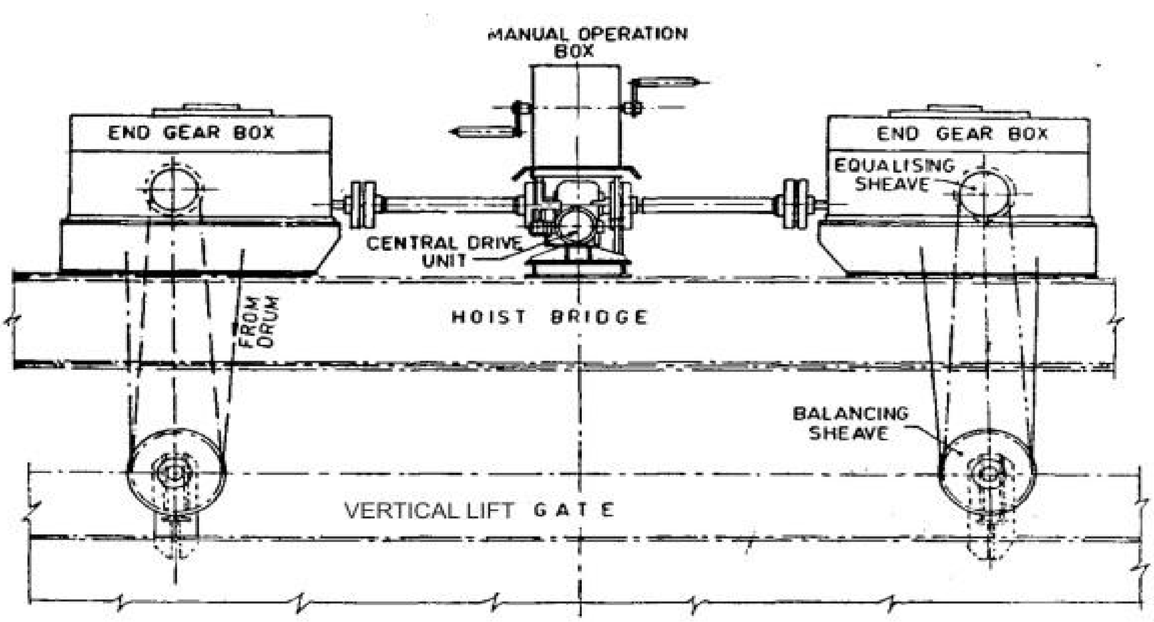


FIGURE 28. ROPE DRUM HOIST ARRANGEMENT FOR VERTICAL LIFT GATE

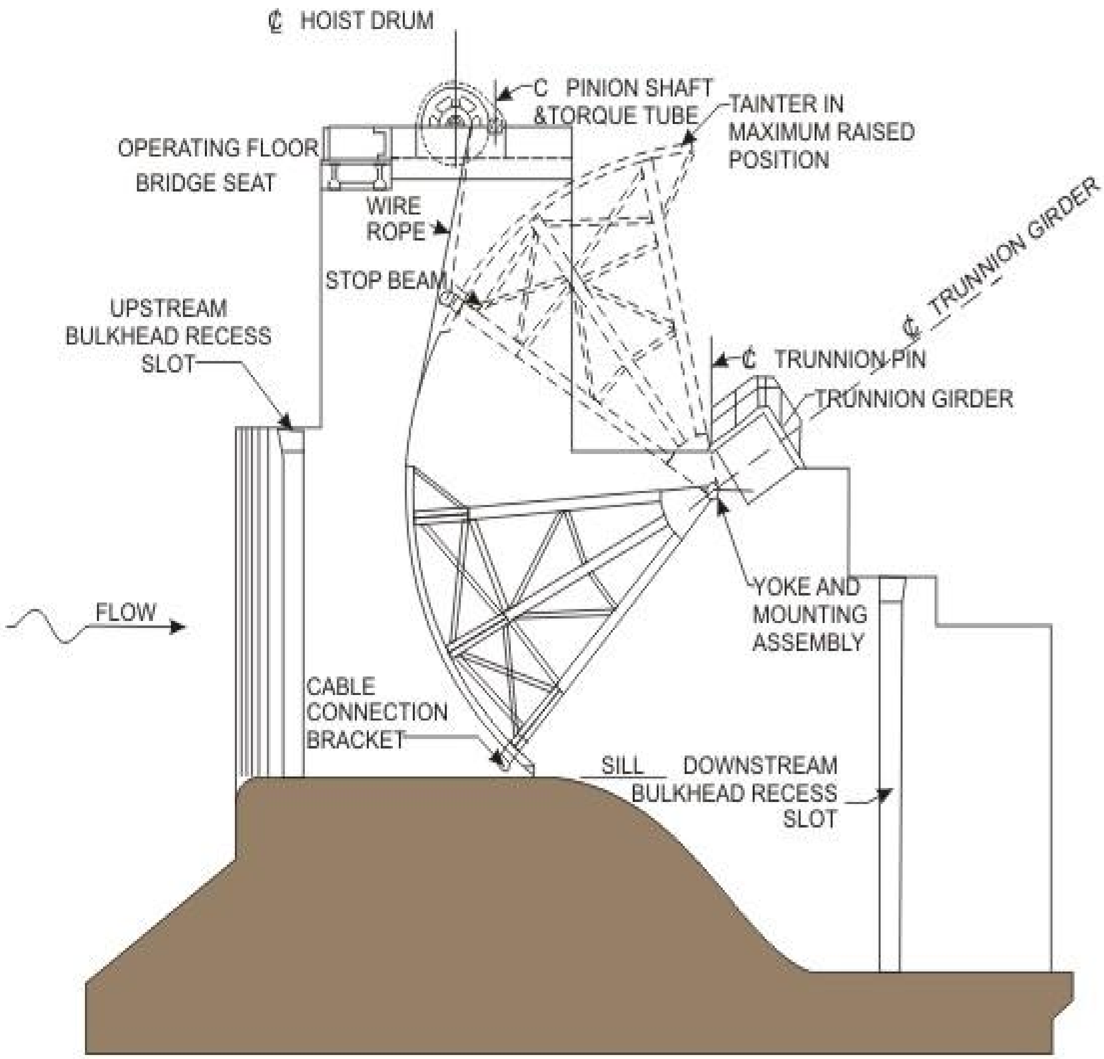


FIGURE 29. WIRE ROPE HOIST SYSTEM FOR RADIAL GATE

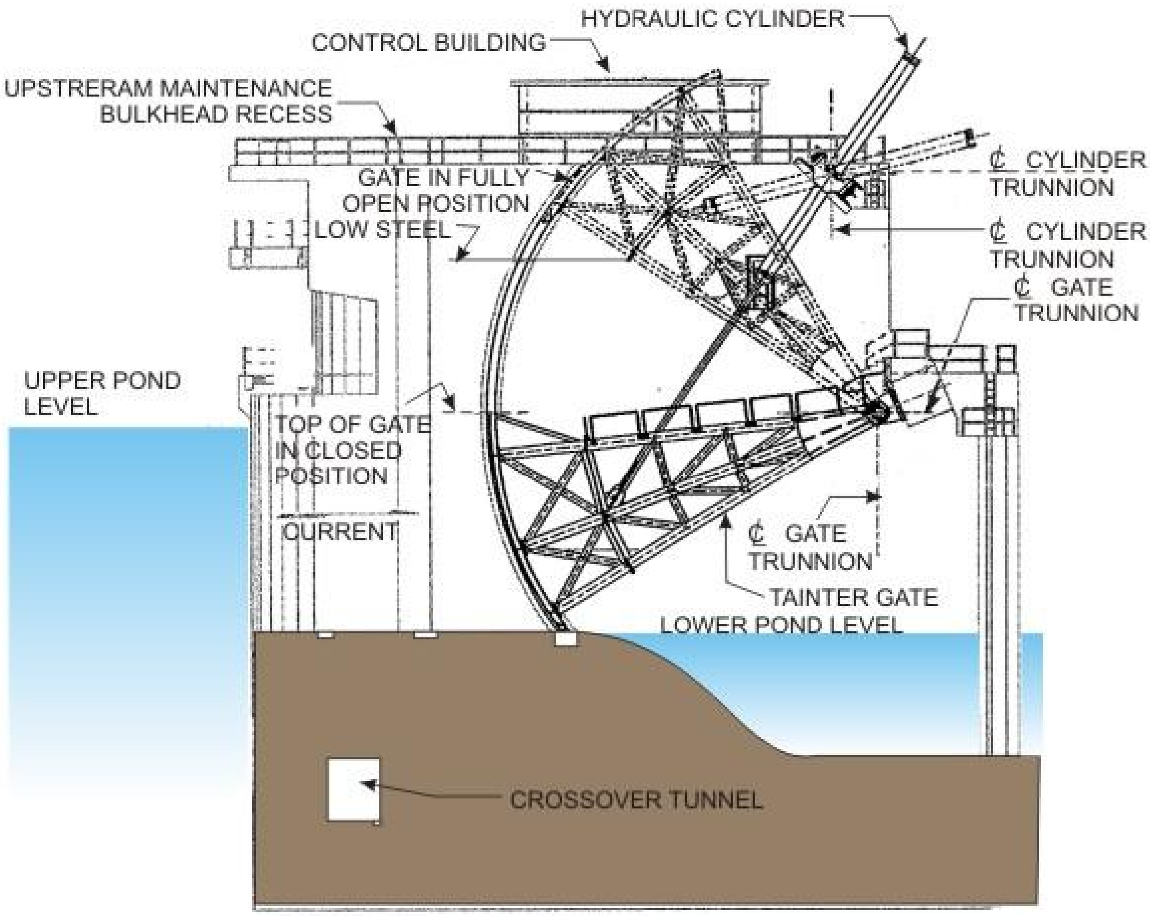


FIGURE 30. HYDRAULIC HOIST OPERATED TAINTER GATE

4.10.6 Valves for flow control

Valves are different from gates by their way of operation as they remain in the water passage both in the closed and open positions. This is unlike a gate which, when not controlling the flow, remains in an external position and in most cases out of water. Different types of valves used in water resources engineering are mostly used to control flow in the high pressure conduits like penstocks conveying water to turbines for generation of hydroelectricity. The Bureau of Indian Standards code IS: 4410 (Part 16, Section 2) – 1981 mentions a list of valves in use for various purposes. The valves that are commonly used for water resources projects are mentioned below:

1. Butterfly valve

A valve in which the disk is turned about 90 degrees from the close to the open position, about a spindle supported on the body of the valve on an axis transverse to that of the valve (Figure 31).

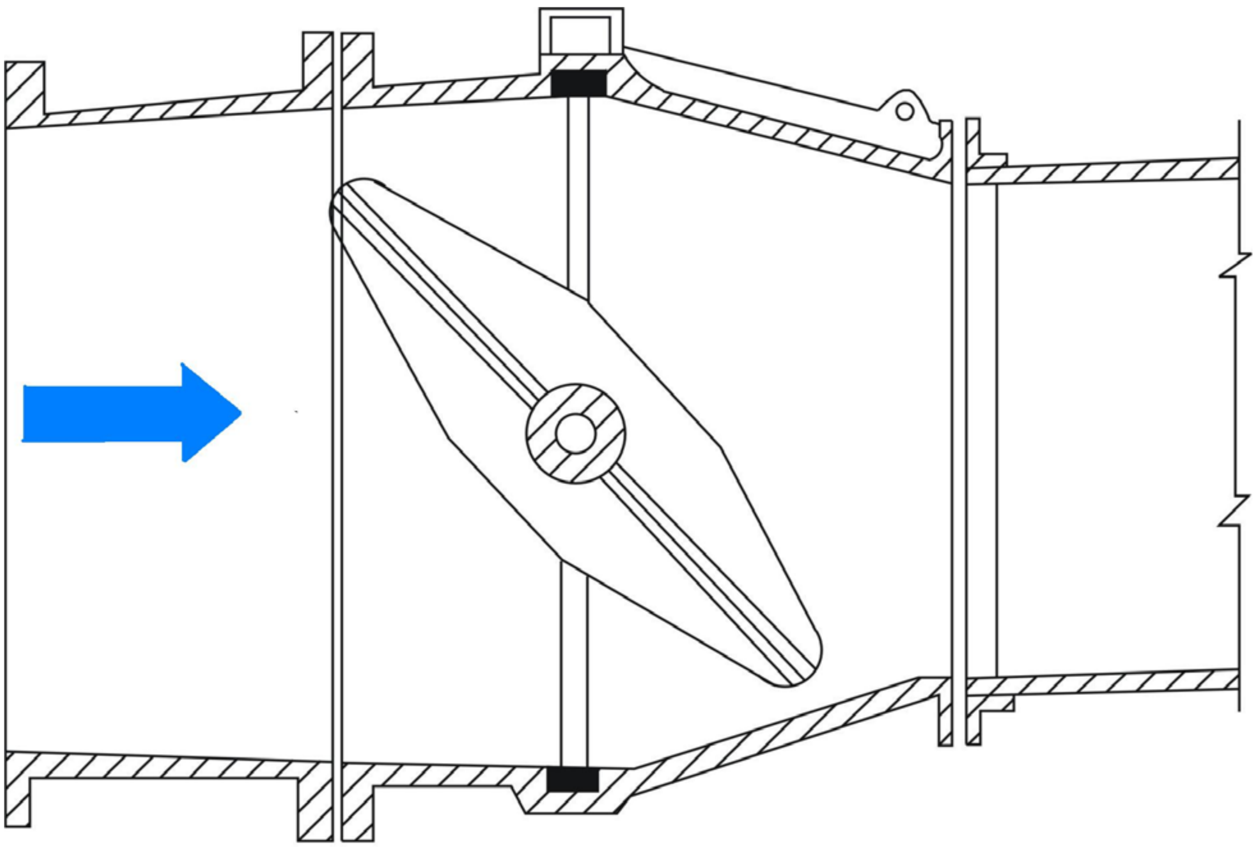


FIGURE 31. BUTTERFLY VALVE

2. Hollow jet valve

A high pressure valve wherein a needle, which, when moved downstream to open the valve, releases water in the form of a hollow jet (Figure 32).

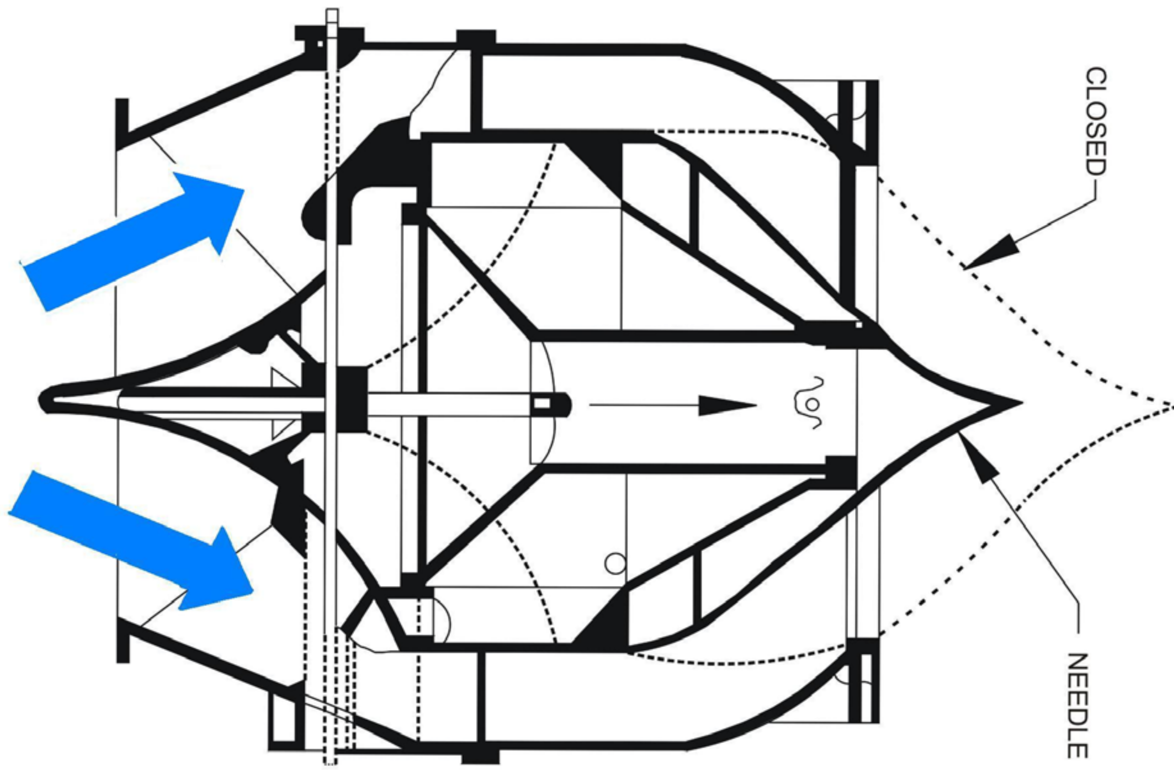


FIGURE32. HOLLOW JET VALVE

3. **Howell-Bunger (Cylindrical) valve**

A valve having two telescopic cylinders with a streamline dispersing cone secured to the inner cylinder by radial ribs. The outer cylinder closes the sideway opening between the cone and the inner cylinder when it is slid in position. In its open position, the water is discharged on the sides of the cylinder in the form of a highly diverging hollow inside in the shape of a cone (Figure 33).

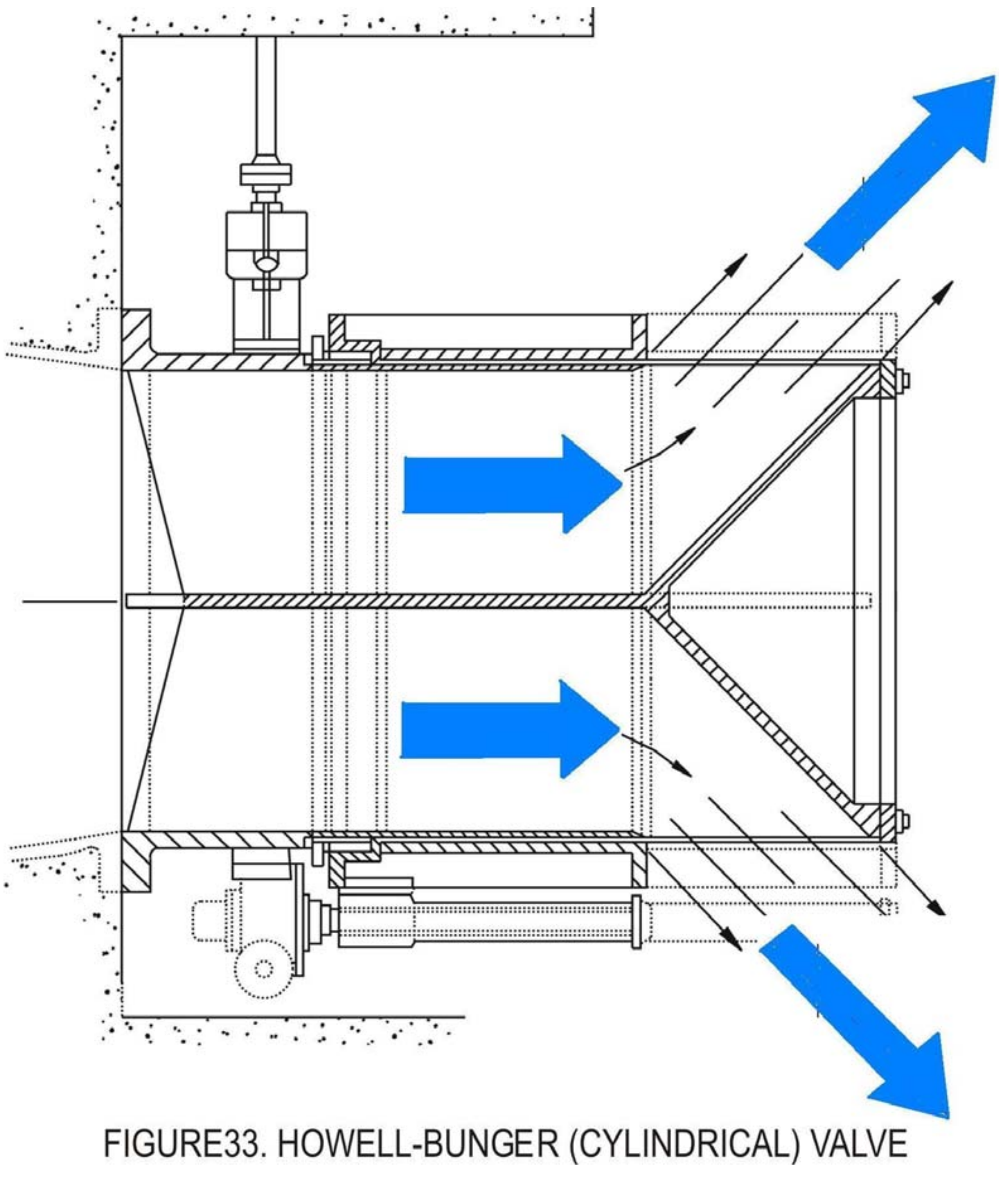


FIGURE33. HOWELL-BUNGER (CYLINDRICAL) VALVE

4. **Needle valve**

A valve with a circular outlet through which the flow is controlled by means of a tapered needle which extends through the outlet, reducing the area of the outlet as it extrudes, and enlarging the area as it retreats.

Balanced Needle Valve -A needle valve of improved design in which the needle is moved by water pressure from the outlet conduit, which acts on interior chambers in the valve. The movement is controlled by a hand wheel installed above the valve, with the motion transmitted through shafting and gearing to a poke positioning device located inside the valve (see Figure 34).

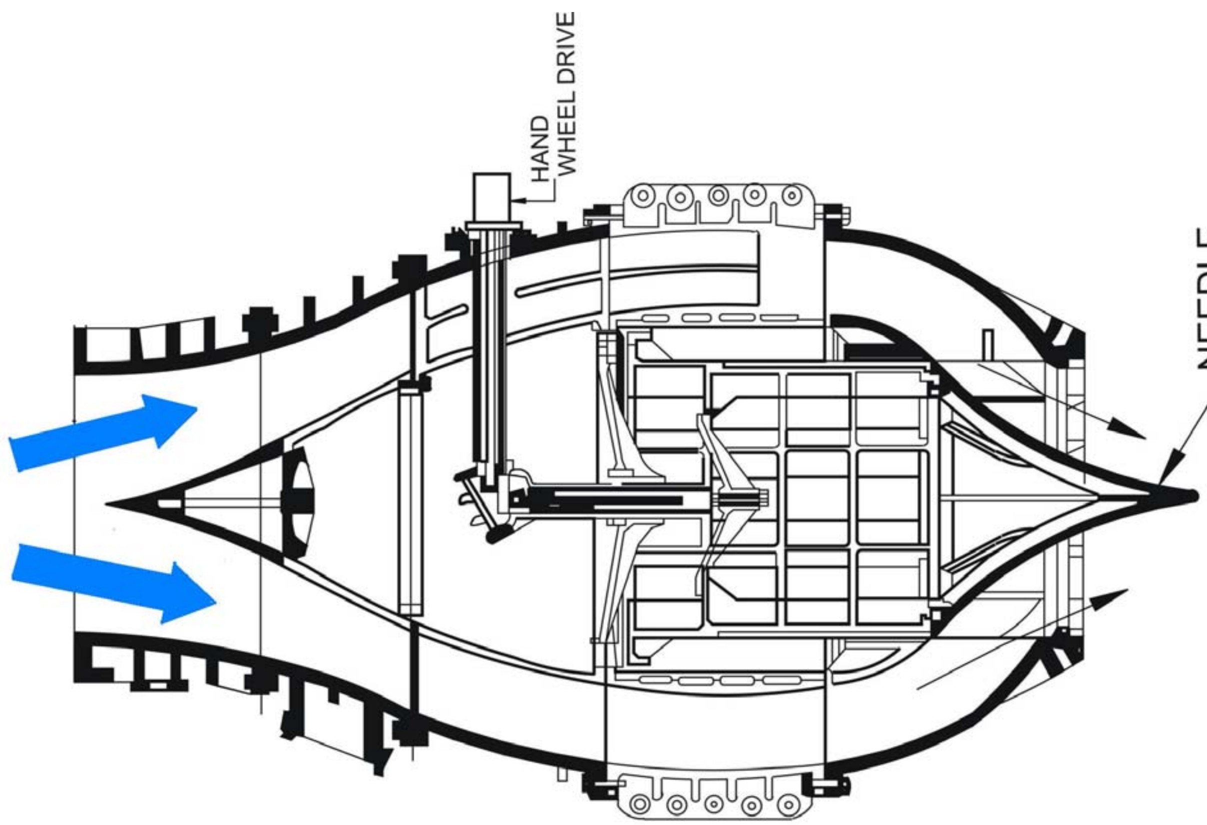


FIGURE34. BALANCED NEEDLE VALVE

Interior Differential Needle Valve- A differential needle valve with a needle that telescopes over a member fixed to the valve body instead of moving within the valve body as in the case of an internal differential needle valve (see Figure 35).

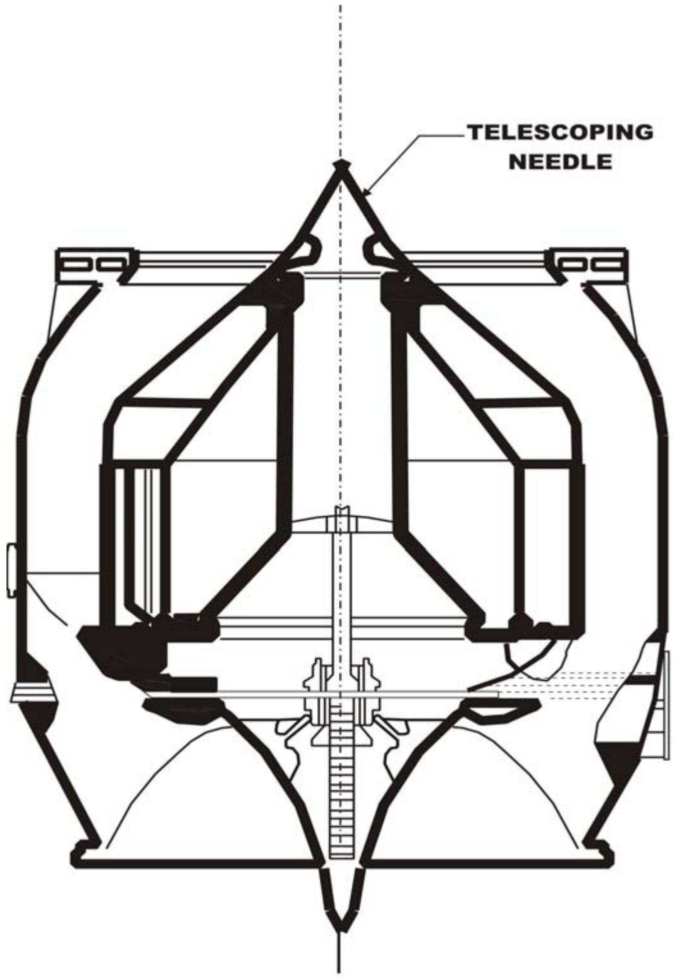


FIGURE 35. Interior differential needle valve

Internal Differential Needle Valve **-** This is an improved type of balanced needle valve with three chambers in the needle. The two end chambers are connected. The valve is operated by the differential thrust resulting from the changes in pressure in the end chambers with respect to that in the central chamber through a valve paradox (see Figure 36).

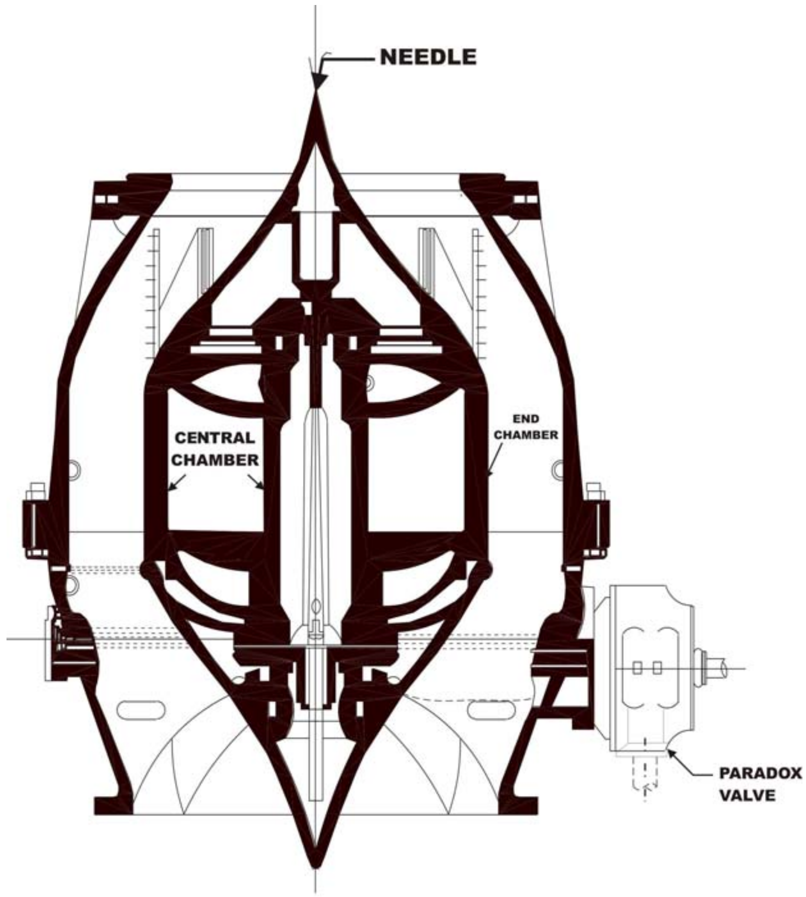


FIGURE 36 Internal differntial needle valve

Motor-operated Needle Valve- **This is a** needle valve in which the position of the needle is controlled by a motor-operated rod (see Figure 37).

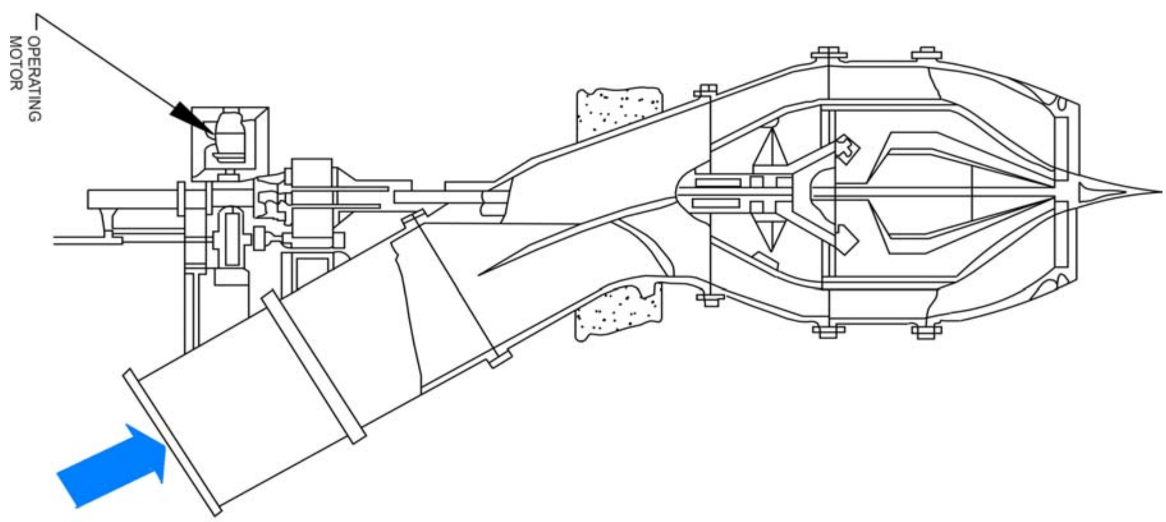


FIGURE 37. MOTOR-OPERATED NEEDLE VALVE

5. **Tube Valve -** An improvement over the needle valve. The water passages are similar to the internal differential valve, except that the downstream end of the needle is omitted. A tube or hollow cylinder similar to that of the cylinder gate, instead of a needle, comprises the moving part of the valve. This is actuated by a hydraulic cylinder and piston and a pressure pump or by a screw with an electric motor or by manual control (see Figure 38).

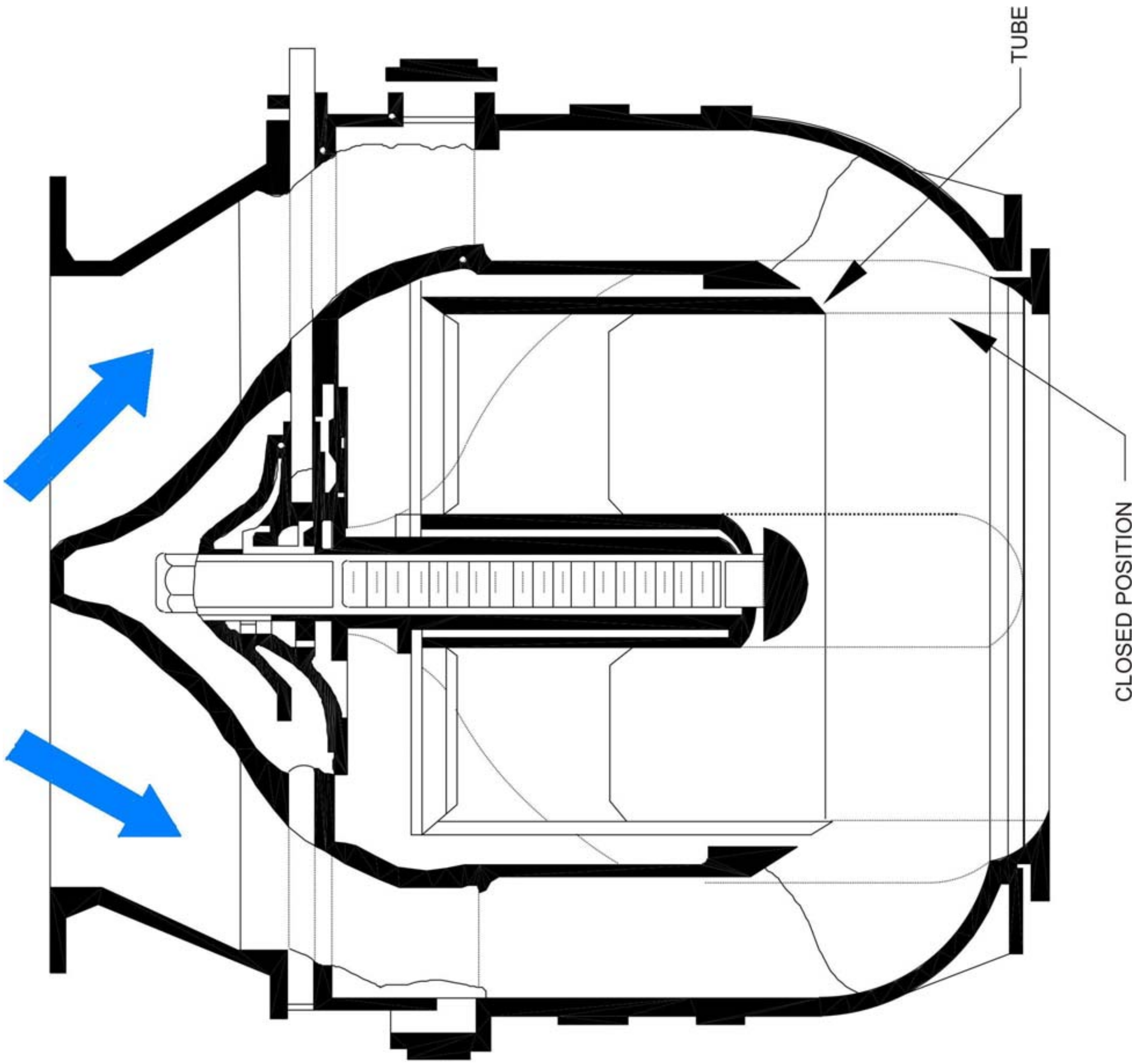


FIGURE 38. TUBE VALVE

6. Spherical or Rotary Valve - A valve consisting of a casing more or less spherical in shape, the gate turning on trunions through 90 degrees when opening or closing, and having a cylindrical opening of the same diameter as that of the pipe it serves (see Figure 39).

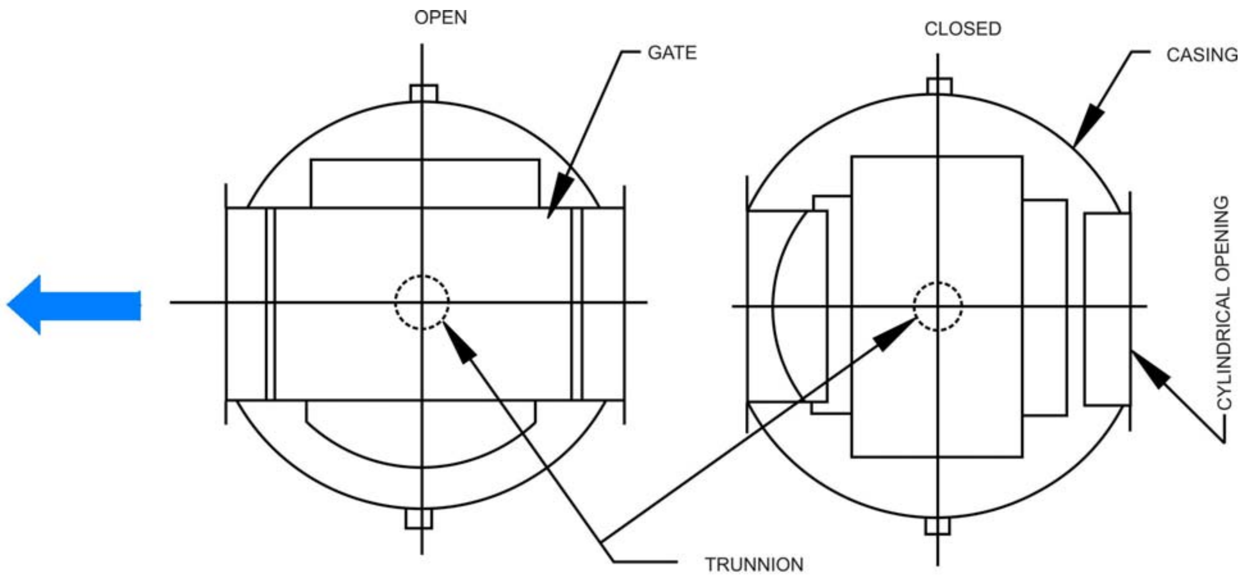


FIGURE 39. SPHERICAL(ROTARY) VALVE

Module 5

HYDROPOWER ENGINEERING

LESSON 1

PRINCIPLES OF HYDROPOWER ENGINEERING

Instructional objectives

On completion of this lesson, the student shall learn about:

1. Potential of hydropower that may be generated from a stream
2. Hydropower potential in India and the world
3. Types of hydropower generation plants
4. layouts of hydropower plants

5.1.0 Introduction

The water of the oceans and water bodies on land are evaporated by the energy of the sun's heat and gets transported as clouds to different parts of the earth. The clouds travelling over land and falling as rain on earth produces flows in the rivers which returns back to the sea. The water of rivers and streams, while flowing down from places of higher elevations to those with lower elevations, loose their potential energy and gain kinetic energy. The energy is quite high in many rivers which have caused them to etch their own path on the earth's surface through millions of years of continuous erosion. In almost every river, the energy still continues to deepen the channels and migrate by cutting the banks, though the extent of morphological changes vary from river to river. Much of the energy of a river's flowing water gets dissipated due to friction encountered with its banks or through loss of energy through internal turbulence. Nevertheless, the energy of water always gets replenished by the solar energy which is responsible for the eternal circulation of the Hydrologic Cycle.

Hydropower engineering tries to tap this vast amount of energy available in the flowing water on the earth's surface and convert that to electricity. There is another form of water energy that is used for hydropower development: the variation of the ocean water with time due to the moon's pull, which is termed as the tide. Hence, hydropower engineering deals with mostly two forms of energy and suggest methods for converting the energy of water into electric energy. In nature, a flowing stream of water dissipates throughout the length of the watercourse and is of little use for power generation. To make the flowing water do work usefully for some purpose like power generation (it has been used to drive water wheels to grind grains at many hilly regions for years), it is necessary to create a head at a point of the stream and to convey the water through the head to the turbines which will transform the energy of the water into mechanical energy to be further converted to electrical energy by generators. The necessary head can be created in different ways of which two have been practically accepted.

These are:

1. Building a dam across a stream to hold back water and release it through a channel, conduit or a tunnel (Figure 1)

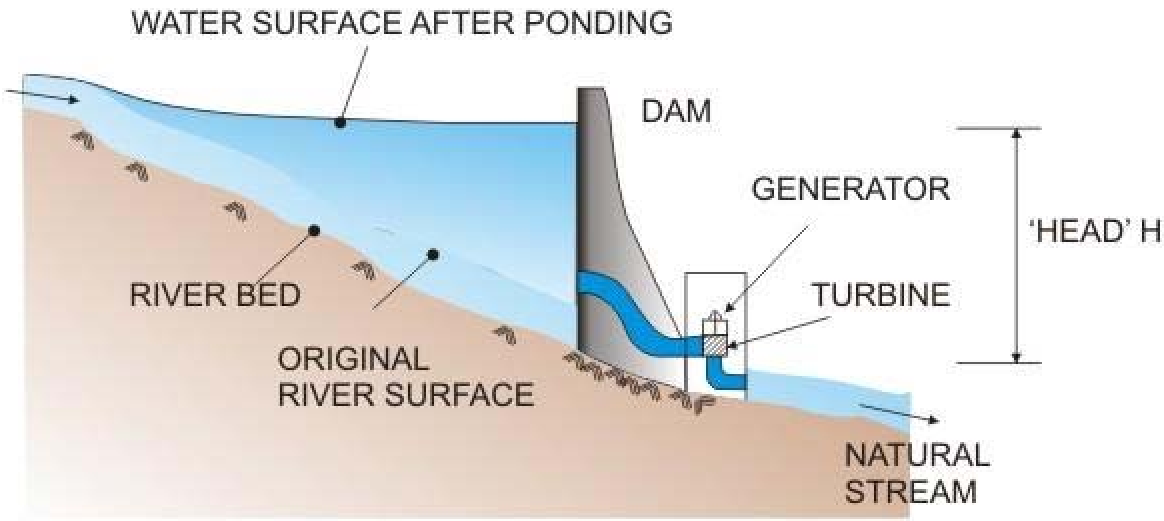


FIGURE 1. Creation of head by constructing a dam

2. Divert a part of the stream by creating a low-head diversion structure like barrage. (Figure 2)

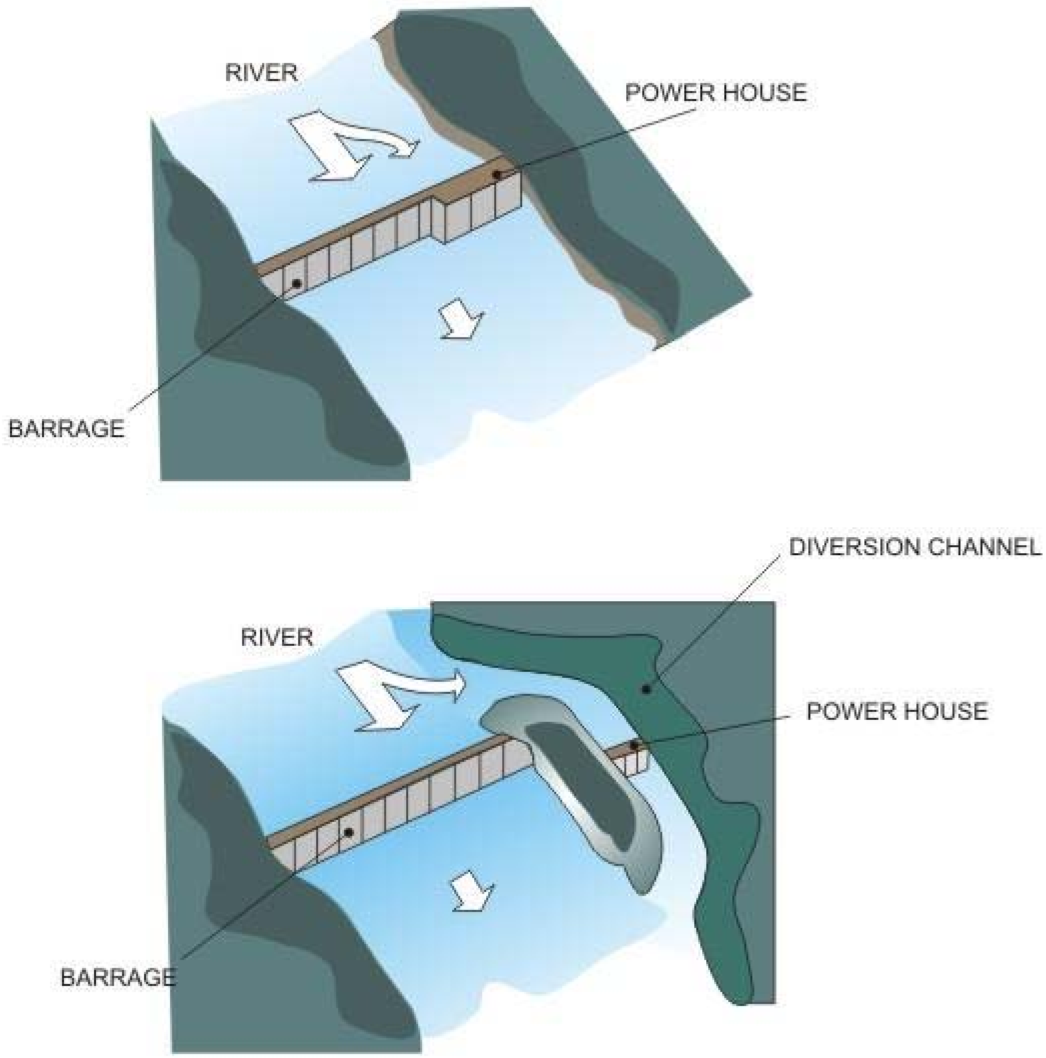


FIGURE 2. Power generation through streamflow diversion
(a) Power house as a part of diversion structure
(b) Power house on diversion channel

A series of integrated power developments along the same watercourse form what may be called a multistage hydroelectric system in which each portion of the river with a power plant of its own is referred to as a stage (Figure 3). The head created by a dam put across a lowland river usually ranges from 30 to 40m. In mountainous terrain, it may run over 200m.

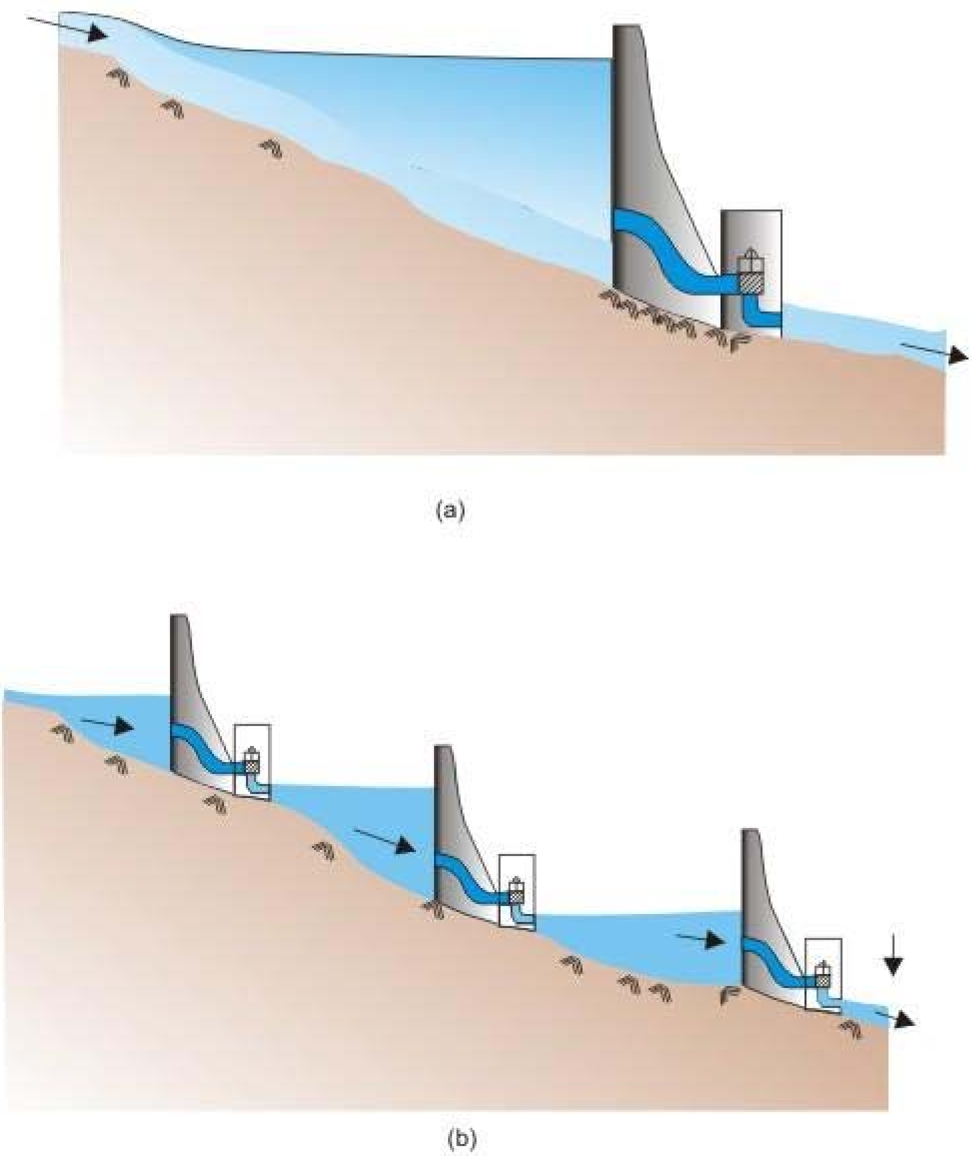


FIGURE 3 . (a) Single-stage hydro-power development scheme
(b) Cascade or Multi-stage hydro-system

The following sections briefly discuss the issues related to the fundamentals of hydropower project development.

5.1.1 Hydropower potential

Electricity from water is usually referred to as Hydro-Power, where the term 'hydro' is the Greek word for water and hydropower is the energy contained in water. It can be converted in the form of electricity through hydroelectric power plants. All that is required is a continuous inflow of water and a difference of height between the water level of the upstream intake of the power plant and its downstream outlet.

In order to evaluate the power of flowing water, we may assume a uniform steady flow between two cross-sections of a river, with **H** (metres) of difference in water surface elevation between two sections for a flow of **Q** (m^3/s), the power (**P**) can be expressed as

$$P = \gamma Q \left(H + \frac{v_1^2 - v_2^2}{2g} \right) \quad \text{[Nm/s]}$$

where $\mathbf{v_1}$ and $\mathbf{v_2}$ are the mean velocities in the two sections. Neglecting the usually slight difference in the kinetic energy and assuming a value of γ as 9810N/m^2, one obtains the expression of power as

$$P = 9810QH \quad \text{[Nm/s]}$$

Since an energy of 1000Nm/s can be represented as 1kW (1kilo-Watt), one may write the following:

$$P = 9.81QH \quad \text{[kW]}$$

The above expression gives the theoretical power of the selected river stretch at a specified discharge.

In order to evaluate the potential of power that may be generated by harnessing the drop in water levels in a river between two points, it is necessary to have knowledge of the hydrology or stream flow of the site, since that would be varying everyday. Even the average monthly discharges over a year would vary. Similarly, these monthly averages would not be the same for consecutive years. Hence, in order to evaluate the hydropower potential of a site, the following criteria are considered:

1. Minimum potential power is based on the smallest runoff available in the stream at all times, days, months and years having duration of 100 percent. This value is usually of small interest
2. Small potential power is calculated from the 95 percent duration discharge
3. Medium or average potential power is gained from the 50 percent duration discharge
4. Mean potential power results by evaluating the annual mean runoff.

Since it is not economically feasible to harness the entire runoff of a river during flood (as that would require a huge storage), there is no reason for including the entire magnitude of peak flows while calculating potential power or potential annual energy.

Hence, a discharge-duration curve may be prepared (Figure 4) which plots the daily discharges at a location in the decreasing order of magnitude starting from the largest daily discharge observed during the year and going upto the minimum daily discharge.

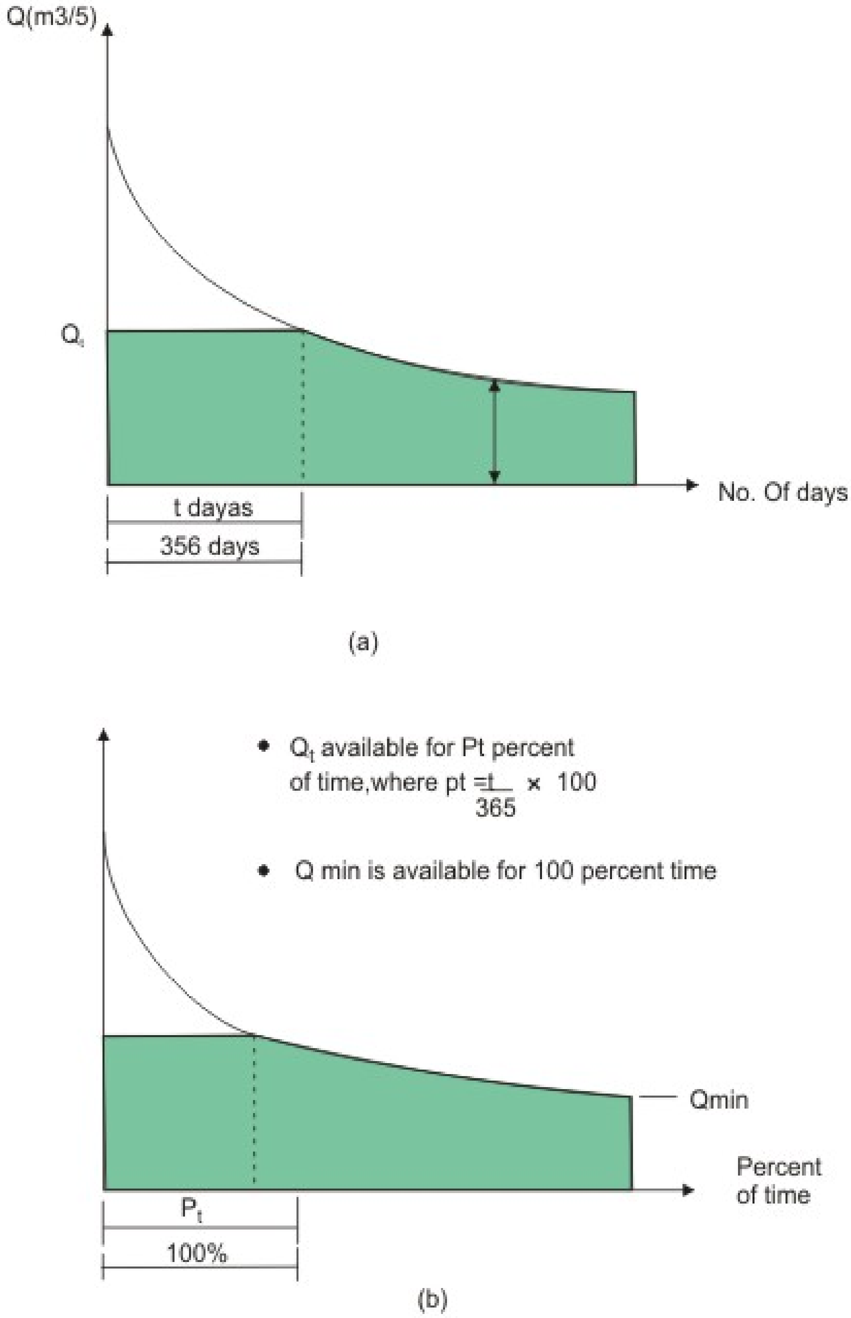


FIGURE 4. Flow curve for one year (a) expressed in time; (b) expressed in percentage of time

From this annual discharge curve, a truncation is made at a discharge Q_t which is the discharge corresponding to a time of 't' days, where t can be the median (say, 182 days or 50 percent duration, denoted by (Q_{182} or $Q_{50\%}$), or a higher Q_t (t less than 182 days) can be selected by specialists who are familiar with the local conditions and future plans for power supply. Accordingly, the annual magnitude of potential (theoretical) energy can be computed in KWh as below and referring to Figure 4:

$$E_p = 24 \times 9.81 H \left(Q_t + \sum_i^{365} Q_i \right)$$

$$\approx 235 H \cdot A \qquad \text{(in kWh)}$$

Where Q_i denotes the daily mean flow during the period 365-t days and A, the hatched area cut by Q_t, where the area under the curve has a unit m^3×day/s.

The massive influx of water in the hydrologic cycle has an estimated potential for generating, on a continuous basis, 40,000 billion units (TWh) of power annually for the whole world (CBIP, 1992). Hydropower potential is commonly divided into three categories:

a) ***Theoretical*** : 40,000 TWh
b) ***Technical*** : 20,000 TWh
c) ***Economical*** : 9,800 TWh

The terms used above are explained below:

Theoretical

The gross theoretical potential is the sum of the potential of all natural flows from the largest rivers to the smallest rivulets, regardless of the inevitable losses and unfeasible sites.

Technical

From technical point of view, extremely low heads (less than around 0.5m), head losses in water ways, efficiency losses in the hydraulic and electrical machines, are considered as infeasible. Hence, the technically usable hydro potential is substantially less than the theoretical value.

Economic

Economic potential is only that part of the potential of more favourable sites which can be regarded as economic compared to alternative sources of power like oil and coal. Economically feasible potential, therefore, would change with time, being dependent upon the cost of alternate power sources. This potential is constantly updated and shows an increasing trend with the exhausting stock of fossil fuel.

The following table taken from CBIP (1992) shows a continental break-up of world's economical hydropower potential. Asia is seen to be endowed with the maximum hydropower potential.

Region	Available potential (Billion units)
Asia (except C.I.S and Russia)	2700
C.I.S and Russia	1100
Africa	1590
North America	1580
South America	1910
Europe(except C.I.S and Russia)	720
Oceania	200
Total	9800

Some nations have enough hydropower to become exporters of electricity. Switzerland, for example, exports electricity to neighbouring France and Italy. Nepal, Bhutan, Peru and Laos are similarly blessed with abundant hydro resources. Within India, Meghalay is probably the only state generating hydropower more than its requirements and exports power to the neighbouring state of Assam.

In India, it has been estimated by the Central Electric Authority, that the hydroelectric potential of the entire country is around 84,044MW at 60 percent load factor. The annual energy contribution of this potential would be about 600 billion units including seasonal/secondary energy which is the additional energy generation in any year above the firm annual energy. Basin wise potential within the country is shown in Figure 5.

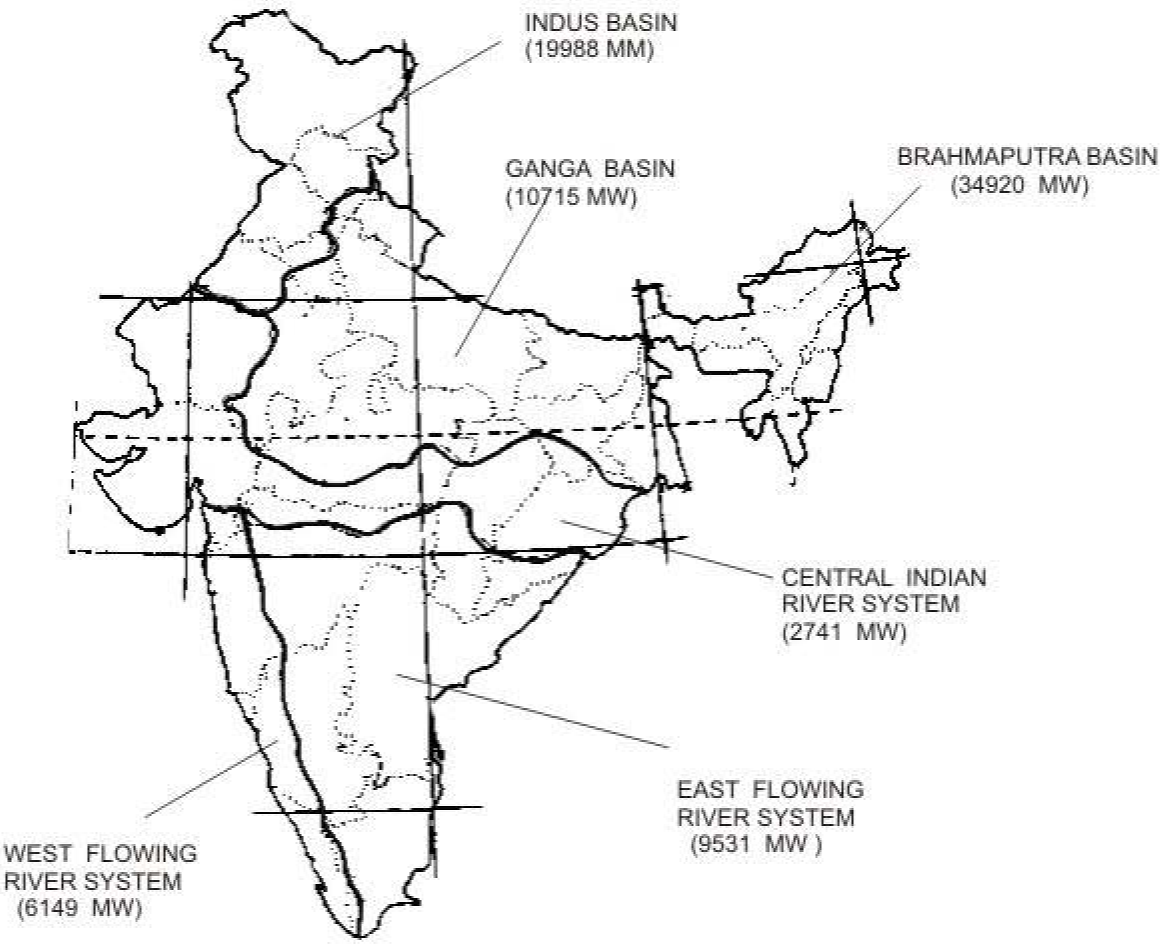


FIGURE 5. Basin-wise hydro power potential of India.

5.1.2 Types of hydroelectric projects

Hydroelectric plants are classified commonly by their hydraulic characteristics, that is, with respect to the water flowing through the turbines that run the generators. Broadly, the following classifications. made are shown in Figure 6.

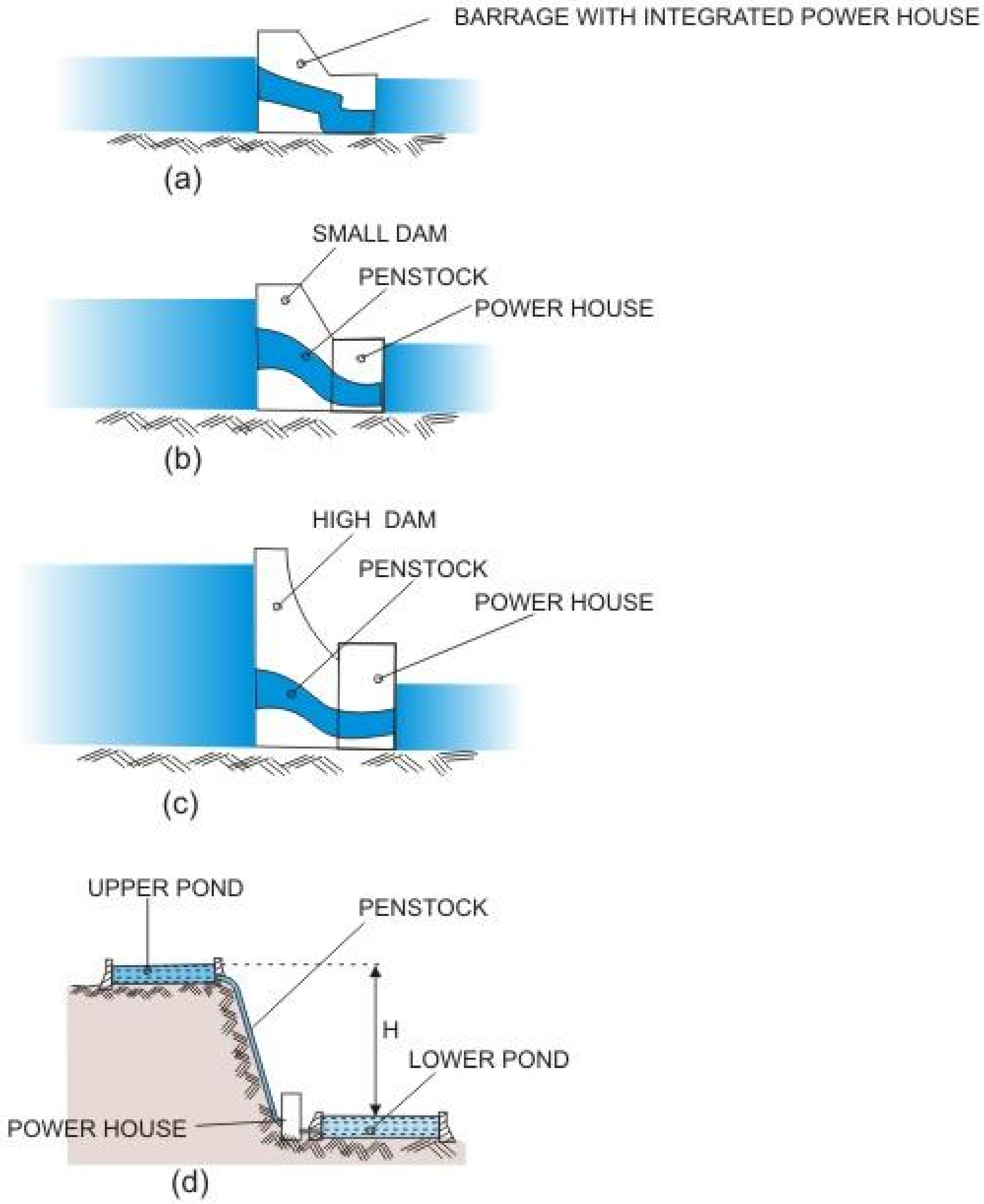


FIGURE 6. Types of hydroelectric schemes
(a) Run-of-river without pondage(little or no storage)
(b) Run-of-river with pondage (storage suitable to balance diurnal variation in power generation)
(c) Storage schemes (reservoirs to store excess water of flood flows)
(d) Pump- storage schemes

1. Run-of-river schemes

These are hydropower plants that utilize the stream flow as it comes, without any storage being provided (Figure6a). Generally, these plants would be feasible only on such streams which have a minimum dry weather flow of such magnitude which makes it possible to generate electricity throughout the year. Since the flow would vary throughout the year, they would run during the monsoon flows and would otherwise remain shut during low flows. Of course, the economic feasibility of providing the extra units apart from the regular units have to be worked out. Further, the monsoon tailwater in rivers with flat slopes becomes higher, causing the plants to become inoperative. Run-of-river plants may also be provided with some storage (Figure6b) to take care of the variation of flow in the river as for snow-melt rivers, emerging from the glaciers of Himalayas. During off-peak hours of electricity demand, as in the night, some of the units may be closed and the water conserved in the storage space, which is again released during peak hours for power generation. A schematic cross sectional view of a typical run-of-river scheme is shown in Figure 7.

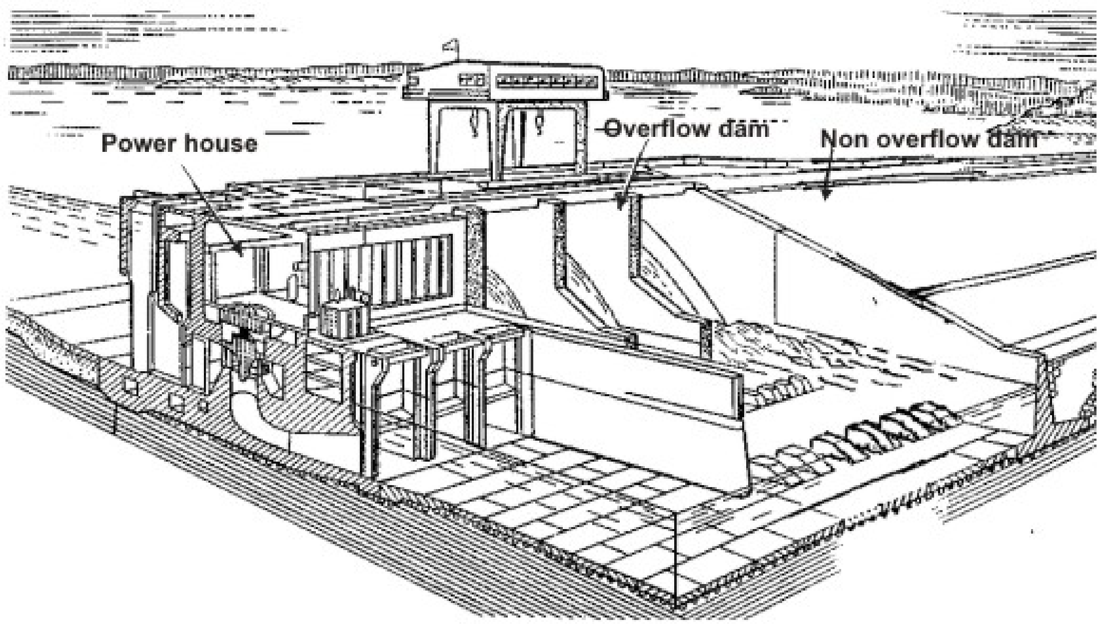


FIGURE 7. A typical run of-river hydroelectric station using a dam and an in-stream power house

2. Storage schemes

Hydropower plants with storage are supplied with water from large storage reservoir (Figure 6c) that have been developed by constructing dams across rivers. Generally, the excess flow of the river during monsoon would be stored in the reservoir to be released gradually during periods of lean flow. Naturally, the assured flow for hydropower generation is more certain for the storage schemes than the run-of-river schemes. A typical schematic cross sectional view of a storage scheme power plant is shown in Figure 8.

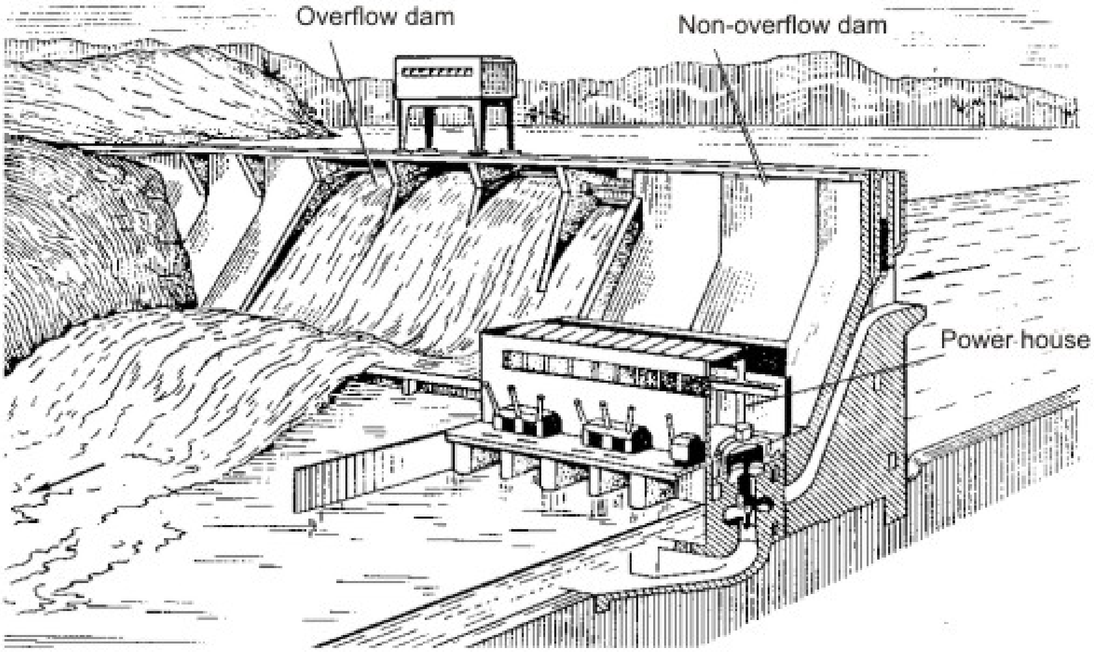


FIGURE 8. A typical storage-type hydroelectric station with a power house build at the toe of the dam

3. Pumped-Storage schemes

Hydropower schemes of the pumped-storage type are those which utilize the flow of water from a reservoir at higher potential to one at lower potential (Figure 6d). A typical schematic view of such a plant is shown in Figure 9. The upper reservoir (also called the head-water pond) and the lower reservoir (called the tail-water pond) may both be constructed by providing suitable structure across a river (Figure 10). During times of peak load, water is drawn from the head-water pond to run the reversible turbine-pump units in the turbine mode. The water released gets collected in the tail-water pond. During off-peak hours, the reversible units are supplied with the excess electricity available in the power grid which then pumps part of the water of the tail-water pond back into the head-water reservoir. The excess electricity in the grid is usually the generation of the thermal power plants which are in continuous running mode. However, during night, since the demand of electricity becomes drastically low and the thermal power plants can not switch off or start immediately, there a large amount of excess power is available at that time.

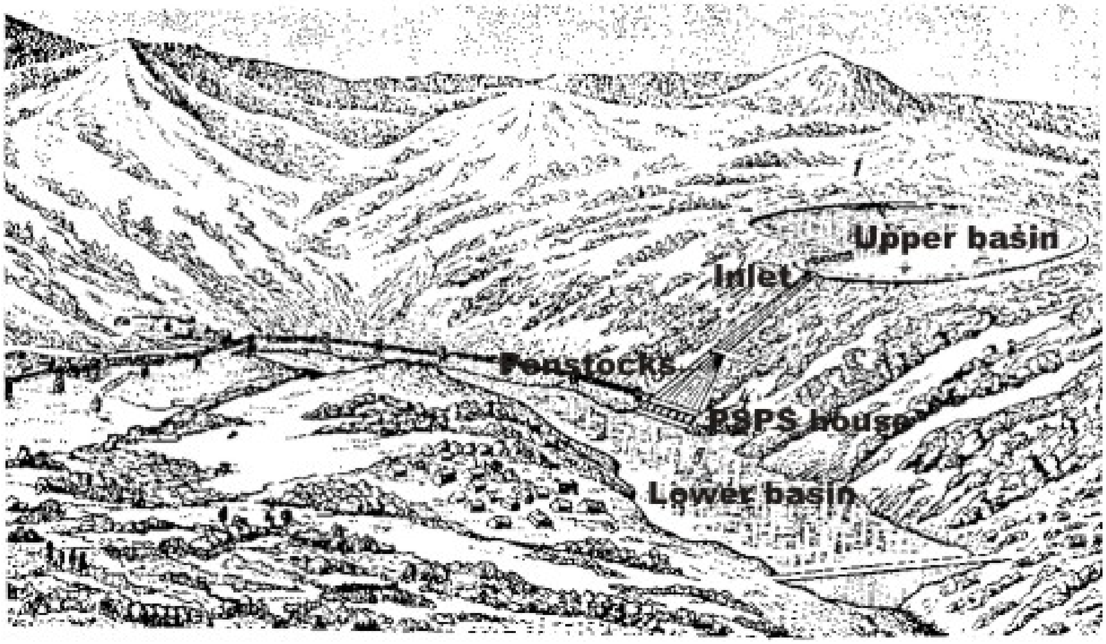


FIGURE 9. General view of pumped stroage power station

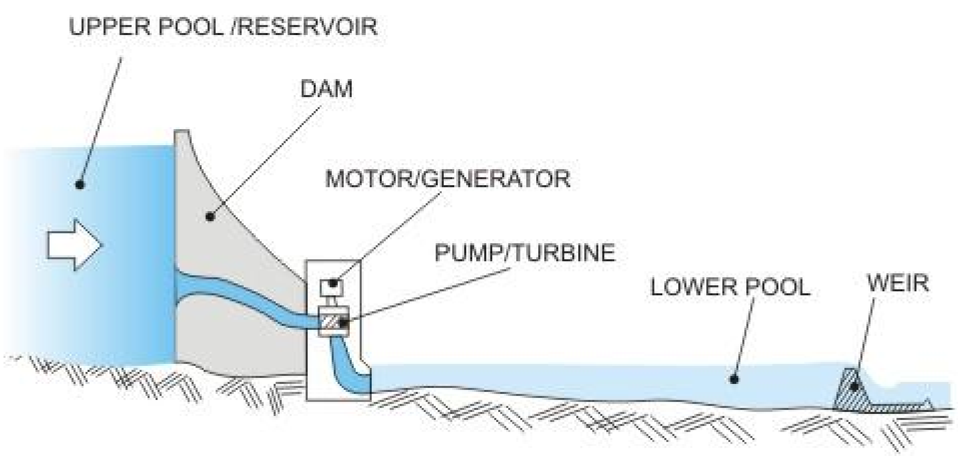


Figure 10. Pump-storage scheme development with upper and lower pools in the same river

4. Tidal power development schemes

These are hydropower plants which utilize the rise in water level of the sea due to a tide, as shown in Figure 11. During high tide, the water from the sea-side starts rising, and the turbines start generating power as the water flows into the bay. As the sea water starts falling during low tide the water from the basin flows back to the sea which can also be used to generate power provided another set of turbines in the opposite direction are installed. Turbines which generate electricity for either direction of flow may be installed to take advantage of the flows in both directions.

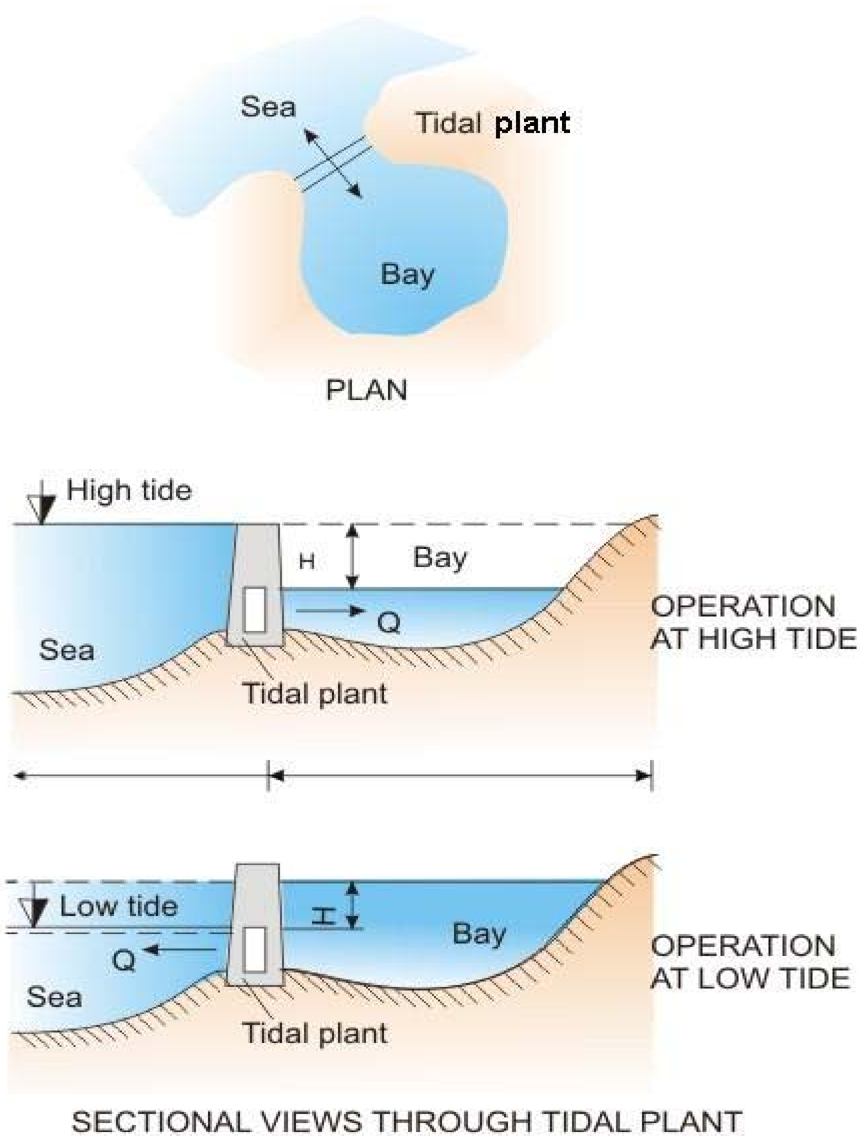


FIGURE 11. Concept of a tidal power development scheme

According to the National Oceanographic and Atmospheric Administration, USA, the potential energy of tides (often referred to as Blue Oil) is estimated at $3*10^6$ MW, of which one-third is dissipated in shallow seas. This implies that the exploitable energy available on sea coasts is of the order of 10^6 MW. Power can be generated where sufficiently large tides are available. According to experts it may be techno-economically possible to eventually develop 170,000MW at 30 sites worldwide. Globally, so far around 265 MW has been developed, although around 120,000MW are in the planning stage.

Hydroelectric power plants are also sometimes classified according to the head of water causing the turbines to rotate. The Bureau of Indian Standards code IS: 4410(Part10)-1998 "Glossary of terms relating to river valley projects: Hydroelectric power station including water conductor system," the following types of power plants may be defined:

1. **Low head power plant:** A power station that is operating under heads less than 30m (Figure12).

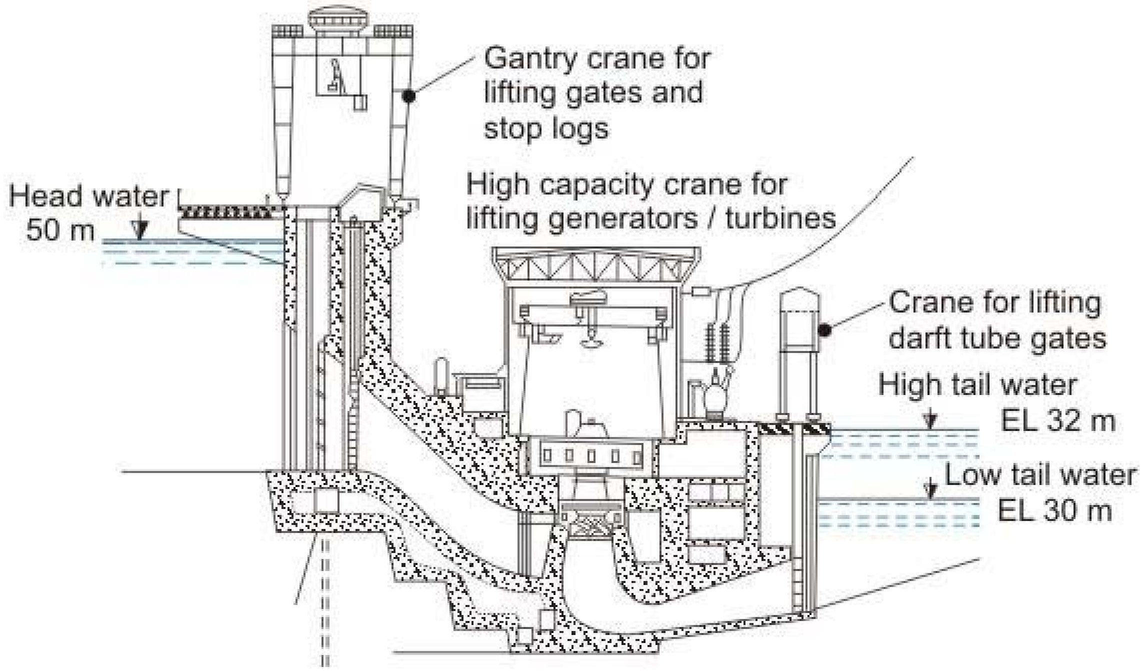


FIGURE 12. Sectional view of a typical low head hydro power station

2. **Medium head power plant:** A power station operating under heads from 30m to 300m. Of course, the limits are not exactly defined and sometimes the upper limit for medium head power station may be taken as 200 to 250m. (Figure 13)

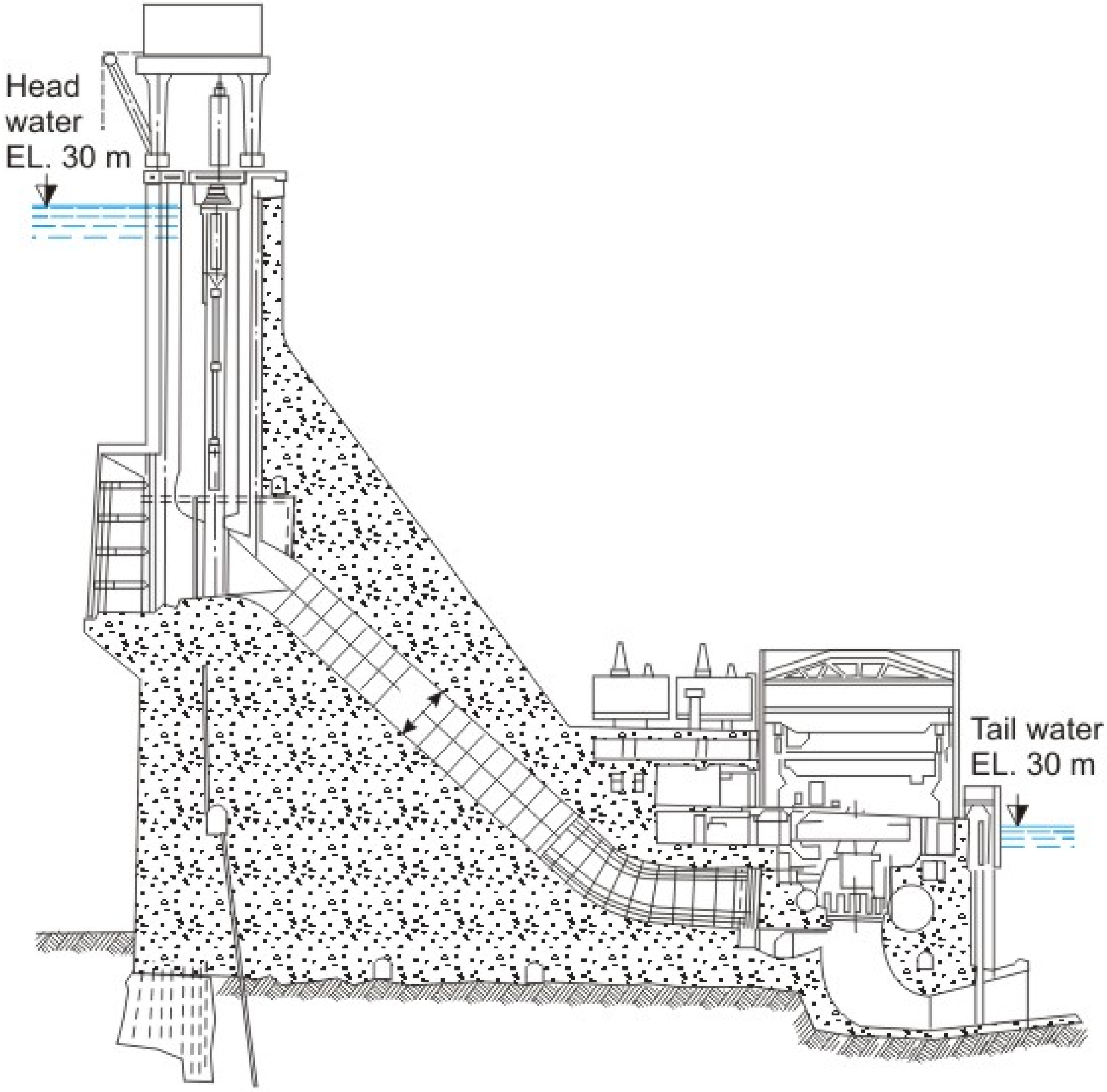
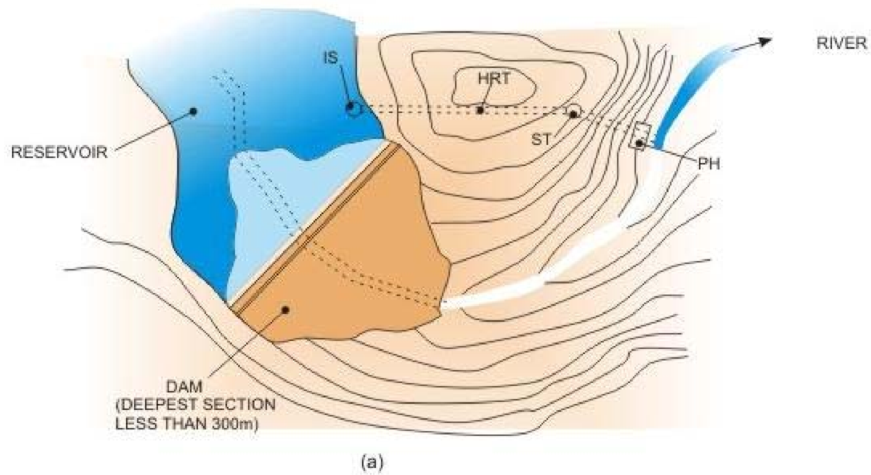


FIGURE 13. Sectional view of a typical medium head hydropower station.

3. **High head power station:** A power station operating under heads above about 300m. A head of 200m/250m is considered as the limit between medium and high head power stations. (Figure 14).



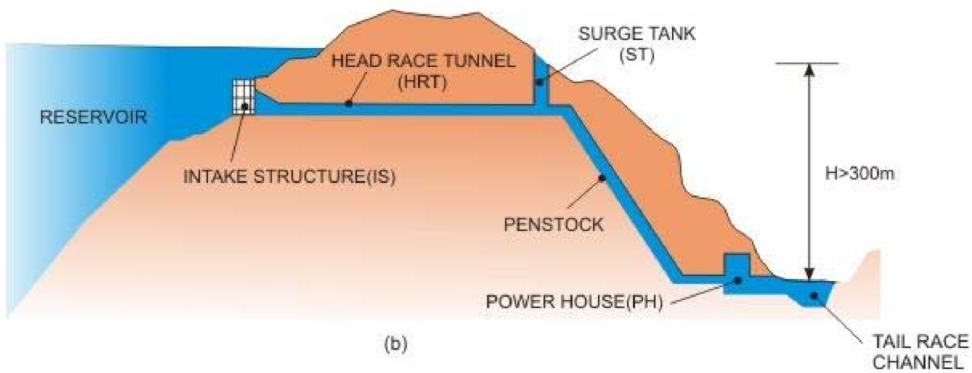


FIGURE 14. (a) Layout of the project showing an embankment dam creating a reservoir with a high head
(b) Sectional view through the water conducting system for hydropower

IS: 4410(part10)-1998 also classifies hydropower plants according to their operating functions as follows:

1. Base load power plant: A power station operating continuously at a constant or nearly constant power and which operates at relating high load factors. It caters to power demand at base of the load curve.
2. Peak load power plant: A power station that is primarily designed for the purpose of operating to supply the peak load of a power system. This type of power station is also, therefore, termed as 'Peaking station'.

According to Mosonyi (1991), hydropower plants can also be classified according to plant capacity as follows:

1. Midget plant: up to 100KW
2. Low-capacity plant up to 1,000KW
3. Medium capacity plant up to 10,000KW
4. High capacity plant > 10,000KW

In India, Micro-hydel plants with capacity less than 5000KW are being encouraged to tap small streams and canal falls. Of the larger hydropower stations in India, the following are at the top of the list:

Sl. No	Project	Number of units × Capacity	Total capacity (MW)
1.	Bhakra	5*108(MW)+5*132(MW)	1200
2.	Dehar	6*165(MW)	990
3.	Koyna	4*165(MW)+4*75(MW)+4*80(MW)	880
4.	Nagarjuna sagar	1*110(MW)+7*100(MW)	891
5.	Srisailam	7*110(MW)	770
6.	Sharavathy	10*89.1(MW)	891
7.	Kalinadi	6*135(MW)	810
8.	Idukki	6*130(MW)	780

A map showing the major hydroelectric power station in India as given in CBIP (1987) is shown in Figure 15.

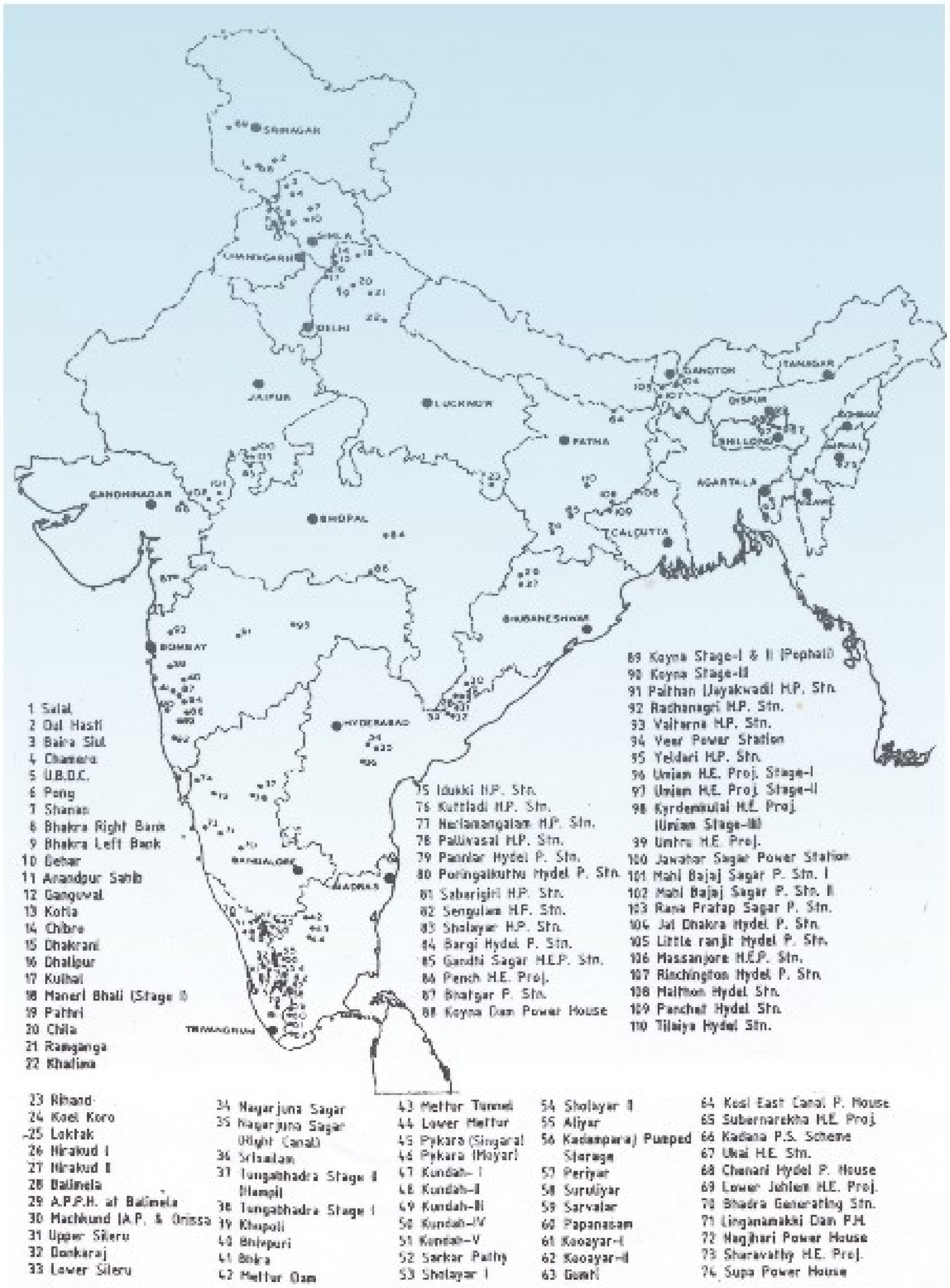


FIGURE 15. Hydro electric power stations in India

World-wide, there are a few hydropower plants with capacity greater than 10,000MW. These are given in the following list.

Sl.no	project	Country	Total Capacity
1.	Turukhnok	C.I.S.	20,000MW
2.	Three Gorges	China	13,400MW
3.	Itaipu	Brazil	12,000MW
4.	Grand Coulee	U.S.A	10,830MW
5.	Guri	Venezuela	10,300MW

On the other hand, China has over 88,000 small hydropower stations with a total installed capacity of 6929MW generating one-third of all the electricity consumed in rural areas. Hence, emphasis on micro-hydel development cannot be overlooked and similar developments can be done in the hilly regions of India where streams and small rivers may be tapped to provide power locally to the neighbouring rural community.

Though there has been a study growth of hydropower development in India over the years (Figure16), the proportional contribution of hydropower to the country's total energy production is rather small (Figure17).

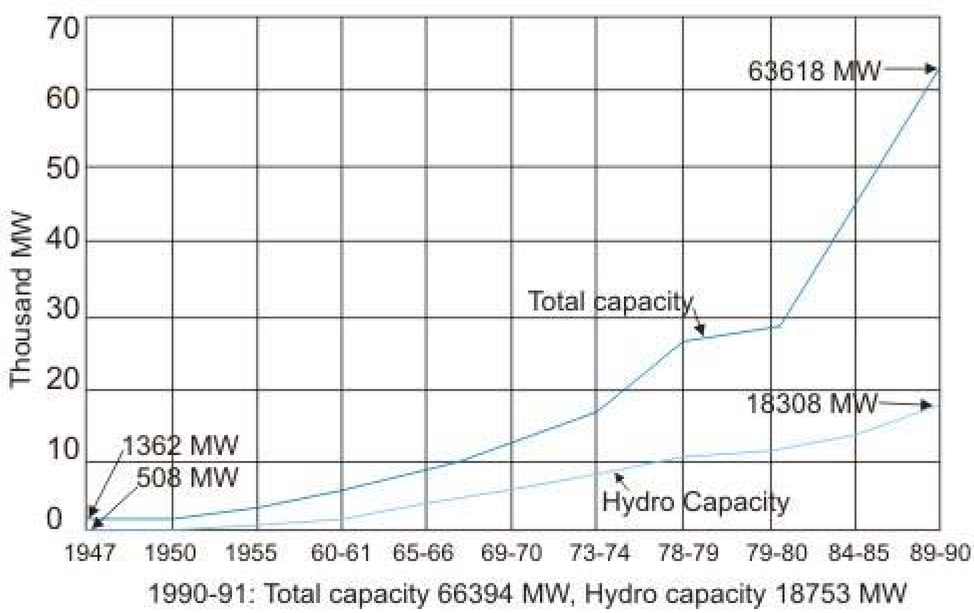


FIGURE 16. Total energy production and hydro power contribution for india

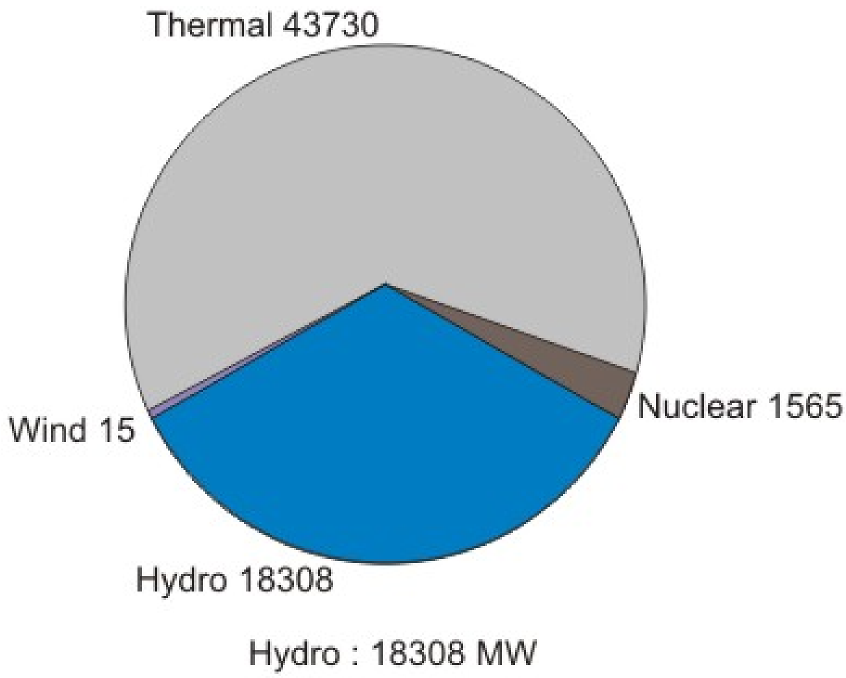


Figure 17. Installed capacity of india's energy producing units. This shows a hydro thermal mixratio of 29:71 approximately

In comparison, there are countries where hydropower production is the major source of electricity as given in the following table:

Rank	Country	Hydro production	Share of electricity
1.	Norway	83.5	99.4
2.	Zambia	8.8	98.9
3.	Zaire	4.3	98.6
4.	Ghana	4.7	98.5
5.	Mozambique	13.6	97.1
6.	Brazil	127.0	92.7
7.	Zimbabwe	4.0	88.9
8.	Sri Lanka	1.5	88.6
9.	New Zealand	16.3	74.1
10.	Nepal	0.2	73.6
11.	Switzerland	33.6	69.7
12.	Austria	29.1	69.3
13.	Canada	251.0	68.4
14.	Colombia	13.8	67.0
15.	North Korea	22.5	64.3
16.	Sweden	61.8	64.1

Ideally, for India, the Hydro: Thermal mix of around 40:60 has been considered to be the optimum.

5.1.3 Hydropower plant scheme layout

Typical components of a hydroelectric plant consist of the following:

1. Structure for water storage and/or diversion, like a dam or a barrage.
2. A head-race water conveying system like a conduit (penstock) or an open channel to transport water from the reservoir or head-water pool up to the turbines.
3. Turbines, coupled to generators
4. A tail race flow discharging conduit of open channel that conveys the water out of the turbine up to the river.

Although the above components are common for all hydropower development schemes the general arrangement for high and medium head power houses are more or less similar. The low head power plants, which are usually of run-of-power type schemes, have a slightly different arrangement as mentioned in the paragraphs below.

High and medium head development

Usually, there could be two types of power scheme layout:

- Concentrated fall schemes
- Diversion schemes

In the concentrated fall type projects, the powerhouse would be built at the toe of a concrete gravity dam, shown as a schematic view in Figure 7 and sectional view in Figure 12. This type of project development is suitable for medium head projects since a high head project would require an enormous concrete gravity dam, which is generally not adopted. A medium or high head project with an earthfill or rockfill dam may have an isolated or off-stream power house as shown in Figure 13. Here, the water is conveyed to the turbines via penstocks laid under, or by-passing, the dam. Spillways are provided separately to take care of floods. A distinction of such projects is that it consists of a long system of water conduits. Surge tanks are sometimes provided at the end of the conduits to relieve them of water hammer, which is the very high pressure developed by causing the stoppage of flow too suddenly at the turbine end.

In the diversion type of layout, the diversion could be using a canal and a penstock (Figure 18) or a tunnel and a penstock (Figure 19). The former is usually called the Open-Flow Diversion System and the latter Pressure Diversion System.

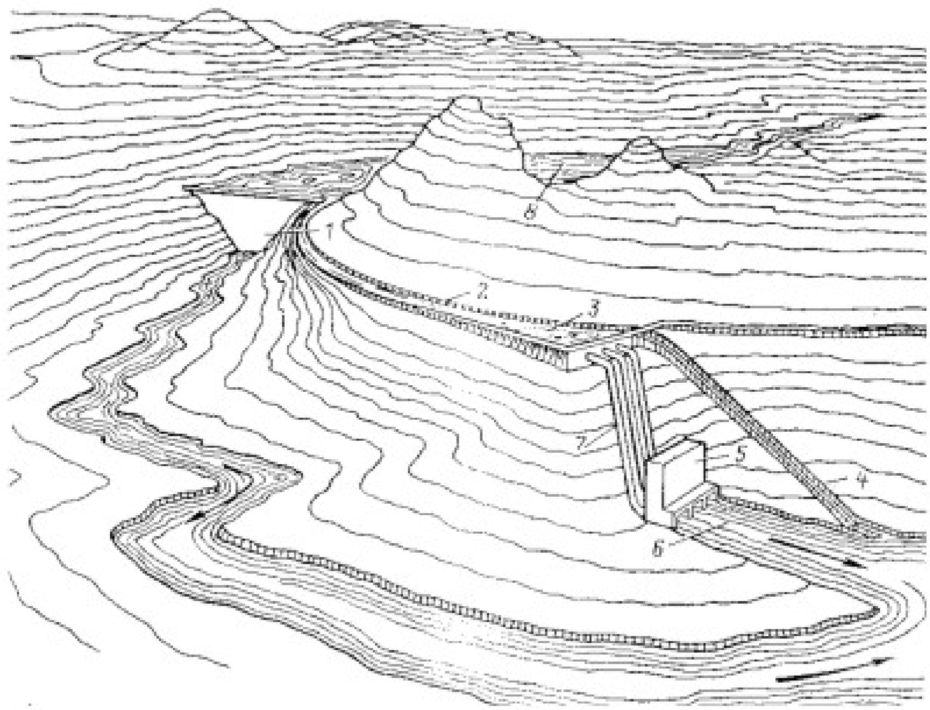


FIGURE 18 HYDROELECTRIC PROJECT BASED ON OPEN FLOW DIVERSION SCHEME
1-DAM .2-INTAKE DIVERSION CONDUIT. 3-HEAD POND. 4- SPILLWAY . 5- POWER HOUSE.
6- TAILVACE. 7-PENSTOCKS. 8-RESERWAIR

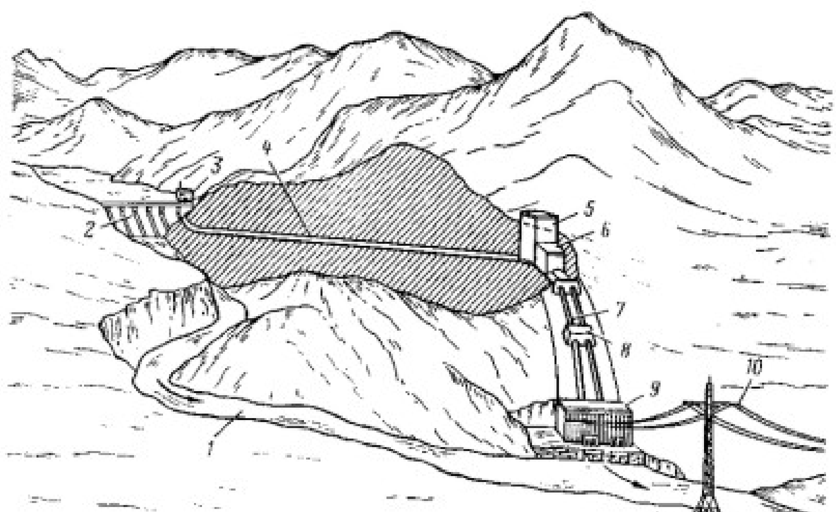


FIGURE 19. HYDROELECTRIC PROJECT USING A PRESSURE DIVERSION SYSTEM
1-WATERCOURSE. 2-DAM. 3- INTAKE STRUCTURES. 4-DIVERSION TUNNEL. 5- SURGE TANK. 6-PENSTOCK FORK HOUSE
7-PENSTOCKS.8- PENSTOCKS SUPPORT.9- POWER HOUSE. 10- POWER LINE

In fact, the combination of open channel and pressure conduit and penstock may be done in a variety of ways shown in Figure 20.

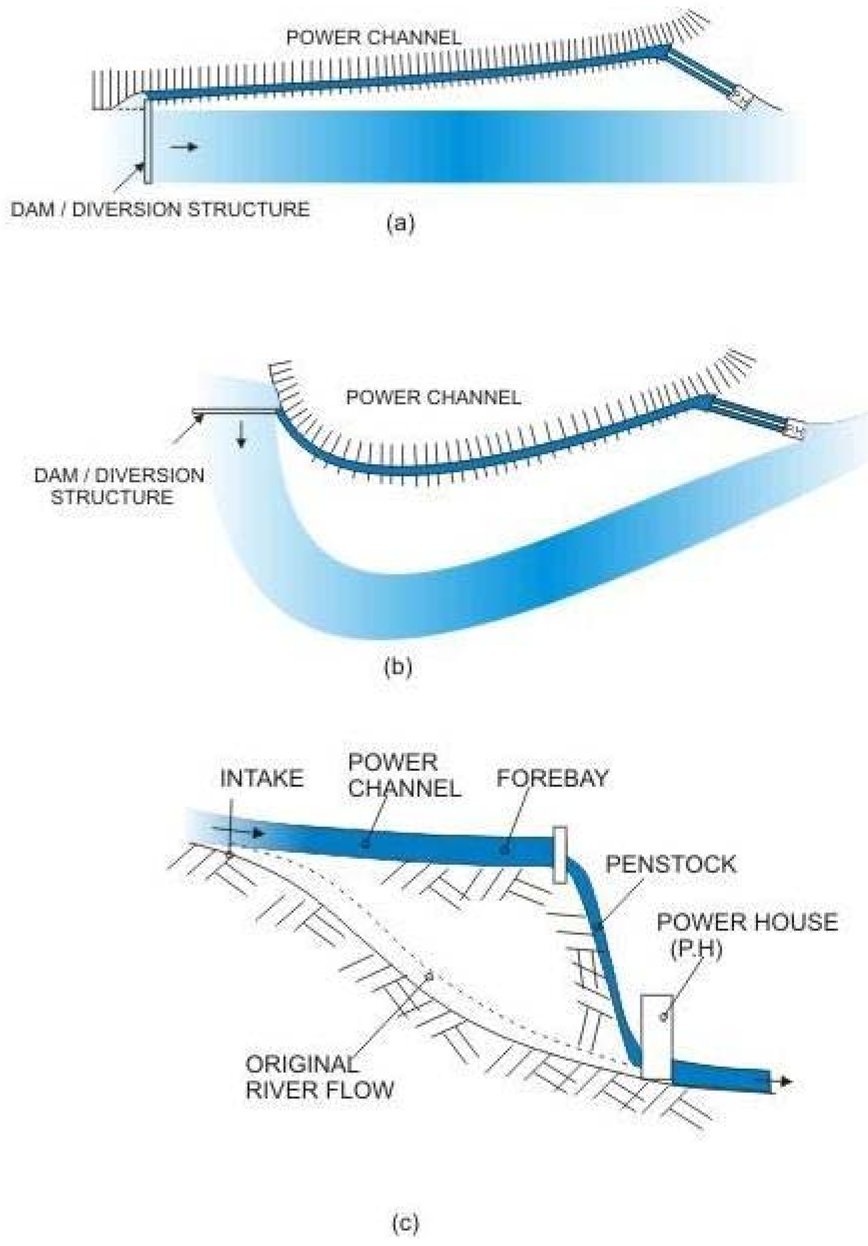
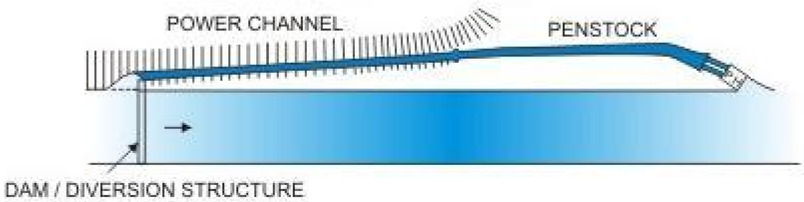
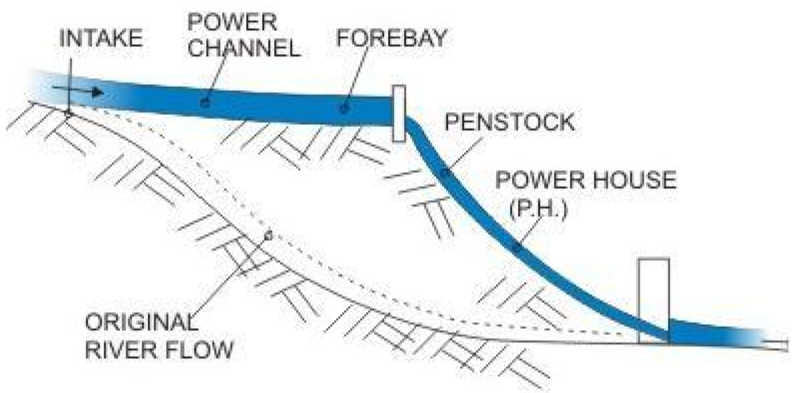


FIGURE 20. DIVERSION HYDRO POWER PROJECT BASED ON OPEN CHANNEL AND PRESSURE FLOW SYSTEMS
(a) LONG CANAL AND SHORT SURFACE PENSTOCK ALONG STRAIGHT RIVER REACH
(b) SAME AS (a) BUT IN A CURVED RIVER REACH
(c) SECTIONAL VIEW ALONG WATER CONDUCTING SYSTEM FOR (a) AND (b)

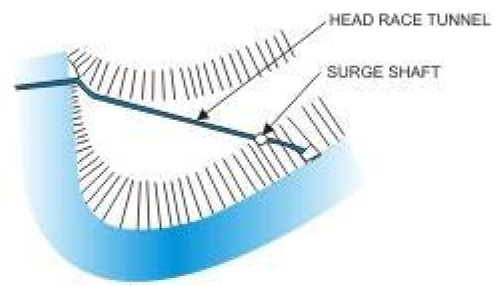


(d)

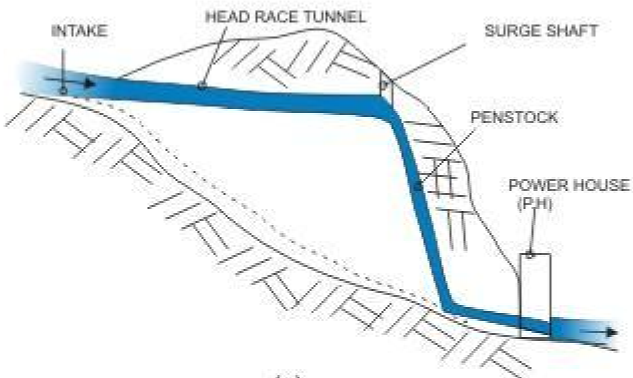


(e)

FIGURE 20. DIVERSION HYDRO POWER PROJECT BASED ON OPEN CHANNEL AND PRESSURE FLOW SYSTEMS
(d) SHORT CANAL AND LONG SURFACE PENSTOCK
(e) SECTIONAL VIEW FOR (d)



(f)



(g)

FIGURE 20. DIVERSION HYDRO POWER PROJECT BASED ON OPEN CHANNEL AND PRESSURE FLOW SYSTEMS
(f) HEAD RACE TUNNEL AND PENSTOCK IN A CURVED RIVER REACH
(g) SECTIONAL VIEW FOR (f)

There could be totally underground projects which consist of only pressure system of water conveyance. A variety of such projects are shown in Figure 21. This type of project layout may be termed as underground diversion schemes where even the power house is built underground.

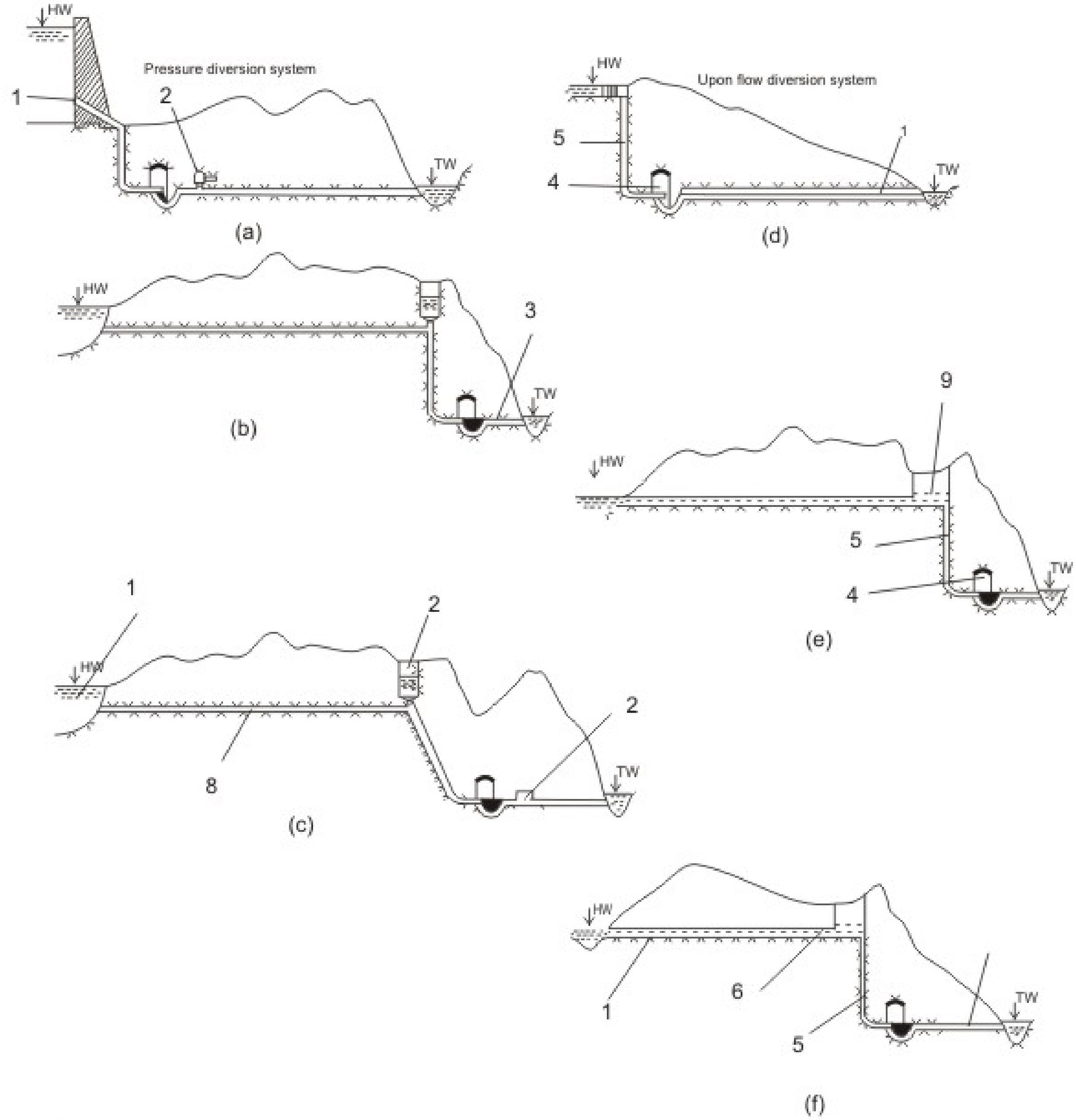


FIGURE 21. Underground project with
(a),(b) and (c) pressure diversion system, and (d),(e) and (f) open flow diversion system;
1-intake structure; 2- surge tank; 3-tailrace pressure tunnel; 4- power house; 5-penstock
6-intake open flow tunnel; 7-tailrace open flow tunnel ;8-intake pressure tunnel; 9--head pond

Low head development

Here too, two types of layouts may be possible:

- In-stream scheme
- Diversion scheme

In the in-stream type of project, the powerhouse would be built as a part of the diversion structure, as shown in Figure 2(a) or a general detailed view in Figure 6. A typical layout of such a power house and its cross section is shown in Figure 22.

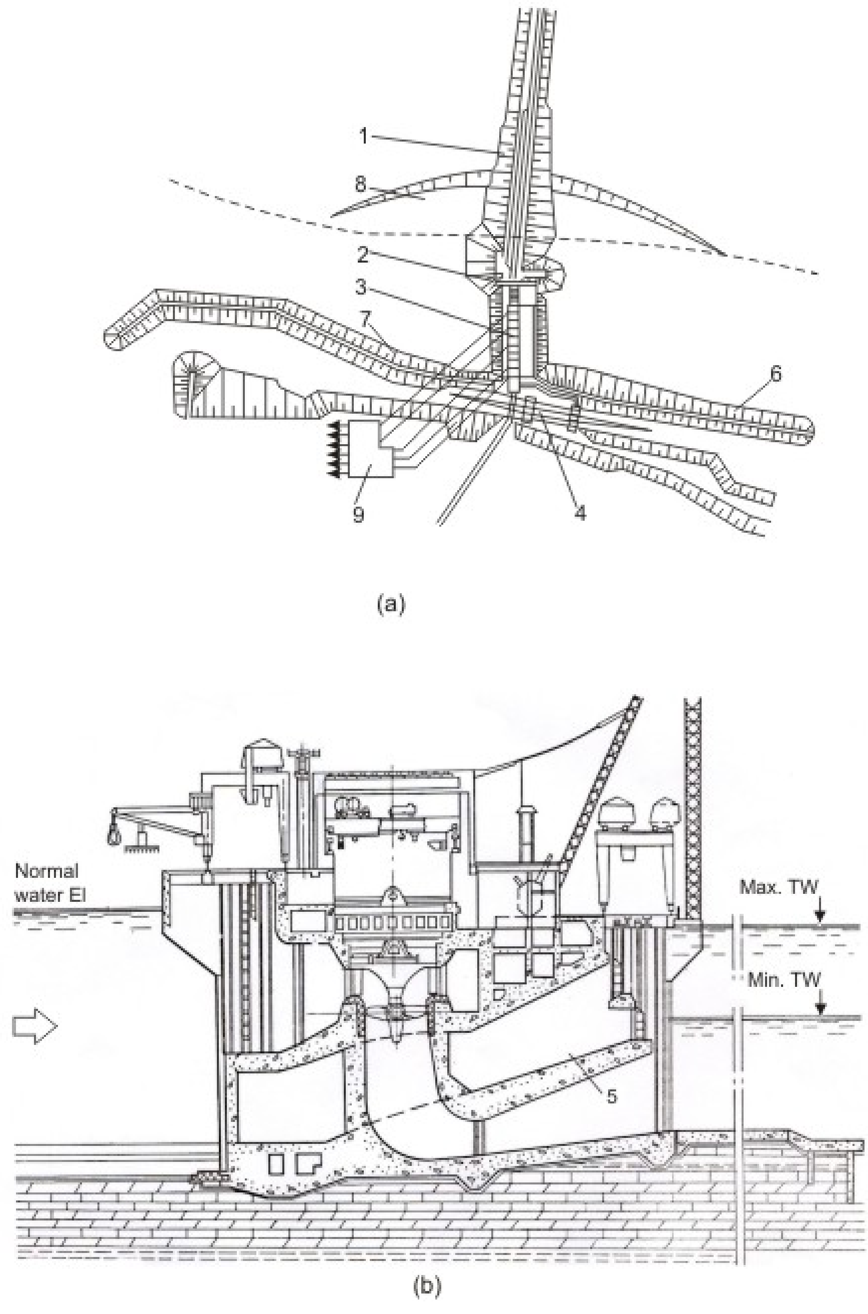


FIGURE 22. A TYPICAL LOW-HEAD IN STREAM POWER HOUSE
(a) plane ;(b) sectional elevation of the power house; 1-earth dam.2-over flow dam
3- power house; 4-lock;5-spillways in power house; 6- navigable canal dike downstream of dam;
7-output dike; 8- left bank clearing; 9- electrical switch yard

In the diversion type of scheme, there has to be a diversion structure as well as a diversion canal, as shown in Figure 2(b). The power house may be located at some convenient point of the canal, that is, at its upstream end, middle, or at the downstream end. The location of the power house depends upon conditions such as hydrological, topographical, geological, environmental economic conditions. The ground water table has to be taken into account.

Position of power houses

As might have been noticed from the layouts, there could be a variety of position for the power house with respect to natural ground level.

IS: 4410(Part10)-1988 differentiates between the following types of power stations, which may be constructed as per site conditions:

1. Surface power station or over ground power station: A power station which is constructed over the ground with necessary open excavation for foundations. Typical examples may be seen from Figs. 7, 11 or 12.
2. Underground power station: A power station located in a cavity in the ground with no part of the structure exposed to outside. A typical example of this type is shown in Figure 23.

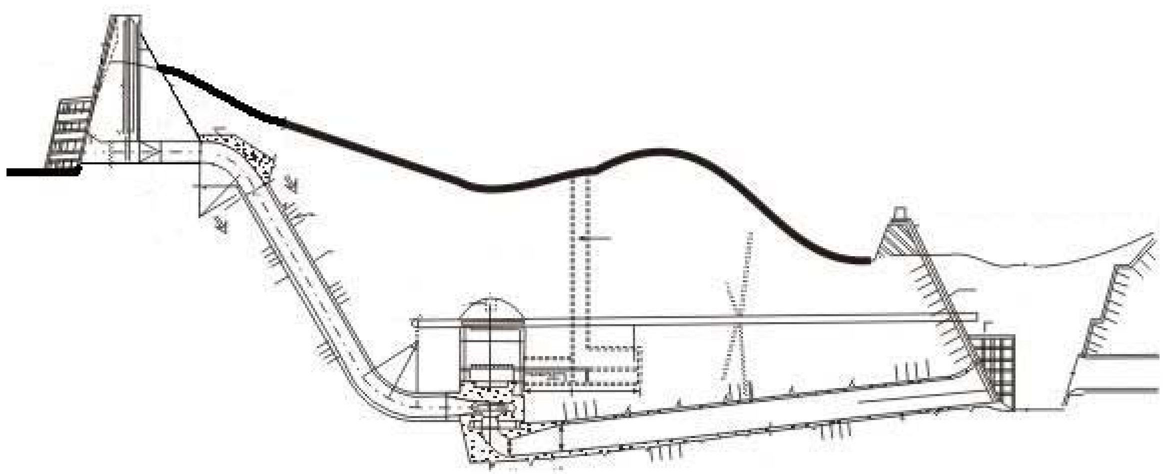


FIGURE 23 Underground power house of Sardar Sarovar Dam project

3. Semi-underground power station: A power station located partly below the ground level and followed by a tail race.

5.1.4 Electrical terms associated with hydropower engineering

Electrical power generated or consumed by any source is usually measured in units of Kilowatt-hour (kWh). The power generated by hydropower plants are normally connected to the national power grid from which the various withdrawals are made at different places, for different purposes. The national power grid also obtains power generated by the non-hydropower generating units like thermal, nuclear, etc. The power consumed at various points from the grid is usually termed as electrical load expressed in Kilo-Watt (KW) or Mega-Watt (MW). The load of a city varies throughout the day and at certain time reaches the highest value (usually in the evening for most Indian cities), called the Peak load or Peak demand. The load for a day at a point of the national power grid may be plotted with time to represent what is known as Daily Load Curve. Some other terms associated with hydropower engineering are as follows:

Load factor

This is the ratio of average load over a certain time period and the maximum load during that time. The period of time could be a day, a week, a month or a year. For example, the daily load factor is the ratio of the average load may be obtained by calculating the total energy consumed during 24 hours (finding the area below the load vs. time graph) and then divided by 24. Load factor is usually expressed as a percentage

Installed capacity

For a hydro electric plant, this is the total capacity of all the generating units installed in the power station. However all the units may not run together for all the time.

Capacity factor

This is the ratio of the average output of the hydroelectric plant for a given period of time to the plant installed capacity. The average output of a plant may be obtained for any time period, like a day, a week, a month or a year. The daily average output may be obtained by calculating the total energy produced during 24hours divided by 24. For a hydroelectric plant, the capacity factor normally varies between 0.25 and 0.75.

Utilization factor

Throughout the day or any given time period, a hydroelectric plant power production goes on varying, depending upon the demand in the power grid and the power necessary to be produced to balance it. The maximum production during the time divided by the installed capacity gives the utilization factor for the plant during that time. The value of utilization factor usually varies from 0.4 to 0.9 for a hydroelectric plant depending upon the plant installed capacity, load factor and storage.

Firm (primary) power

This is the amount of power that is the minimum produced by a hydro-power plant during a certain period of time. It depends upon whether storage is available or not for the plant since a plant without storage like run-of-river plants would produce power as per the minimum stream flow. For a plant with storage, the minimum power produced is likely to be more since some of the stored water would also be used for power generation when there is low flow in the river.

Secondary Power

This is the power produced by a hydropower plant over and above the firm power.

References

Mosonyi, Emil (1991) “Water power development”, Akademia Budapest.

Module
5
HYDROPOWER ENGINEERING

LESSON 2

HYDROPOWER WATER CONVEYANCE SYSTEM

As indicated in Lesson 5.1 a dam or diversion structure like a barrage obstructs the flow of a river and creates a potential head which is utilized by allowing the water to flow through the water conducting system upto the turbines driving the generators and then allowing it to discharge into the river downstream. Right from the intake of the water conducting system, where water enters from the main river, up to the outlet where water discharges off back into the river again, different structural arrangements are provided to fulfil certain objectives, the important ones being as follows:

1. The water inflowing into the conveyance system should be free from undesirable material, as far as possible, that may likely damage the turbines or the water conducting system itself.
2. The energy of the inflowing water may be preserved, as far as possible, throughout the water course so that the turbine-generator system may extract the maximum possible energy out of the flowing water.

As an example of the first case, it may be cited that in hilly rivers, there are good chances of sand, gravel, and even boulders getting into the water conducting system along with the flowing water. The bigger particles may choke the system whereas the smaller ones may erode the turbine blades by abrasive action. Apart from these, floating materials like trees or dead animals and in some projects in the higher altitudes ice blocks may get sucked into the system which may clog the turbine runners.

The main components of a water conveyance system consists of the following:

1. Intake structure
2. Water conducting system comprising of different structures
3. Outflow structure, which is usually a part of the turbine tail end

The water conducting system, again, may be of two types

1. Open channel flow system
2. Pressure flow system

In the pressure flow system, there could be further classification into the two types, as:

1. Low-pressure conduits and tunnels
2. High-pressure conduits, commonly called the penstocks

In either of the above cases, some provision is usually made to prevent the undesirable effects of a power rejection in the generator that may cause the turbine to spin exceedingly fast, resulting in a closure of the valves controlling the flow of water at the turbine end. If the closure is relatively fast, high pressures may develop in pressured systems conducting water to the turbine. For open channel systems, this may lead to generation of surges in the water surface which may even cause spillage of the channel banks if adequate freeboard is not provided.

This chapter discusses the important issues related to the different components of a hydropower Water Conveyance System.

5.2.1 Intakes

An intake is provided at the mouth of a water conveyance system for a hydropower project. It is designed such that the following points are complied, as far as possible:

1. There should be minimum head loss as water enters from the reservoir behind a dam or the pool behind a barrage into the water conducting system.
2. There should not be any formation of vortices that could draw air into the water conducting system.
3. There should be minimum entry of sediment into the water conducting system.
4. Floating material should not enter the water conducting system.

The position and location of an intake in a hydropower project would generally depend upon the type of hydropower development, that is, whether the project is of run-of-river type or storage type. For each one of these hydropower projects, there are a few different types, the important ones of which are explained in the following paragraphs.

Run-of-river type intake

- Intakes adjacent to a diversion structure like a barrage. Here, an intake for a tunnel is placed upstream of the diversion structure to draw water from the pool (Figure 1). For a canal intake (Figure 2), the head regulator resembles that of an irrigation canal intake. It may be observed from Figure 3 that the canal conveying water, also called the power canal, leads to a Forebay before leading to the turbine unit. The exit passage from the turbines is called the Tail Race Channel. There is also a Bye-Pass Channel to release water when the turbines shut down suddenly.

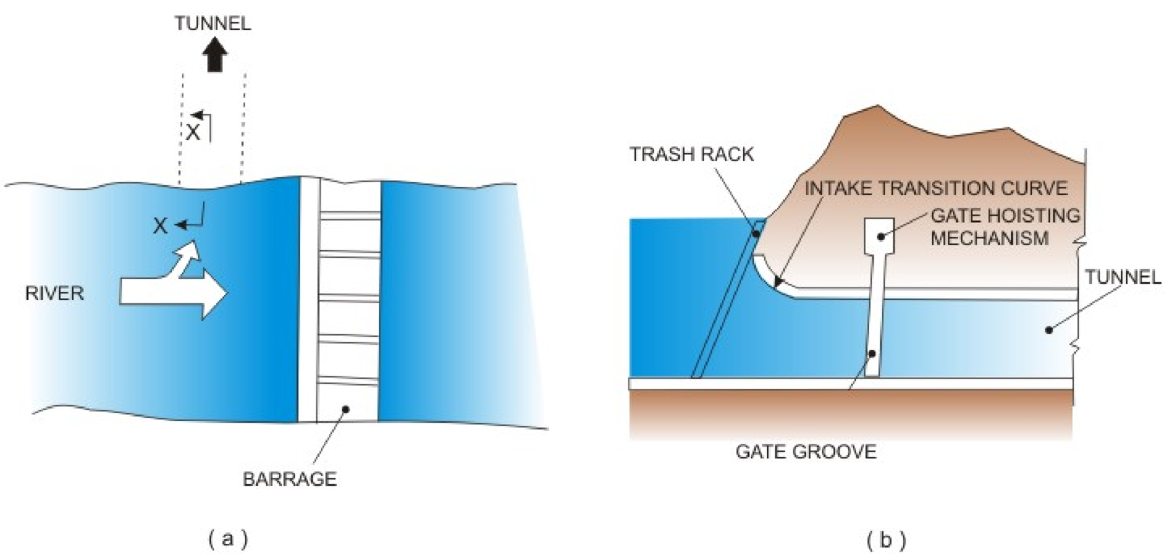


FIGURE 1. Intake adjacent to a barrage leading to a tunnel
(a) Plan , (b) Section X-X

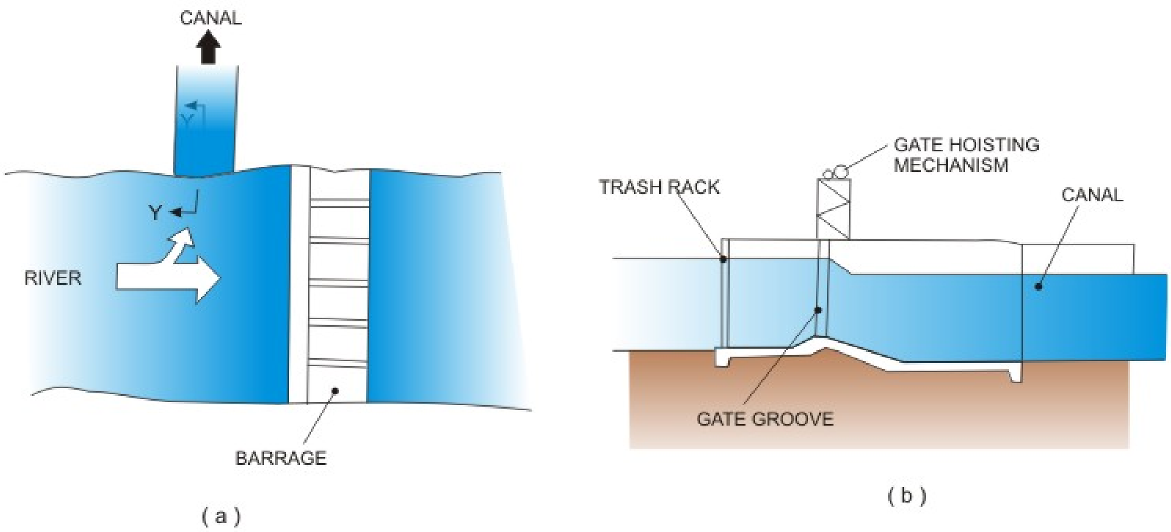


FIGURE 2. Intake adjacent to a barrage leading to a canal (a) Plan (b) Section through head regulator

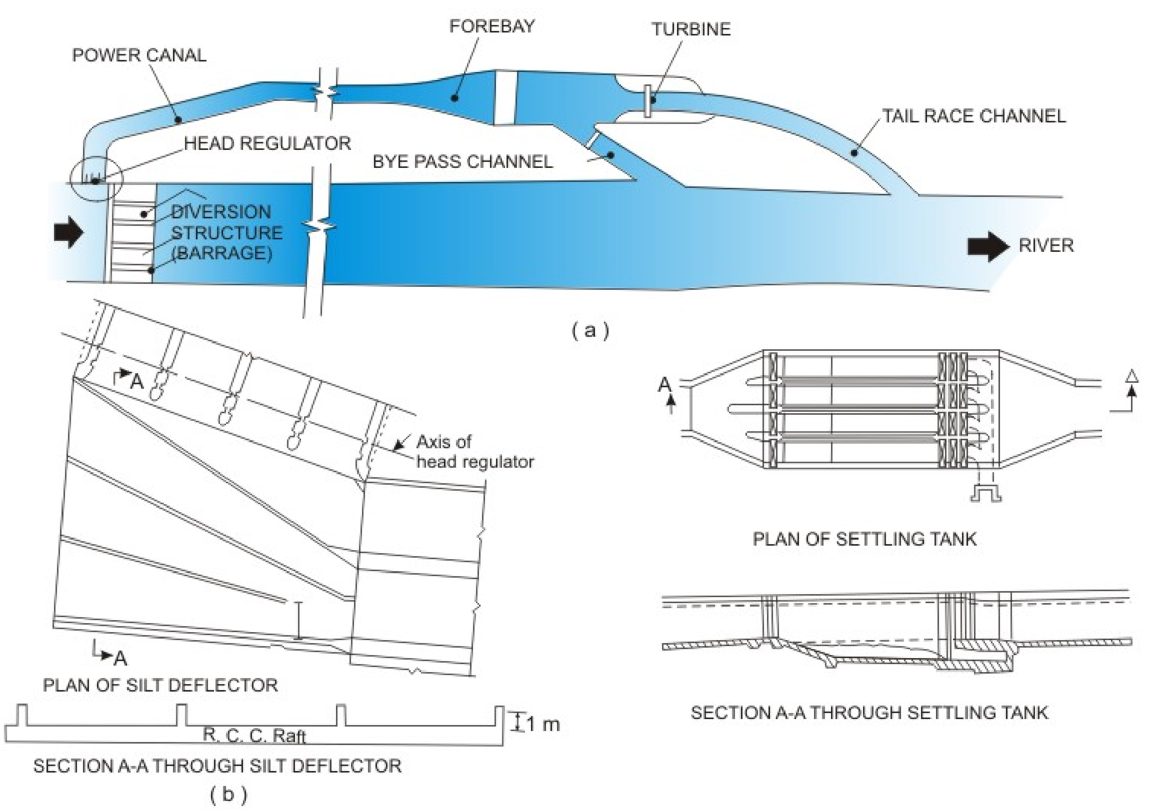


FIGURE 3. (a) Power canal leading to fore-bay at head of turbine unit, (b) Detail 'A' of head regulator for canals in hilly region equipped with silt deflector for preventing boulder entry & settling tank to remove sediment that too entered the canal.

- Intakes for in-stream power house. These are used for powerhouses located across rivers or canals to utilize the head difference across a canal drop. Here, the intake length is kept quite short and leads to either a vertical axis Kaplan turbine or a horizontal axis bulb turbine (Figure 4).

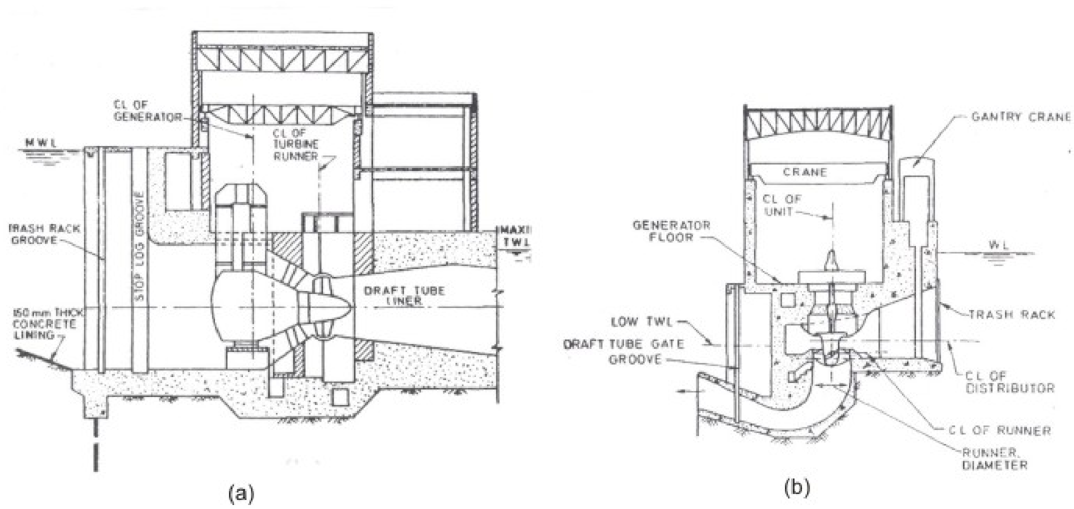


FIGURE 4. Intakes for river or canal fall power house integrated with turbine unit. (a) Kaplan turbines, (b) Bulb turbines.

Reservoir type intakes

- Intakes for concrete dams are located on the upstream face of the dam as shown in Figure 5. The face of the intake is rectangular and is reduced to a smaller rectangular section through a transitory shape known as the bell-mouth. From the smaller rectangular section, another transition is provided to change the shape to circular.

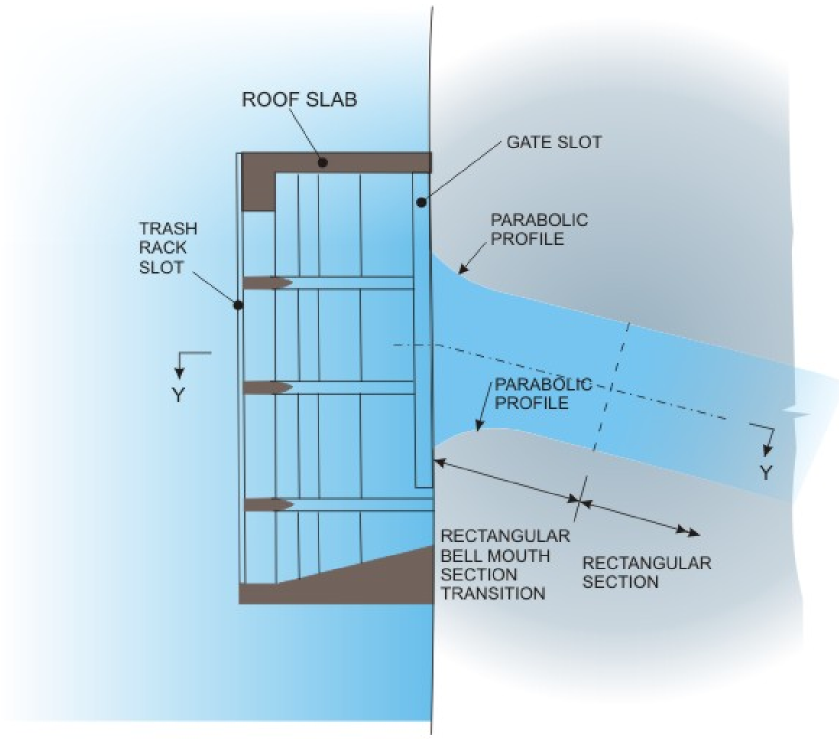


FIGURE 5. Intake from a reservoir upstream of a storage dam
(a) Sectional elevation X - X.

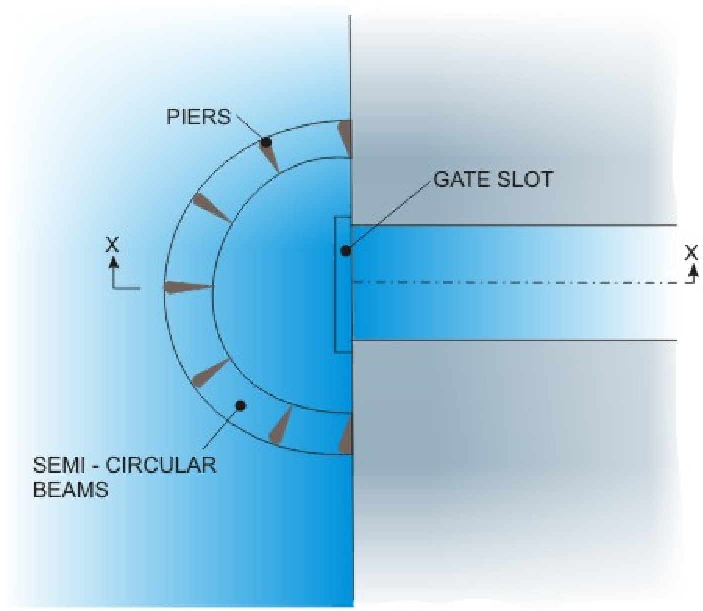
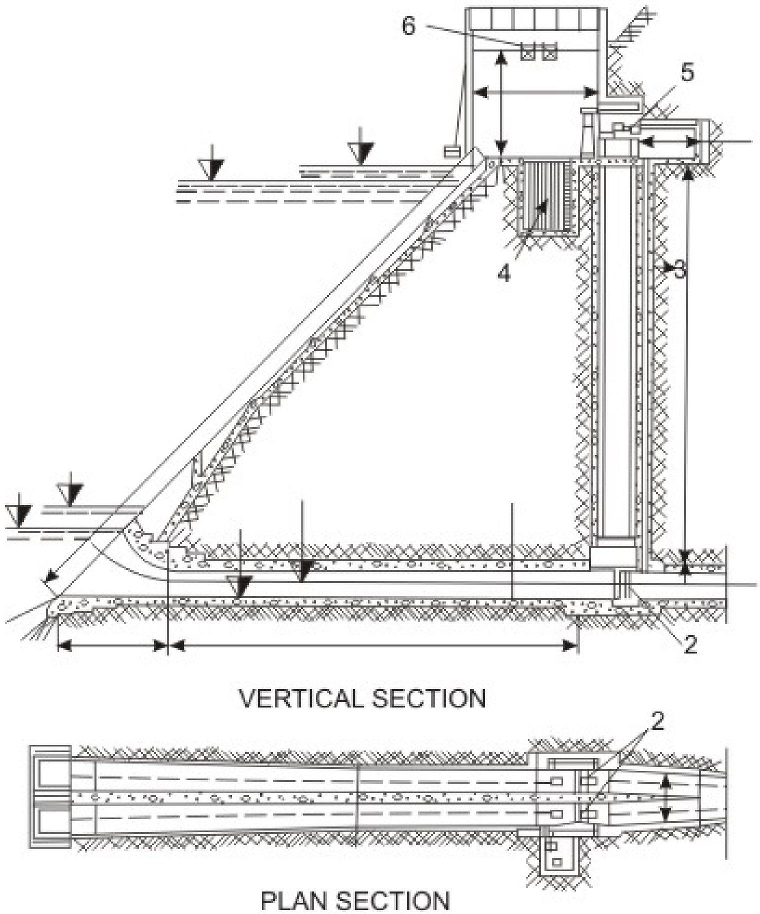


FIGURE 5. Intake from a reservoir upstream of a storage dam
(b) Sectional plan Y - Y.

- Intakes for embankment dams are usually in the form of a conduit, which is laid below the dam and whose intake face is inclined (Figure 6) or are provided in the form of a tower (Figures 7 and 8). A tower type intake is constructed where there is a wide variation of the water level in the reservoir.



I - Trashrack ; 2 - Membrane valve ; 3 - Aeration pipe ; 4 - Gate room ; 5 - Winch ; 6 - Crane

FIGURE 6. Sloping intake for an embankment dam

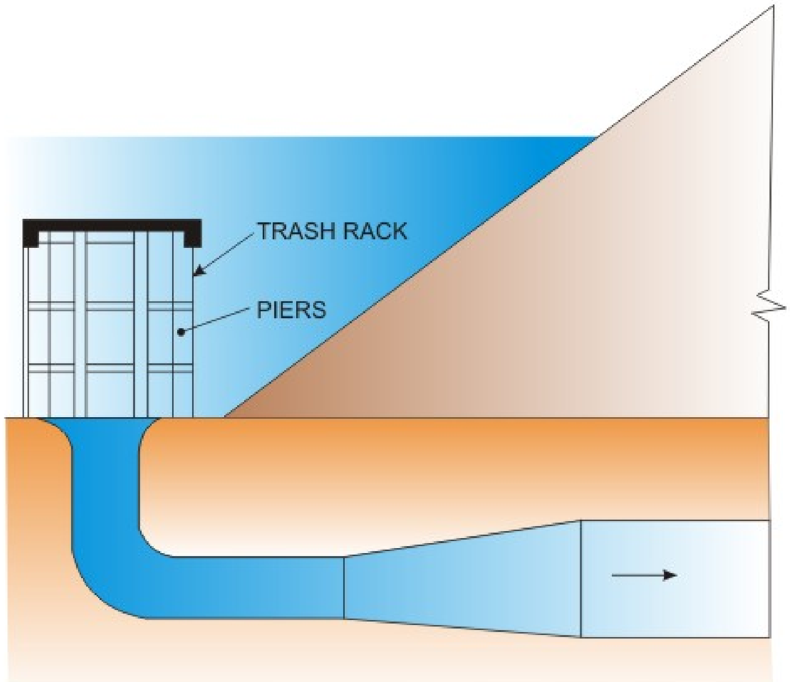


FIGURE 7. Tower type intake for an embarkment dam with flow control by a vertical lift or radial gate in conduit

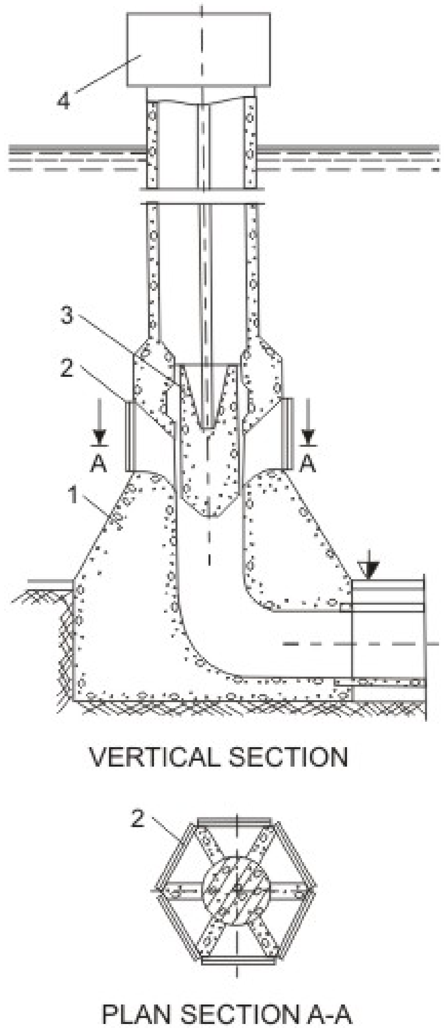


FIGURE 8. Tower type intake with cylindrical gate for flow control

- Intakes which have pressure tunnels off-taking from a storage reservoir and where the intake is located at a distance from the dam, say through the abutments, then the intake structure of such layout may be of inclined type or tower type as was provided in conjunction with the dam itself.

The choice and location of the intake structure depends upon the following factors.

a) Type of development, that is, run-of-the-river or storage dam project;
b) Location of power house vis-à-vis the dam ;
c) Type of water conductor system, that is, tunnel, canal or penstock;
d) Topographical features of area;
e) In cases where there is a considerable movement of boulders, stones and sand in the downstream direction, the intake should be arranged so that the effect of such movement will not lead to a partial restriction or blockage of the intake. In respect of storage reservoir intakes the sill level of the intake should be aimed to be kept above the sedimentation level at or near the dam face arrived at; and

f) The intake can often be located so as to enable it to be constructed before the level of the reservoir is raised.

Detail about the design of hydropower intakes may be obtained from the Bureau of Indian Standards code IS: 9761-1995 “Hydropower intakes-criteria for hydraulic design”.

In all the above intakes it may be noticed that a Trash Rack Structure is provided at the entry. A trash rack is actually a grill or a screen for preventing entry of suspended or floating material into the water conducting system. It is made usually of metallic strips welded in vertical and horizontal directions at regular spacings.

5.2.2 Water Conducting System

After flowing through the intake structure, the water must pass through the water conveyance system may be either of closed conduit type, as shown in Figure1 (tunnel off-taking from upstream of the river diversion) or could be open-channels as shown in Figure 2. High pressure intakes, for example as in the entry to penstocks (Figure 9) would be either reinforced concrete lined or steel lined. In this section we discuss the various types of water conducting passages.

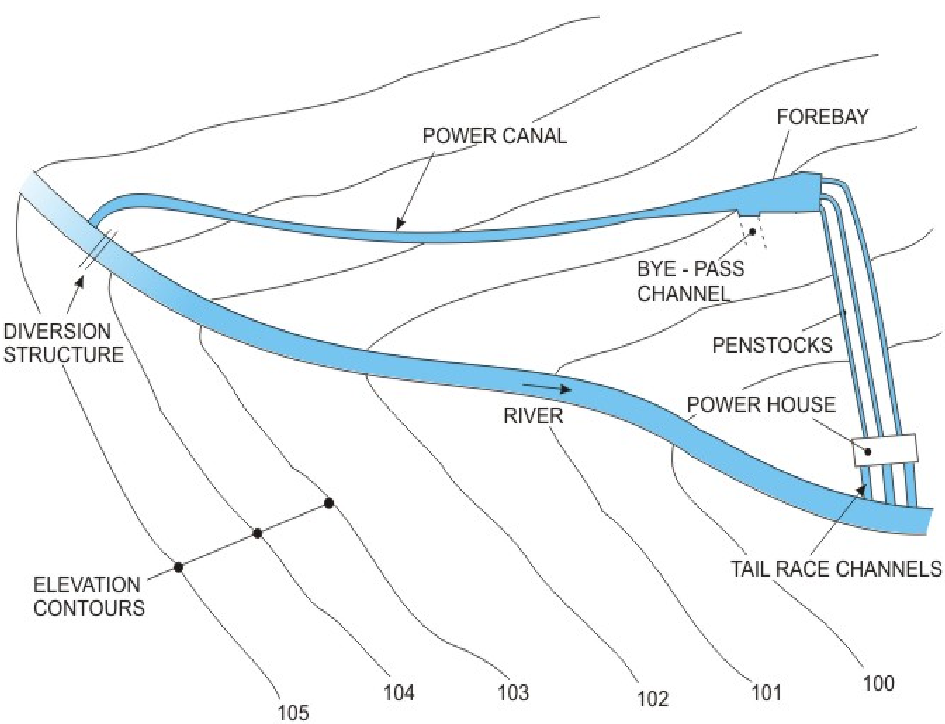


FIGURE 9. Alignment of a power canal along a hill slope

Open channels

These are usually lined canals to reduce water loss through seepage as well as to minimize friction loss. The design of canals for hydropower water conveyance follows the same rules as for rigid bed irrigation channels, and are usually termed as power canals.

A power canal that offtakes from a diversion structure (Figure2c) has to flow along the hill slope as may be observed from the alignment shown in Figure 9. A cross section of the canal would show that there would usually be high ground on one bank and falling ground on the other (Figure 10). It is important to stabilize the uphill cut-slope with some kind of protection in order to prevent fallout of loose blocks of stone into the canal. Some stretch of the canal could also be such that the bank with low ground needs to be supplemented with an artificially created embankment (Figure11). As observed from Figure 9, a power canal ends at a forebay, which is broadened to act as a small reservoir. From the forebay, intakes direct the water into the penstocks. There usually is a bye-pass channel which acts as a spillway to pass on excess water in case of a valve closure in the turbine of the hydropower generating unit. If such an escape channel is not provided, there are chances that under sudden closure of the valves of the turbines, surge waves move up the power canal. Hence, sufficient free board has to be provided for the canals.

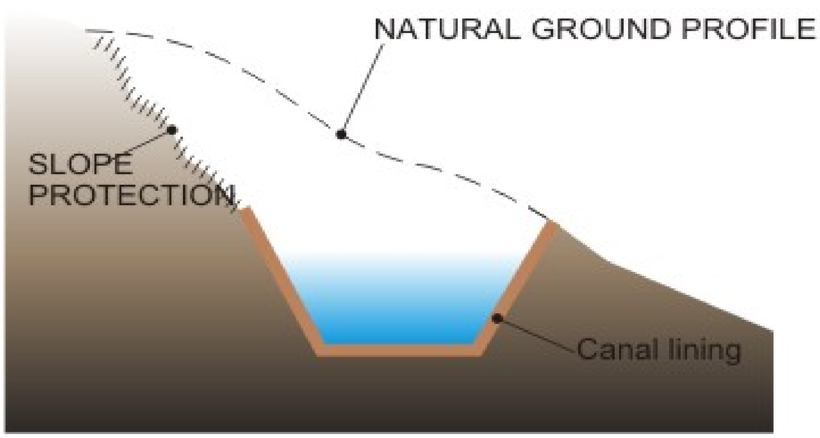


FIGURE 10. Cross section of a power canal in cutting.

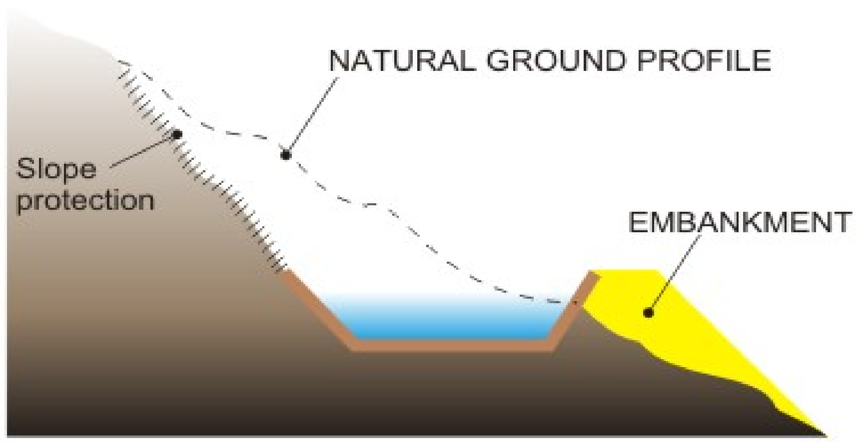


FIGURE 11. Cross section of a power canal in partly cutting & partly filling.

Tunnels

As shown in Figure1, a river diversion structure may also direct water into a tunnel. A typical section through a tunnel is shown in Figure 12. The initial portion of the tunnel from the intake upto the Surge-Tank is termed as the Head Race Tunnel (HRT) and beyond that it houses the penstock or steel-conduits, which sustains a larger pressure than the HRT. The HRT may either be unlined (in case of quite good quality rocks) or may be lined with concrete. The surge tank is provided to absorb any surge of water that could be generated during a sudden closure of valve at the turbine end. Normally, the water level in the surge tank would be marginally lower than that at the intake (see Figure 12) and the difference of levels depends upon the friction loss in the HRT. Thus, when the HRT runs full, it is subjected to a much low pressure compared to the penstock. If a HRT is concrete lined, the reinforcement in the concrete may be nominal as the lining is only to assist in preventing fallout of rock blocks into the tunnel. However, if the rock mass above the tunnel is very weak, then the tunnel lining may have to support a larger rock weight, in which case the reinforcement has to be designed accordingly. A tunnel should also be designed for the empty condition, assuming the outside rock to be saturated with water.

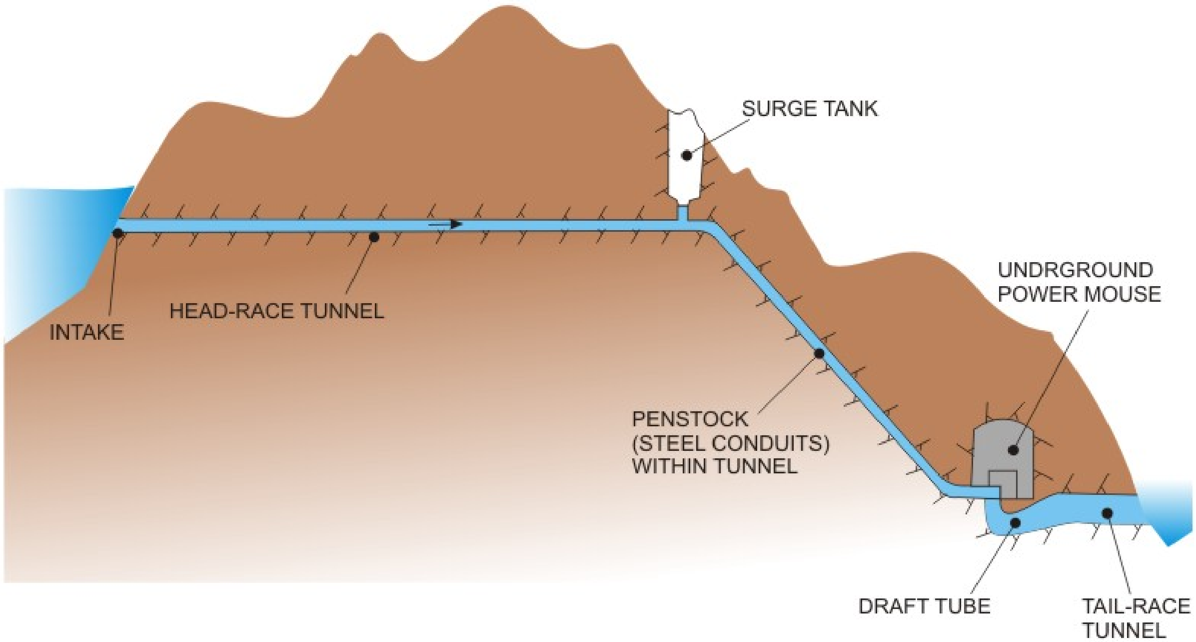


FIGURE 12. Section through a typical tunnel development for water conveyance of a hydropower system.

These aspects pertain to the structural design of a tunnel. Apart from this, there has to be a geometric design, finalising the shape of a tunnel. Section 5.2.3 discuss these issues of tunnel design.

Surge tanks

As explained, a surge tank (or surge chamber) is a device introduced within a hydropower water conveyance system having a rather long pressure conduit to absorb the excess pressure rise in case of a sudden valve closure. It also acts as a small storage from which water may be supplied in case of a sudden valve opening of the turbine. In case of a sudden opening of turbine valve, there are chances of penstock collapse due to a negative pressure generation. If there is no surge tank.

There are different types of surge tanks that are possible to be installed. The Bureau of Indian Standards code IS: 7396(Part1)-1985 "Criteria for hydraulic design of surge tanks" describes the most common types of surge tanks which are as follows:

1. Simple Surge Tank: A simple surge tank is a shaft connected to pressure tunnel directly or by a short connection of cross-sectional area not less than the area of the head race tunnel (Figure 13).

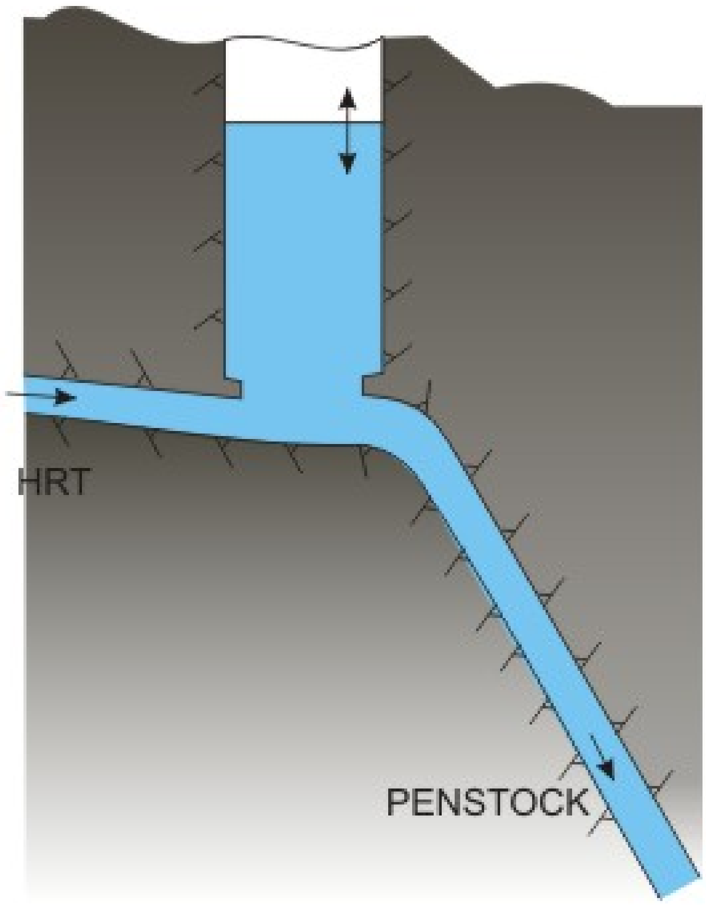


FIGURE 13. Simple surge tank

2. Restricted Orifice Surge Tank: A simple surge tank in which the inlet is throttled to improve damping of oscillations by offering greater resistance and connected to the head race tunnel with or without a connecting/communicating shaft (Figure 14).

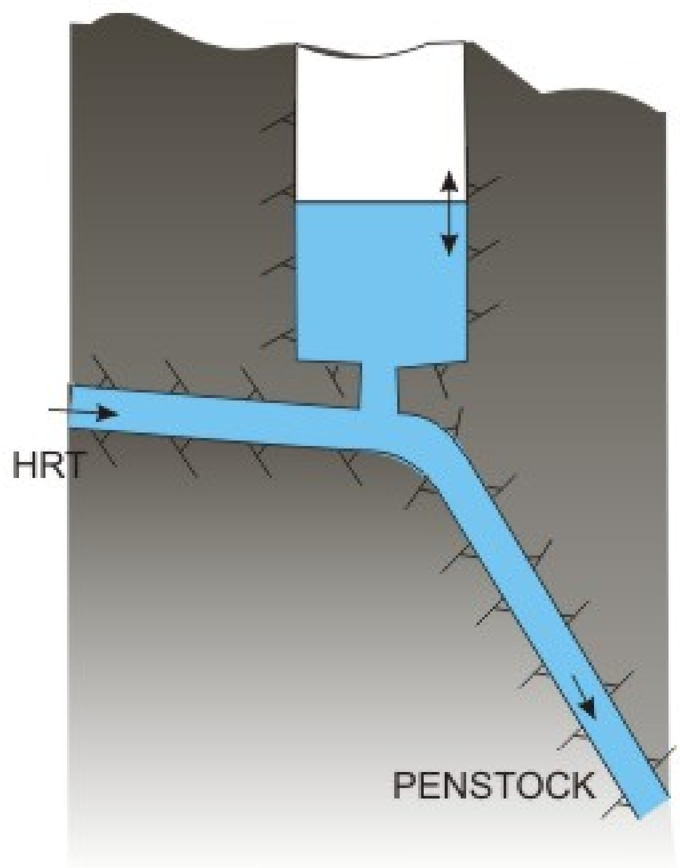


FIGURE 14. Restricted orifice surge tank

3. Differential Surge Tank: Differential Surge tank is a throttled surge tank with an addition of a riser pipe may be inside the main shaft, connected to main shaft by orifice or ports. The riser may also be arranged on one side of throttled shaft as shown in Figure 15. Port holes are generally at the bottom of the riser at the sides.

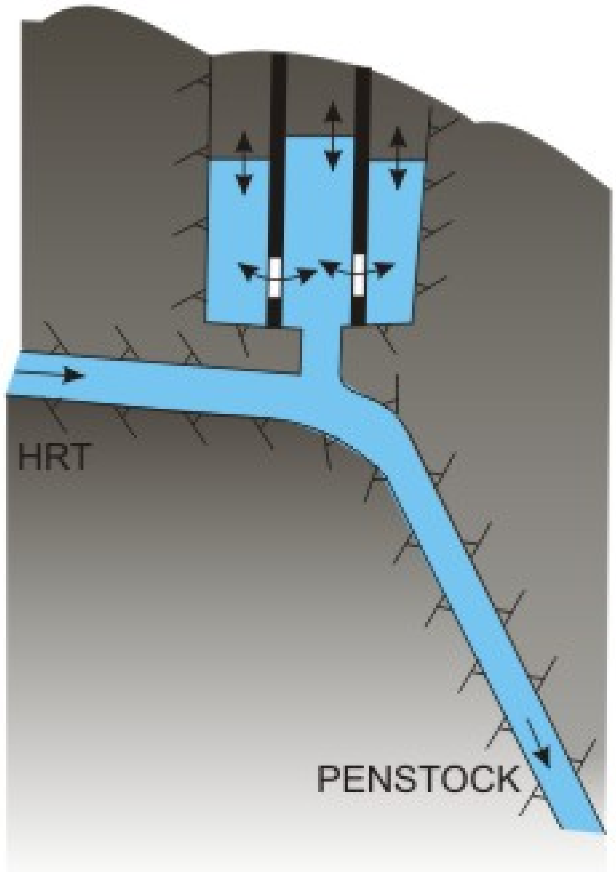


FIGURE 15. Differential surge tank

In an underground development of hydropower system, tail race surge tanks are usually provided to protect tail race tunnel from water hammer effect due to fluctuation in load. These are located downstream of turbines which discharge into long tail race tunnels under pressure. The necessity of tail race surge tank may be eliminated by ensuring free-flow conditions in the tunnel but in case of long tunnels this may become uneconomical than a surge tank.

The Bureau of Indian Standards code IS: 7396(Part2)-1985 deals with the different types of surge tank that may be provided in the Tail Race Tunnel (TRT). A typical view of a surge tank in a TRT is shown in Figure 16.

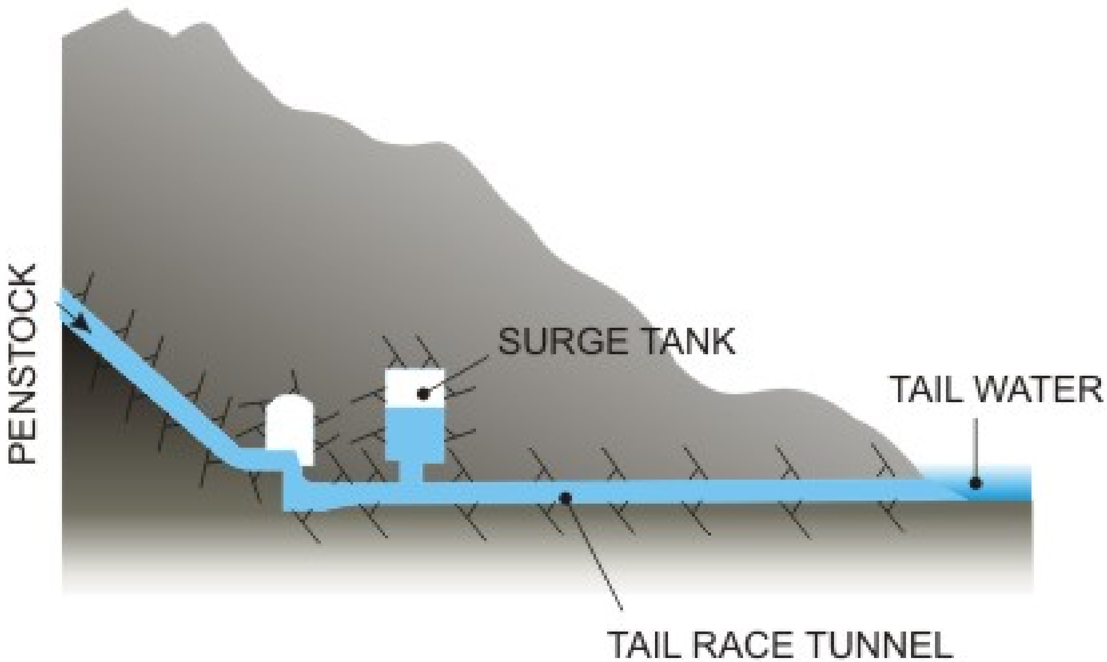
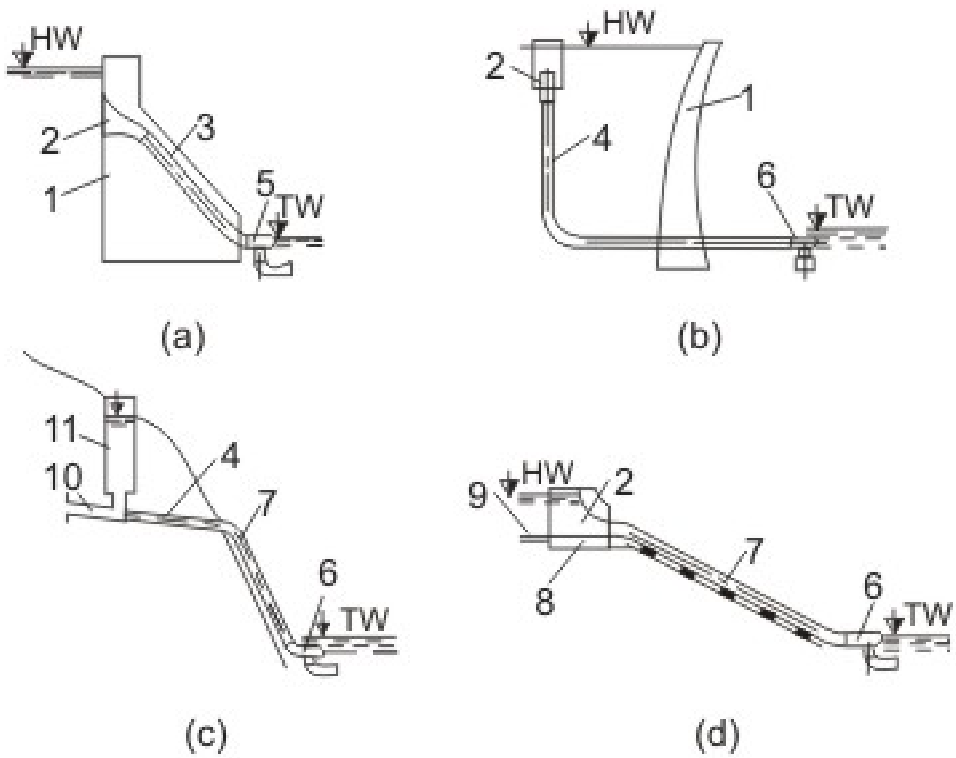


FIGURE 16. Surge tank in tail-race tunnel

Apart from the above types, there could be special types of surge tanks in multiple units which are discussed in IS: 7396 (Part3) and IS: 7396(Part 4) respectively.

Penstock

A penstock is a steel or reinforced concrete conduit to resist high pressure in the water conveyance system and may take off directly from behind a dam, from a forebay, or from the surge tank end of a head race tunnel as shown in Figure 17. Similar to a tunnel, a penstock needs to be designed for different types of loads. Further, they have to be equipped with different accessories, which may be different for overground or ground embedded types. These aspects of penstocks are thus discussed separately in Section 5.2.4.



1=Dam;2=Intake;3=Embedded or concrete-jacket penstock; 4=Tunnel penstock;5=Power house at dam toe;6=Isolated power house;7=Exposed penstock;8=Head pond;9=Diversion conduit;10=Diversion tunnel;11=Surge tank

FIGURE 17. Layout of penstocks.
a) penstocks of power house built at dam toe;
b) penstocks of isolated power house;
c) penstocks of pressure diversion project;
d) penstocks of open-flow diversion project

5.2.3 Tunnels

Tunnels need to be designed and constructed in an efficient manner for the best performance. The Bureau of Indian Standards code IS: 4880-1976 "Code of practice for design of tunnels conveying water" (Parts 1 to 4) provide guidelines for design of a tunnel under various situations. The following paragraphs provide the salient points from these codes.

Tunnel layout

The first aspect that needs to be decided for a tunnel is the alignment, that is, the route layout of the tunnel in plan. Figure 18 shows the possible alignment for the tunnel water conveyance system for a hydropower system using tunnel.

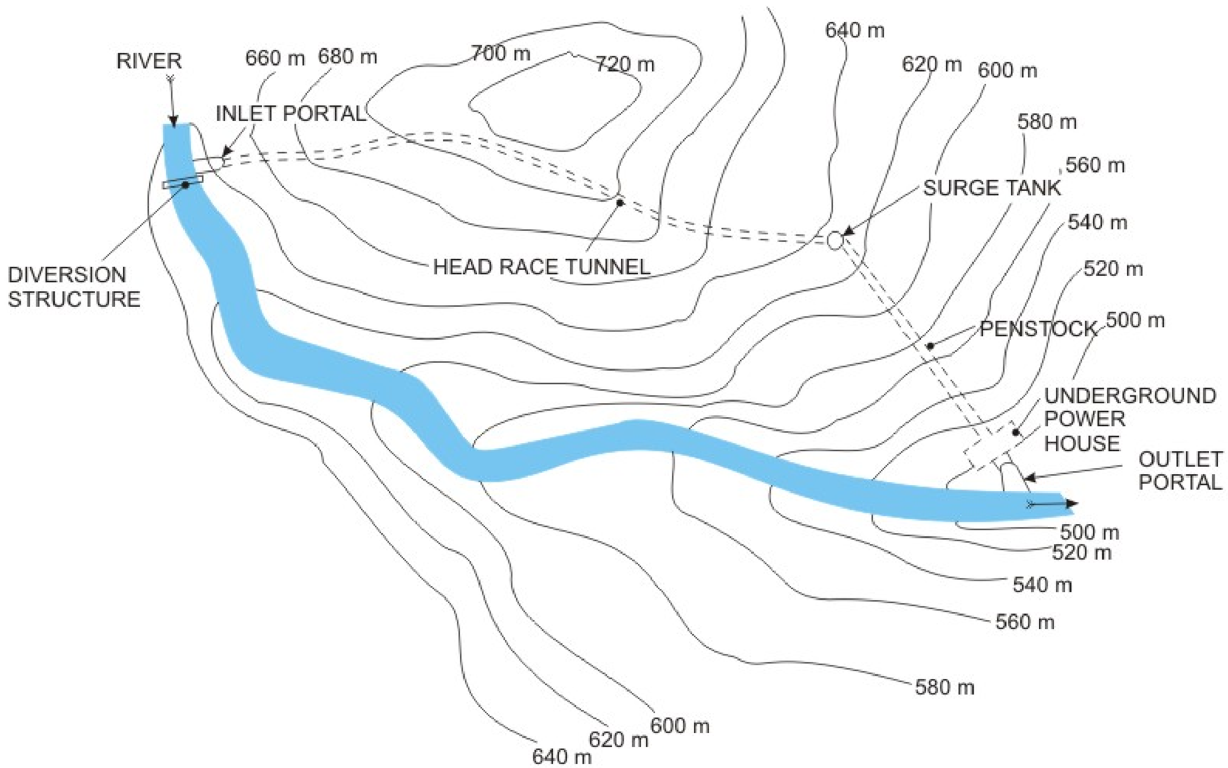


FIGURE 18. Typical layout of a tunnel alignment & other structures

The layout is usually governed by the geological features of the surrounding hills. Complicated geological conditions and extraordinary geological occurrences such as intra-thrust zones, very wide shear zones, geothermal zones of high temperature, cold/hot water springs, water charged rock masses, intrusions, fault planes, etc. should preferably be avoided. Sound, homogeneous isotropic and solid rock formations are the most suitable for tunneling work. However, in the Himalayan region, such conditions are rather rare compared to the hills of peninsular India. This is because the Himalayan geological formations are mostly sedimentary in nature whereas the peninsular upland of the country is of igneous nature. Hence, geological investigations have to be carried out in detail before a tunnel alignment is finalized.

Tunnel section

The second aspect requires the determination of the size and shape of the tunnel. The size or cross sectional area can be determined from the amount of water that is to be conveyed under the given head difference. Regarding shape, the following types are generally provided for hydropower tunnels:

1. Circular Section (Figure 19): The circular section is most suitable from structural considerations. However, it is difficult for excavation, particularly where cross-sectional area is small. For tunnels which are likely to resist heavy inward or outward radial pressures, it is desirable to adopt a circular section. In case where the tunnel is subjected to high internal pressure, but does not have good quality of rock and/or adequate rock cover around it, circular section is considered to be the most suitable.

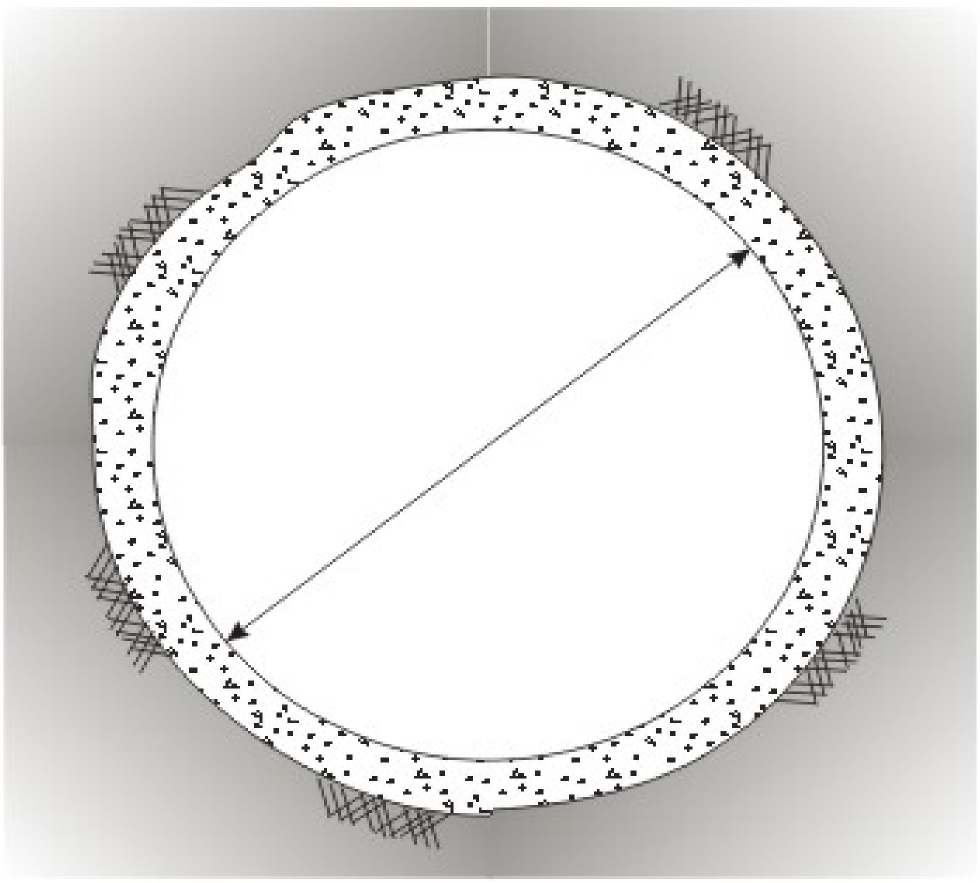
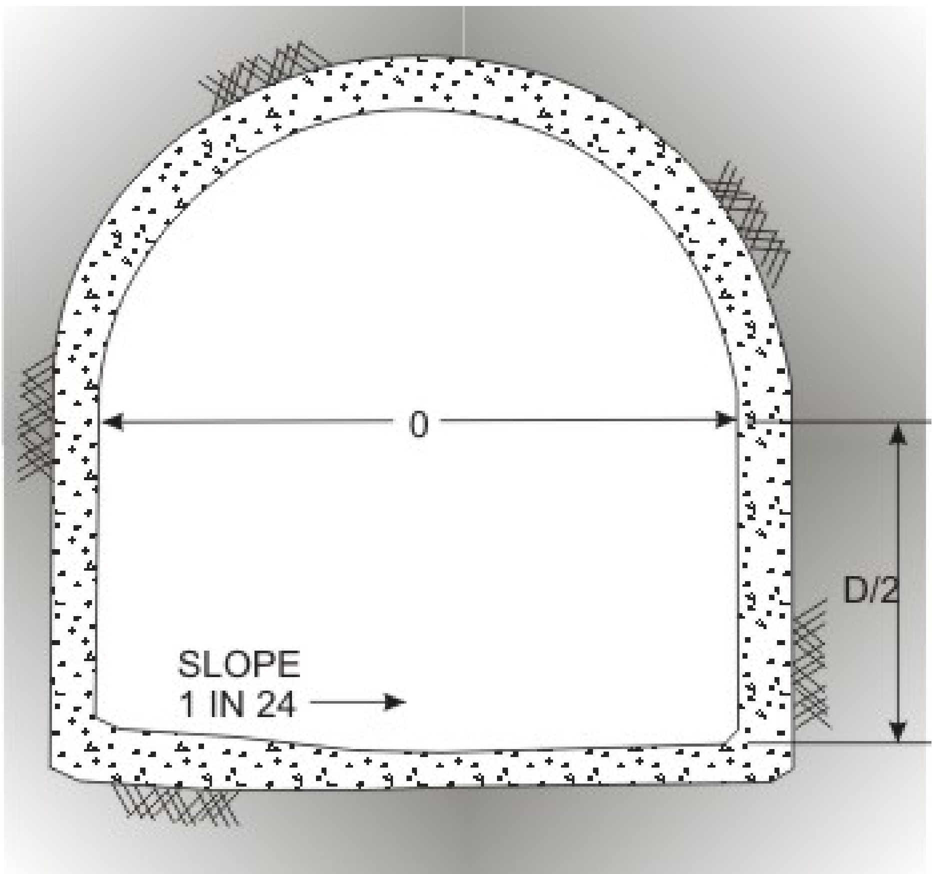


FIGURE 19. Circular shaped section of tunnel

2. D section (Figure 20): This type of section would be found suitable in tunnels located in massive igneous, hard, compacted, metamorphic and good quality sedimentary rocks where the external pressures due to water or unsound strata upon the lining is slight and also where the lining is not required to be designed against internal pressure. The principal advantages of this section over horse-shoe section (discussed in next paragraph) are the added width of the invert which gives more working floor space in the heading during driving and the flatter invert which helps to eliminate the tendency of wet concrete to slump and draw away from the tunnel sides after it has been cast.



D : Width of tunnel

FIGURE 20. D-shaped section of tunnel

3. Horse-Shoe and Modified Horse-Shoe Sections (Figure 21 a and b): These sections are a compromise between circular and D sections. These sections are strong in their resistance to external pressures. Quality of rock and adequate rock cover in terms of the internal pressure to which the tunnel is subjected govern the use of these sections. Modified horse-shoe section offers the advantage of flat base for constructional ease and change over to circular section with minimum

additional expenditure in reaches of inadequate rock cover and poor rock formations.

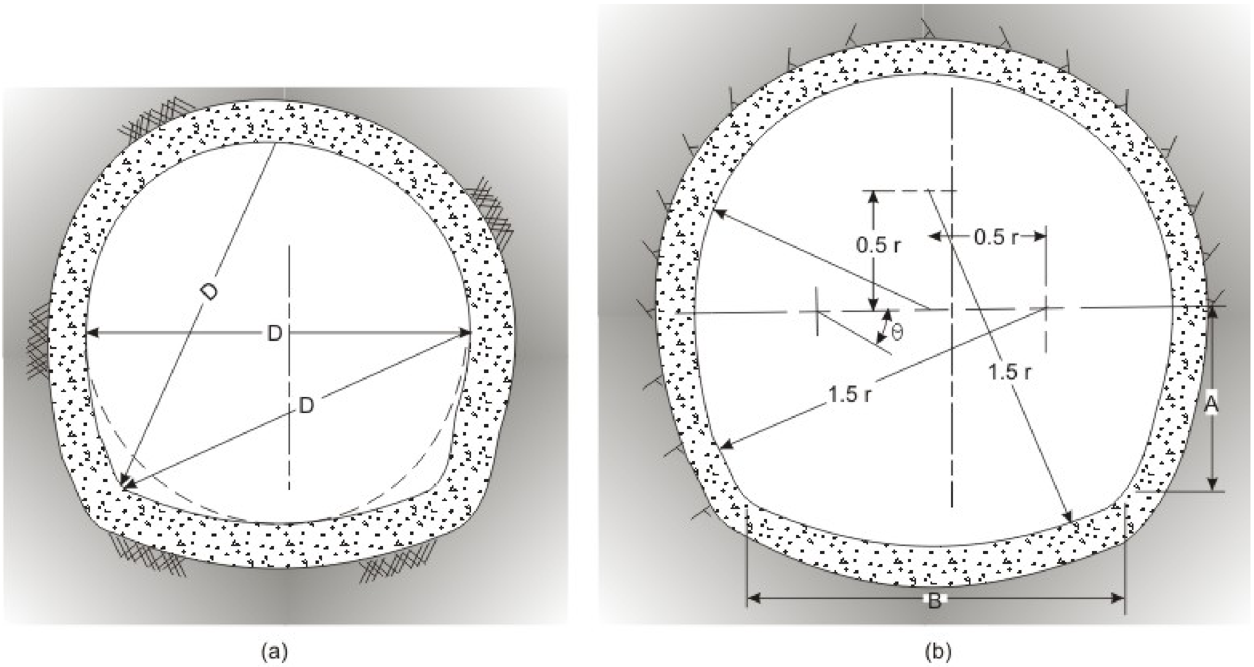


FIGURE 21. (a) Horse shoe section ; (b) Modified horse shoe section of tunnels

4. Egg Shaped and Egglipse Sections (Figure 22 a and b): Where the rock is stratified, soft and very closely laminated (as laminated sand stones, slates, micaceous schists, etc) and where the external pressures and tensile forces in the crown are likely to be high so as to cause serious rock falls, egg shaped and egglipse sections should be considered. In the case of these sections there is not much velocity reduction with reduction in discharge. Therefore, these sections afford advantage in cases of sewage tunnels and tunnels carrying sediments. Egglipse has advantage over egg shaped section as it has a smoother curvature and is hydraulically more efficient.

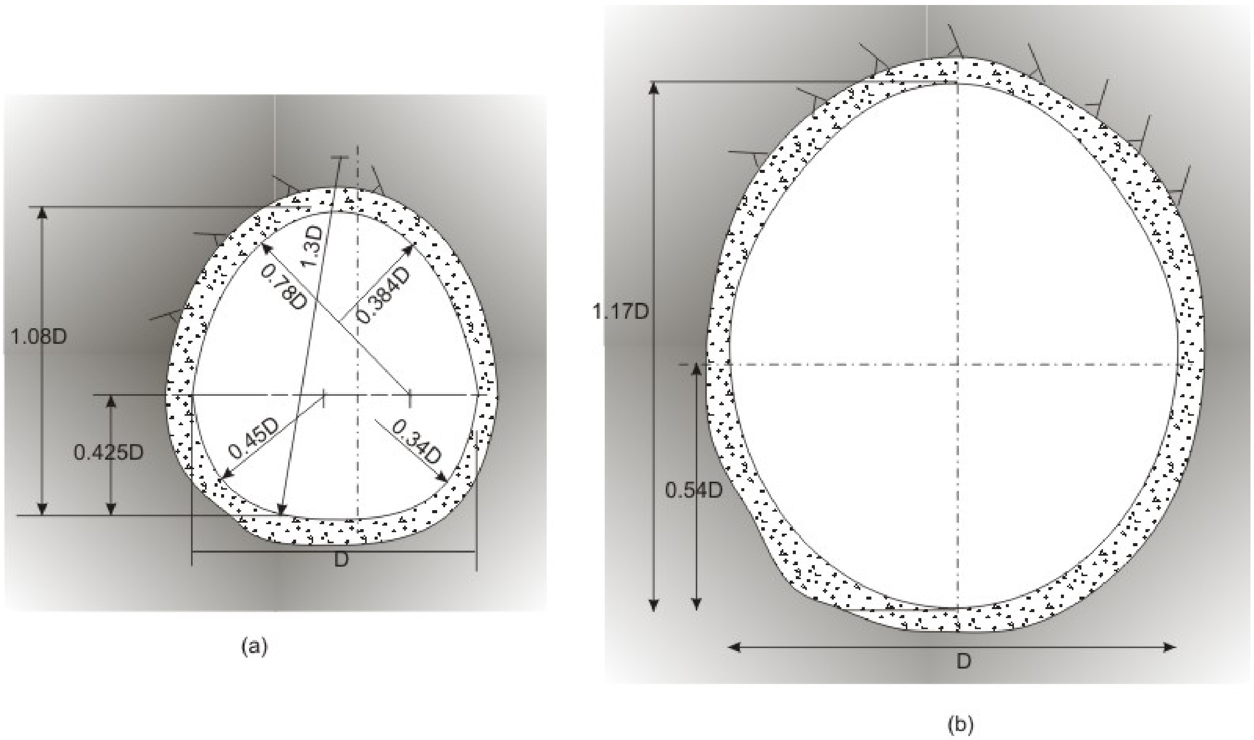


FIGURE 22. (a) Egg shaped section ; (b) Egg lipse section of tunnels

In addition to the sections mentioned above there may be other composite geometrical sections which may be adopted particularly for tunnels which are free flowing and often only partly lined. If the characteristics of a rock formation are fairly well known it may be possible to evolve a section which is likely to fit the shape in which the rock will break naturally. Thus, while a horse-shoe or D section is fairly easy to obtain in some formations there are others where the tunnel crown tends to break into a form more nearly square, and if there is no risk of heavy external pressure upon the lining or if the tunnel is to be unlined there is no reason why the designed cross section should not be made to suit the characteristics of the rock.

Tunnel entrance and exits

It is also essential to design the entry and exit points of the tunnel very carefully. Where the tunnel emerges out of the hill slope, a structure in the form of an arch is usually provided, which is called the portal (see Figure 18). Since at these points the water enters or leaves the tunnel, they are prone to hydraulic head loss and proper transition shape has to be provided to keep the loss minimum and to avoid cavitation. The length and slope of the transition depends upon the velocity and flow conditions prevailing in the tunnel, economics, construction limitations, etc. It is generally preferred that a hydraulic model study is conducted to arrive at an efficient but economic transition.

Where a tunnel meets a surge tank, some head loss may be expected because of the expansion. Similarly, head losses have also to be taken into account for any contraction as well in the shape of the tunnel. As seen from Figure15, there could be a possible change of alignment in plan of a tunnel and this may also lead to a loss in a pressure tunnel.

Tunnel flow problems

The presence of air in a pressure tunnel can be a source of grave nuisance as discussed below:

a) The localization of an air pocket at the high point in a tunnel or at a change in slope which occasions a marked loss of head and diminution of discharge.
b) The slipping of a pocket of air in a tunnel and its rapid elimination by an air vent can provoke a water hammer by reason of the impact between two water columns.
c) The supply of emulsified water to a turbine affects its operation by a drop in output and efficiency thus adversely affecting the operation of generator. The presence of air in a Pelton nozzle can be the cause of water hammer shocks. Admission of air to a pump may occasion loss of priming.
d) If the velocity exceeds a certain limit air would be entrained causing bulking.

Source of Air

Air may enter and accumulate in a tunnel by the following means:

a) During filling, air may be trapped along the crown at high points or at changes in cross-sectional size or shape;
b) Air may be entrained at intake either by vortex action or by means of hydraulic jump associated with a partial gate opening; and
c) Air dissolved in the flowing water may come out of solution as a result of decreases in pressure along the tunnel.

Remedial Measures

The following steps are recommended to prevent the entry of air in a tunnel:

a) Shallow intakes are likely to induce air being sucked in. Throughout the tunnel the velocity should either remain constant or increase towards the outlet end. It should be checked that at no point on the tunnel section negative pressures are developed.
b) Vortices that threaten to supply air to a tunnel should be avoided, however, if inevitable they should be suppressed by floating baffles, hoods or similar devices.
c) Partial gate openings that result in hydraulic jumps should be avoided.
d) Traps or pockets along the crown should be avoided.

Tunnel structural design

The geometric and hydraulic design of a tunnel is followed by the structural design, which investigates the loads that are expected on the tunnel opening from the surrounding rockmass and whether a support is required to hold it in place or a lining is necessary to resist the pressure of the rock and water pressure from the saturated joints and cracks of the surrounding rocks.

Only some limited geological formations are so perfectly intact that they require no external support for their stability. In general, most of the tunnels are driven through rocks with certain defects requiring provision of some form of support until a lining can be completed. Thus, the basic philosophy of design of an underground excavation (tunnelling, surge tanks, power houses etc.) is such as to utilise the rock mass itself as the principal structural material, creating as little disturbance as possible during the excavation process and adding as little as possible in the way of steel supports or shotcrete (which is a wire mesh fixed to the tunnel wall by nails and sprayed with cement slurry with or without steel fibre is used to form a layer, as explained further on).
The type of rock support that has to be provided for a tunnel depends upon the type of rock quality, which is classified according to its behaviour when an opening is made in the rock. The Bureau of Indian Standards code IS: 15026-2002 “Tunnelling methods in rock masses-guidelines” indicates the features of the various types of rocks that are generally encountered. It also recommends the type of excavation method that is to be adopted and the type of support that would be appropriate.

The methods for providing temporary or permanent supports to the tunnels are as described the following paragraphs:

Steel supports

These are built of steel sections, usually I-sections, either shaped or welded in pieces in the form of a curve or a straight section as shown in Figure 23.

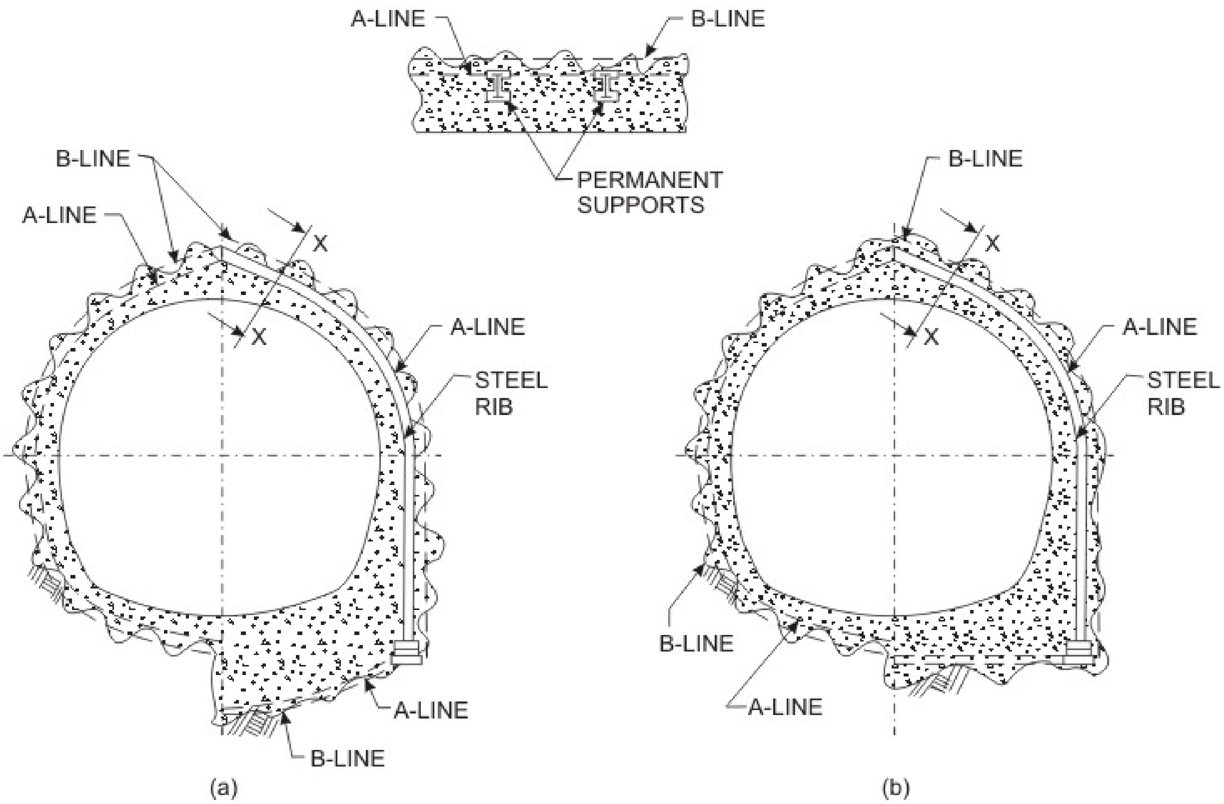


FIGURE 23. Steel support for tunnel and finished in the form of (a) Horse shoe section & (b) Circular section

IS: 15026-2002 recommends various types of steel sections, also called steel ribs, as follows:

a) Continuous rib (Figure 24a)
b) Rib and post (Figure 24b)
c) Rib and wall plate (Figure 24c)
d) Rib, wall plate and post (Figure 24d)
e) Full circle rib (Figure 24e)
f) Invert strut with continuous rib (Figure 24f)

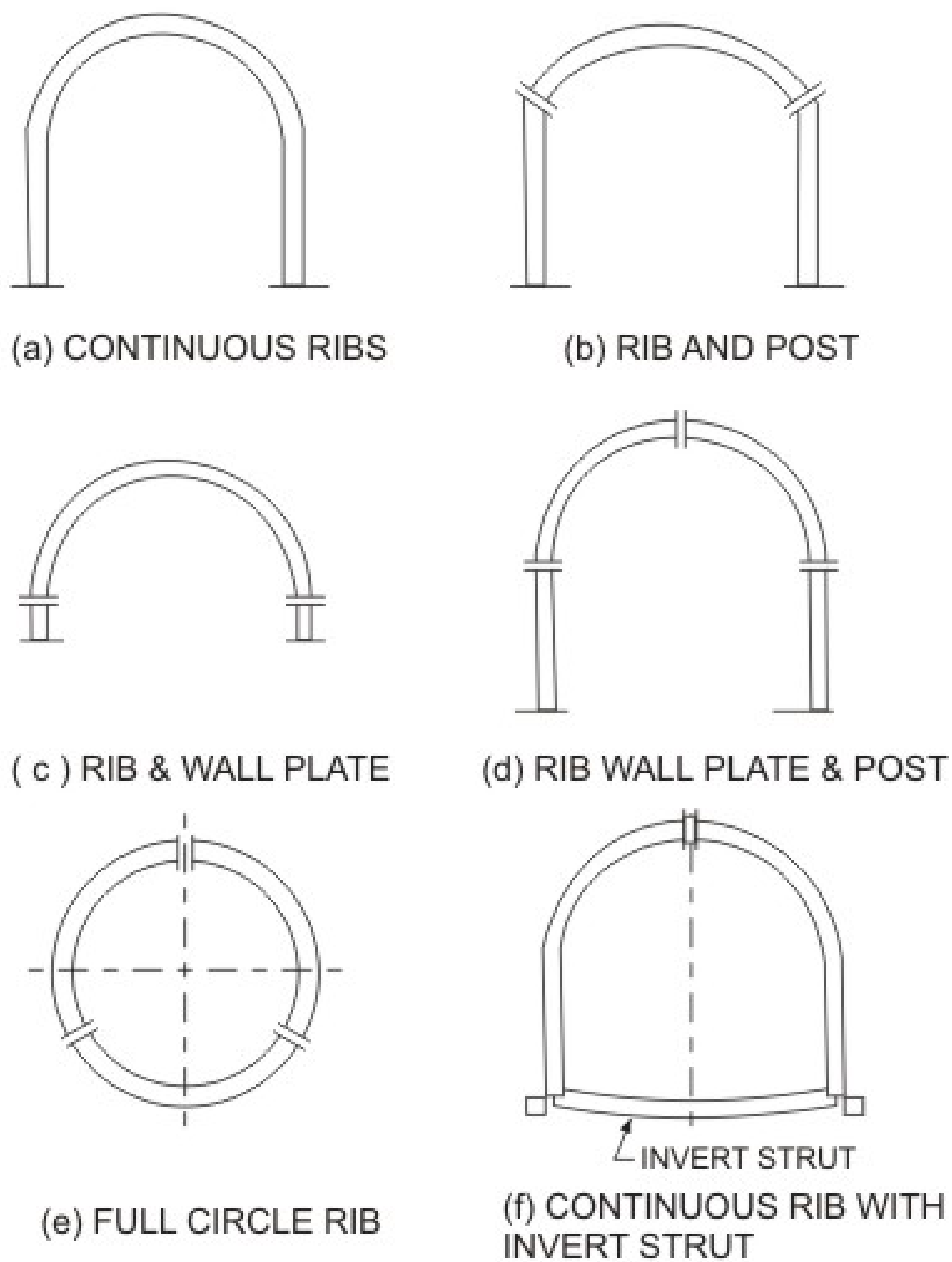


FIGURE 24. Types of steel support syste

Grouting

This is a cement mortar with proportion of cement, sand and water in the ratio 1:1:1 by weight usually, though it may be modified suitably according to site conditions.

Grouting is carried out to fill discontinuities in the rock by a suitable material so as to improve the stability of the tunnel roof or to reduce its permeability or to improve the properties of the rock. Grouting is also necessary to ensure proper contact of rock face of the roof with the lining. In such cases grouting may be done directly between the two surfaces. All the different types of grouting may not be required in each case. The grouting procedures should aim at satisfying the design requirements economically and in conformity with the construction schedules. The basic design requirement generally involve the following:

a) Filling the voids, cavities, between the concrete lining and rock and /or between the concrete and steel liner;
b) Strengthening the rocks around the bore by filling up the joints in the rock system;
c) Strengthening the rock shattered around the bore;
d) Strengthening the rock, prior to excavation by filling the joints with cementing material and thus improving its stability; and
e) Closing water bearing passages to prevent the flow of water into the tunnel and/or to concentrate the area of seepage into a channel from where it can be easily drained out.

Rock/roof bolts

Roof bolts are the active type of support that improve the inherent strength of the rock mass which acts as the reinforced rock arch whereas, the conventional steel rib supports are the passive supports and supports the loosened rock mass externally. All rock bolts should be grouted very carefully in its full length.

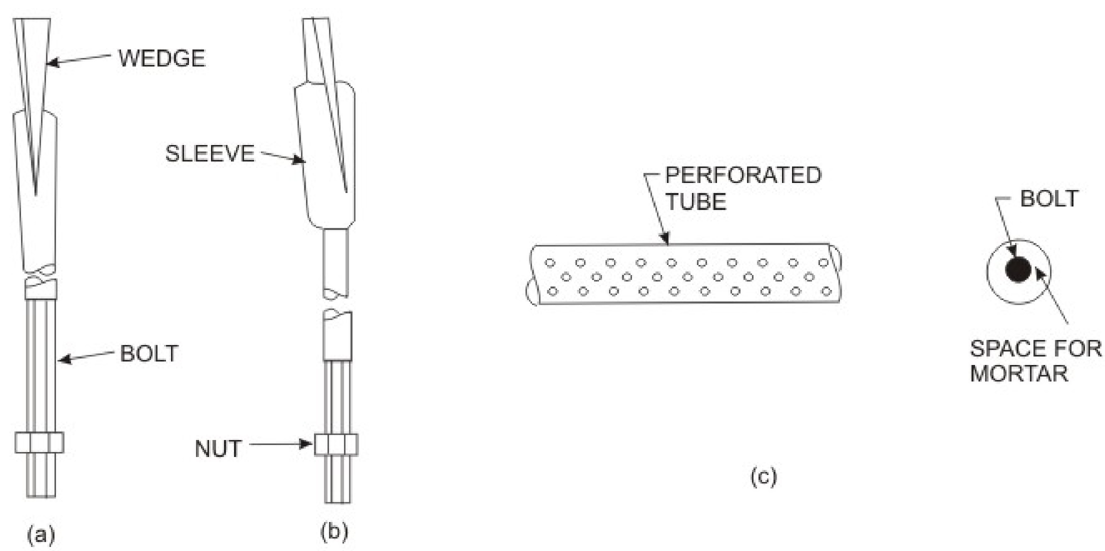


FIGURE 25 Different types of rock/roof bolts
(a) Wedge & slot bolt
(b) Wedge & sleeve bolt
(c) Perfo bolts

There are many types of rock bolts and anchors which may also be used on the basis of past experience and economy. The common types of rock bolts used in practice are the following:

Wedge and Slot bolt

These consist of mild-steel rod, threaded at one end, the other end being split into two halves for about 125 mm length. A wedge made from 20 mm square steel and about 150mm long shall be inserted into the slot and then the bolt with wedge driven with a hammer into the hole which will force the split end to expand and grip the rock inside the hole forming the anchorage. Thereafter, a 10 mm plate of size 200×200 mm shall be placed over which a tapered washer is placed and the nut tightened (see Figure 25a). The efficiency of the spiliting of the bolt by the wedge depends on the strata at the end of the hole being strong enough to prevent penetration by the wedge end and on the accuracy of the hole drilled for the bolt. The diameter of such bolt may be 25mm or 30mm. Wedge and slot bolts are not effective in soft rocks.

Wedge and Sleeve bolts

This consists of a 20 mm diameter rod, one end of which is cold-rolled threaded portion while other end is shaped to form a solid wedge forged integrally with the bolt and over this wedge a loose split sleeve of 33 mm external diameter is fitted (see Figure 25b). The anchorage is provided in this case by placing the bolt in the hole and pulling it downwards while holding the sleeve by a thrust tube. Split by the wedge head of the bolt, the sleeve expands until it grips the sides of the tube. Special hydraulic equipment is needed to pull the bolts.

Perfo bolts

This method of bolting consists of inserting into a bore hole a perforated cylindrical metal tube which is previously filled with cement mortar and then pushing a plain or ribbed bolt. This forces part of the mortar to ooze out through the perforations in the tube and come into intimate contact with the sides of the bore hole thus cementing the bolt, the tube and the rock into one homogeneous whole (see Figure 25c).

Steel fibre reinforced shotcrete (SFRS)

Steel fibre reinforced shotcrete either alone or in combination with rock bolts (specially in large openings) provides a good and fast solution for both initial and permanent rock support. Being ductile, it can absorb considerable deformation before failure.

Controlled blasting should be used preferably. The advantage of fibre reinforced shotcrete is that smaller thickness of shotcrete is needed, in comparison to that of conventional shotcrete. Fibre reinforced shotcrete along with resin anchors is also recommended for controlling rock burst conditions because of high fracture toughness of shotcrete due to specially long steel fibres. This can also be used effectively in highly squeezing ground conditions. It ensures better bond with rock surface. With mesh, voids and pockets might from behind the mesh thus causing poor bond and formation of water seepage channels as indicated in Figure 26.

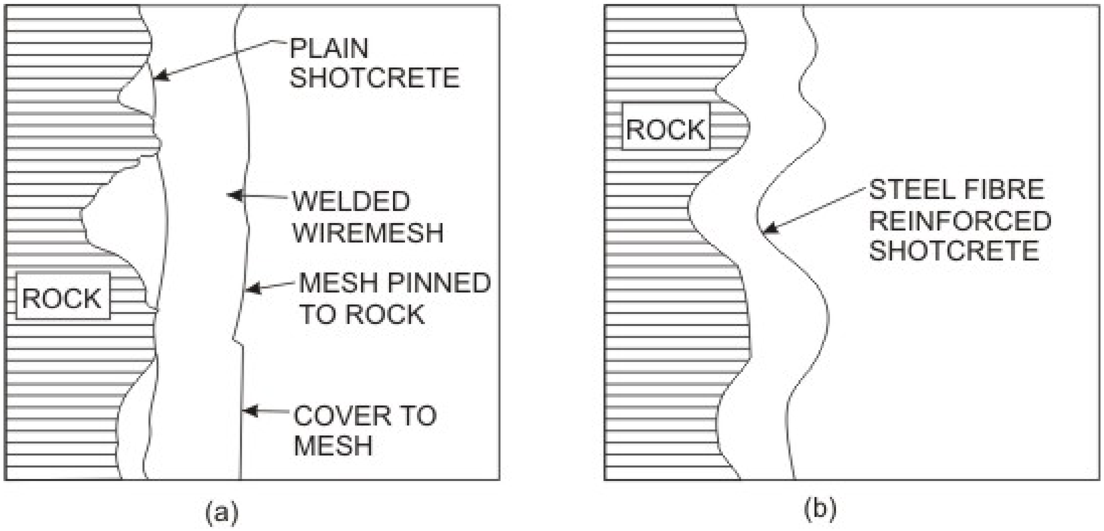


FIGURE 26. Differences in shotcrete consumption when wire mesh (a) or steel fibres (b) are used

The major draw-back of normal shotcrete is that it is rather weak in tensile, flexural and impact resistance strength. These mechanical properties are improved by the addition of steel fibres. Steel fibres are commonly made into various shapes to increase their bonding intimacy with the shotcrete (see Figure 27). It is found that hooked ends types of steel fibres behave more favourably than other types of steel fibres in flexural strength and toughness. Accelerators play a key role to meet the requirement of early strength.

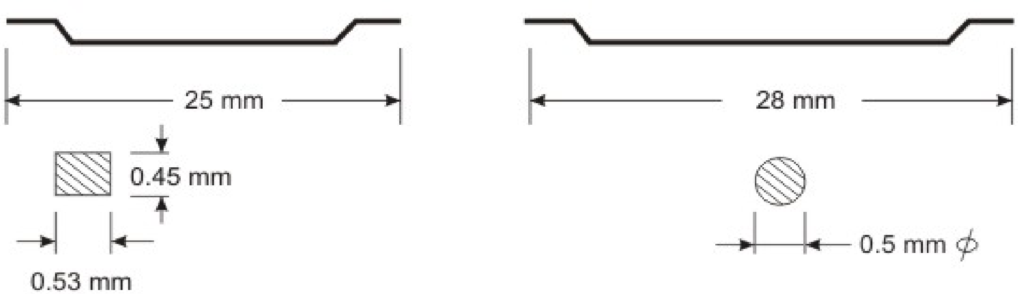


FIGURE 27. Typical fibre used in shotcrete work.

Steel fibres make up between 0.5 to 2 percent of the total volume of the mix (1.5 to 6 percent by weight). Shotcrete mixes with fibre contents greater than 2 percent are difficult to prepare and shot.

Concrete lining

This is a protective layer within the tunnel made of plain or reinforced concrete. Tunnels may be completely lined, partially lined, or even unlined. Tunnels in good sound rock may be kept unlined. However, lining is recommended when:

a) The internal water pressure exerted by water conveyed by the tunnel is high, say above 100m of water head. For very good competent rock, tunnels may be kept unlined for pressures even up to 200m water head.
b) The rock strata through which the tunnel passes has low strength and where the rock is anisotropic.

Lining a tunnel increases the cost of a project and should be adopted considering the advantages expected as given below:

a) Lining transmits part of the internal water pressure to the surrounding rock which, to some extent, is balanced by the external rock pressure. In tunnel empty condition, it helps to resist the external rock load together with the support system.
b) Lining may be carried together with the tunnel excavation work and hence minimizes the danger of accidental rock falls within the tunnel.
c) Lining helps to reduce water loss through joints in rocks by seepage.
d) Lining is invariably provided at the inlet and outlet portals of a tunnel, even if located within competent rock.

Tunnels conveying water under free flow conditions may be un-reinforced. The external rock load is expected to be carried by the steel supports. Usually, a tunnel lining has to be reinforced when the depth of rock cover (from the tunnel soffit up to the free surface of the hill) is less than the internal water pressure.

The design of concrete linings for tunnels may be done according to the recommendations of the following Bureau of Indian Standards code IS: 4880(Part IV)-1971 “Code of practice for design of tunnels conveying water (structural design of concrete lining in soft strata and soils”.

The construction of tunnels could be by manual methods like drilling holes, placement of explosive, blasting, and then removal of the muck from the head-face or by competent rocks well. As soon as the tunnel face is excavated to a certain depth, the temporary supports are provided to prevent any rock fall or squeezing. At the same time, or later, permanent supports are also put in place.

5.2.4 Penstocks

As mentioned before, a penstock is usually steel or reinforced concrete lined conduit that supplied water from the reservoir, forebay or surge tank at the end of a head race tunnel to the turbines. A penstock is subjected to very high pressure and its design is similar to that for pressure vessels and tanks. However, sudden pressure rise due to value closure of turbines during sudden load rejection in the electric grid necessitates that penstocks be designed for such water hammer pressures as well. Penstocks, at their lowermost end meets a controlling value, from where the water is led to the spiral casing of the turbine, details of which would be discussed in the next lesson.

Since penstocks convey water to the turbines and form a part of the hydropower water conveyance system, it is necessary that they provide the least possible loss of energy head to the flowing water. According to the Bureau of Indian Standards code IS: 11625-1986 “Criteria for hydraulic design of penstocks”, the following losses may be expected for a penstock:

a. Head loss at trash rock
b. Head loss at intake entrance
c. Friction losses, and
d. Other losses as at bends, bifurcations, transitions, values, etc.

Based on the above losses, the diameter of the penstock pipes have to be fixed, such that it results in an overall economy. This is because if the diameter of a penstock is increased, for example, the friction losses reduce resulting in a higher head at turbine and consequent generations of more power. But this, at the same time, increases the cost of the penstock. This leads to the concept of Economic Diameter of Penstock which is one such that the annual cost, including cost of power lost due to friction and charges of amortization of construction cost, maintenance, operation, etc. is the minimum.

A penstock made of steel may be constructed as a seamless pipe, rolled or drawn from mild steel if the diameter is within 0.5m. Larger diameter pipes are usually manufactured from steel plates welded together. The joints have to be carefully tested by ultrasonic or radiographic methods which ensures that high pressure may be tolerated by the pipes.

Penstocks may also be classified according to their location with respect to the ground surface. If they are buried within ground or laid inside a tunnel drilled (see Figure 18) within the mass of a hill, then they have to be designed to take the load of the surrounding soil or rock. Such buried or embedded penstocks may be differentiated from those that are laid above the ground surface, termed as the surface penstocks, which are subjected to variation in temperature of the surroundings especially due to the sum’s direct radiation. Such and other advantages and disadvantages of embedded and surface penstocks may be listed as under:

Sl. No	***Embedded Penstocks***	***Surface Penstocks***
1.	Protection against temperature effect	Subjected to temperature variations
2.	Landscape does not get affected	Landscape becomes scared with the Penstocks presence

3.	Less accessible for inspection	Easily accessible for inspection
4.	Greater expenses for large diameter penstocks in rocky soil	Economical under such circumstances
5.	Does not require separate support. Does not require expansion joints	Requires anchorages for support necessitating in expansion joints

The following Bureau of Indian Standards codes may be referred for the design of embedded and surface penstocks respectively.

IS: 11639-1986 “Criteria for structural design of penstocks”
  Part1: Surface penstocks
  Part2: Buried / embedded penstocks

A penstock is not only a single straight piece of pipeline. It has to certain additional pieces, called specials, to allow it to be located over undulating terrain or within curved or contracted tunnels, provide access for inspection, etc. Design of these special attachments to a penstock is provided by the Bureau of Indian Standards code IS: 11639(Part3)-1996 “Structural design of penstocks-criteria (Specials for penstocks)”. The following paragraphs briefly described these specials and the purpose they serve.

Bends

Depending on topography, the alignment of the penstock is often required to be changed, in direction, to obtain the most economical profile so as to avoid excess excavation of foundation strata and also to give it an aesthetic look with the surroundings. These changes in direction are accomplished by curved sections, commonly called penstock bends. For ease of fabrication, the bends are made up of short segments of pipes with mitered ends.

Bends may be only in one plane, in which case it is known as a simple bend. If the curvature or change in alignment is in two planes- horizontal as well as vertical- then it is called a compound bend.

Reducer piece

In the case of very long penstocks, it is often necessary to reduce the diameter of the pipe as the head on the pipe increases. This reduction from one diameter to another should be effected gradually by introducing a special pipe piece called reducer piece. The reducer piece is a frustum of a cone. Normally the angle of convergence should be kept between 5 degrees ton 10 degrees so as to minimize the hydraulic loss at the juncture where the diameter is reduced.

Branch pipe

Depending upon the number of units a single penstock feeds, the penstock branching is defined as bifurcation when feeding two units, trifurcation when feeding three units and manifold when feeding a greater number of units by successive bifurcations. Branch pipes of bifurcating type are generally known as “wye” pieces which may be symmetrical or asymmetrical.

Generally the bifurcating pipe has two symmetric pipes, after the bifurcating joints, and the deflection angle of the branching pipes ranges between 30degrees to 75 degrees. In order to reduce the head loss, a smaller deflection angle is advantageous. However, the lesser the bifurcating angle, greater the reinforcement required at the bifurcating part. The wye branches should be given special care in design to ensure safety of the assembly under internal pressure of water. The introduction of a bifurcation considerably alters the structural behavior of the penstock in the vicinity of the branching.

Expansion joints

Expansion joints are installed in exposed penstocks between fixed point or anchors to permit longitudinal expansion, or contraction when changes in temperature occur and to permit slight rotation when conduits pass through two structures where differential settlement or deflection is anticipated. The expansion joints are located in between two anchor blocks generally downstream of uphill anchor block. This facilitates easy erection of pipes on steep slopes.
Expansion joints should have sufficient strength and water tightness and should be constructed so as to satisfactorily perform their function against longitudinal expansion and contraction. The range of variations to be used for calculation of expanded or contracted length of penstocks should be determined keeping in consideration the maximum and minimum temperature of the erection sites.

Manholes

Manholes are provided in the course of the penstock length to provide access to the pipe interior for inspection, maintenance and repair.

The normal diameter of manholes is 500 mm. Manholes are generally located at intervals of 120-150 metres. For convenient entrance, exit manholes on the penstock may be located on the top surface or lower left or right surface along the circumference of the penstock.

The manhole, in general, consists of a circular nozzle head, or wall, at the opening of the pipe, with a cover plate fitted to it by bolts. Sealing gaskets are provided between nozzle head and cover plate to prevent leakage. The nozzle head, cover plates and bolts should be designed to withstand the internal water pressure head in the penstock at the position of the manhole.

Bulk heads

Bulkheads are required for the purpose of hydrostatic pressure testing of individual bends, after fabrication, and sections or whole of steel penstock and expansion joints, before commissioning. Bulkheads are also provided whenever the penstocks are to be closed for temporary periods, as in phased construction.

Air vents and valves

These are provided on the immediate downstream side of the control gate or valve to facilitate connection with the atmosphere.

Air inlets serve the purpose of admitting air into the pipes when the control gate or valve is closed and the penstock is drained, thus avoiding collapse of the pipe due to vacuum excessive negative pressure. Similarly, when the penstock is being filled up, these vents allow proper escape of air from the pipes.

The factors governing the size of the vents are length, diameter, thickness, head of water, and discharge in the penstock and strength of the penstock under external pressure.

Manifold

The portion beyond the main penstock which feeds the branches for the individual units, when two or more units are fed from one penstock. Apart from the above, the following are required for aligning and holding a penstock in place.

Anchorage/ Anchor Block/Anchor pier

This is a structure built to hold down penstocks in position at the points where the direction or inclination of the axis changes and also at some regular intervals. In the closed type of anchor, the penstock is embedded in concrete. In the open-type, the penstock is anchored to concrete by rings. Intermediate supports are also provided for penstocks between two anchor blocks, over which the pipe can slide while expanding or contracting. Sometimes thrust blocks are provided on either side of branch connections to resist unbalanced forces at the penstock connection and thus maintain alignment of outlet headers.

Concrete saddle supports

These are a type of intermediate supports with concrete base shaped to suit the bottom of the pipe. A well lubricated steel plate, rolled to suit the shape of the pipe shell in contact, is provided in between the concrete surface and the pipe to facilitate smooth movement of the pipe over saddles.

Module 5

HYDROPOWER ENGINEERING

LESSON 3

HYDROPOWER EQIUPMENT AND GENERATION STATIONS

Instructional objectives:

On completion of this lesson, the student shall learn about:

1. Equipment employed for converting water energy to electrical energy
2. Different types of turbines
3. Guidelines for selecting a specific turbine
4. layout of power houses

5.3.0 Introduction

The powerhouse of a hydroelectric development project is the place where the potential and kinetic energy of the water flowing through the water conducting system is transformed into mechanical energy of rotating turbines and which is then further converted to electrical energy by generators. In order to achieve these functions, certain important equipments are necessary that control the flow entering the turbines from the penstocks and direct the flow against the turbine blades for maximum efficient utilization of water power. Other equipments necessary are couplings to link the turbine rotation to generator and transformers and switching equipment to convey the electric power generated to the power distribution system.

A powerhouse also accomodates equipment that are necessary for regular operation and maintenance of the turbine and power generating units. For example, overhead cranes are required for lifting or lowering ofturbines and generator during installation period or later for repair and maintenance. For the crane to run, guide rails on columns are essentially required. The maintenance of a unit is done by lifting it by the crane and transporting it to one end of the power house where abundant space is kept for placing the faulty unit. A workshop nearby provides necessary tools and space for the technicians working on the repair of the units.

A control room is also essential in a powerhouse from where engineers can regulate the valves controlling water flow into the turbines or monitor the performance of each unit to the main power grid.

Power houses that receive water from a reservoir through a penstock may be termed as power generating units detached from head works. There is another class of powerhouse that utilize the water head directly from the water body. These are usually the run-of the river type power houses mentioned in Lesson 5.1, which are located as a part of a barrage in a river or those which utilize the head difference of a canal fall.

The detached power houses may be surface or underground types depending upon its position with respect to the ground surface. In-stream or run-of-river power houses are mostly surface type.

Turbines are of different types like reaction or impulse-types. They may also be divided as with horizontal or vertical axes. This lesson discusses all the salient features of a hydropower generating station including the layout, structural components and mechanical parts.

5.3.1 Hydraulic turbines

These form the prime mover which transforms the energy of water into mechanical energy of rotation and whose prime function is to drive a hydroelectric generator. The turbine runner and the rotor of the generator are usually mounted on the same shaft, and thus the entire assembly is frequently referred to as the turbo-generator.

Hydroelectric plants utilize the energy of water falling through a certain difference in levels which may range from a few meters to 1500m or even 2000m. To handle such a wide range of pressure heads, various turbines differing in design of their working is employed. Modern hydraulic turbines are divided into two class - impulse and reaction. An impulse turbine is one in which the driving energy is supplied by the water in kinetic form, and a reaction turbine is one in which the driving energy is provided by the water partly in kinetic and partly in pressure form. The basic types of impulse and reaction turbines are given in the following table.

Turbine types	Class	Head range
Propeller turbines:		
Fixed blade turbines	Reaction	10-60m
Adjustable blade (Kaplan turbine)	Reaction	10-60m
Diagonal flow turbines	Reaction	50-150m
Francis turbine	Reaction	30-400m (even up to 500 to 600m)
Pelton turbine	Impulse	Above 300m

Each turbine has a water passageway which incorporates a turbine casing, stay vanes for support, wicket gates for flow control, a runner that rotates the generator, and a draft tube or the exit channel downstream of the turbine.

Figure1 shows the typical positioning of a reaction and impulse turbines with respect to the incoming water conducting system and the draft tube, which is the exiting duct of the flowing water.

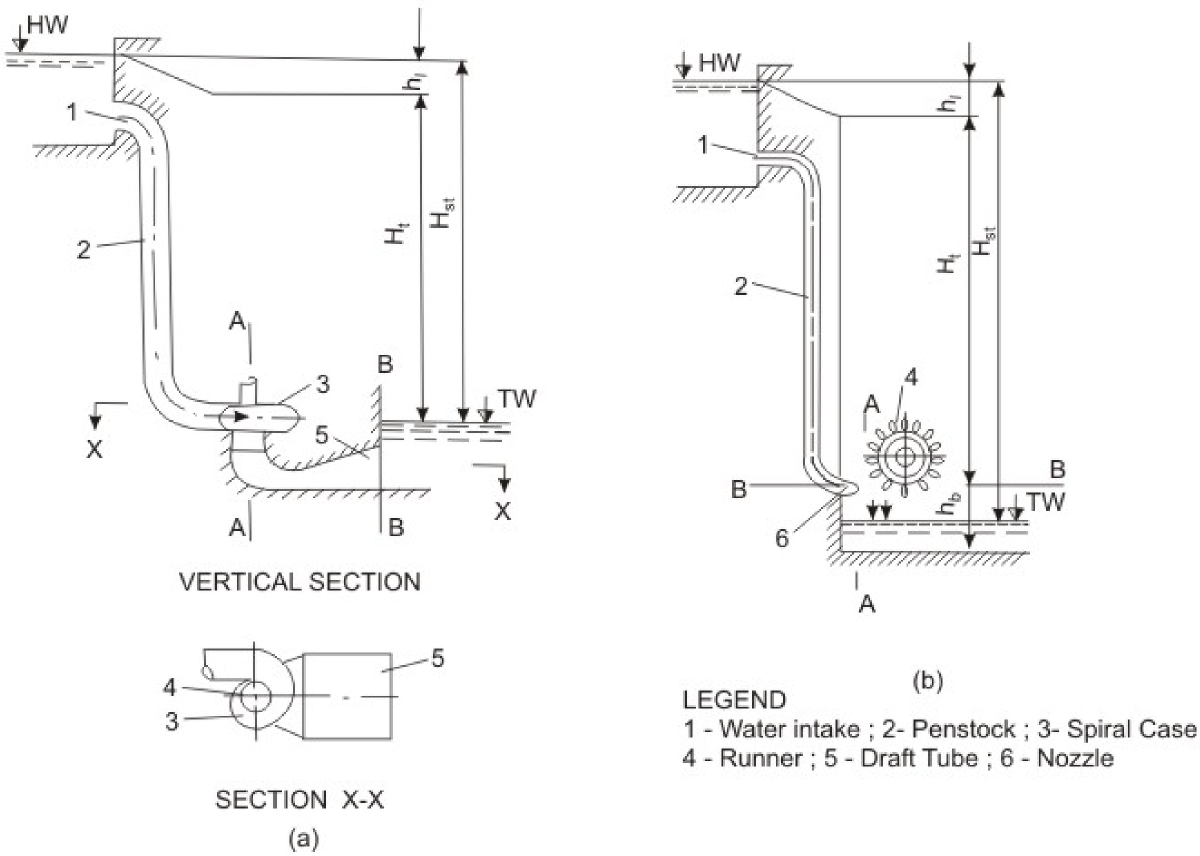


FIGURE 1 Layout for (a) Reaction Turbine ; (b) Impulse Turbine.

For the reaction type of turbine, (Figure 1a), the turbine section is assumed to begin at the entrance to the turbine case (section A-A in the figure) and end at the exit from the draft tube (section B-B). It may be noted that this setting for a reaction turbine ensures the following characteristics:

1. The wheel passage remains completely filled with water
2. The water acting on the wheel vanes is under pressure greater than atmospheric
3. The water enters all round the periphery of the wheel through the scroll case
4. Energy in the form of both pressure and kinetic is utilized by the wheel

In the Figure1, H_t is the head of water on the turbine and is the difference in water specific energy between beginning and end of the turbine section.

For the impulse type turbine (Figure1b), the following characteristics of the turbine setting differentiates it from the impulse type of turbines:

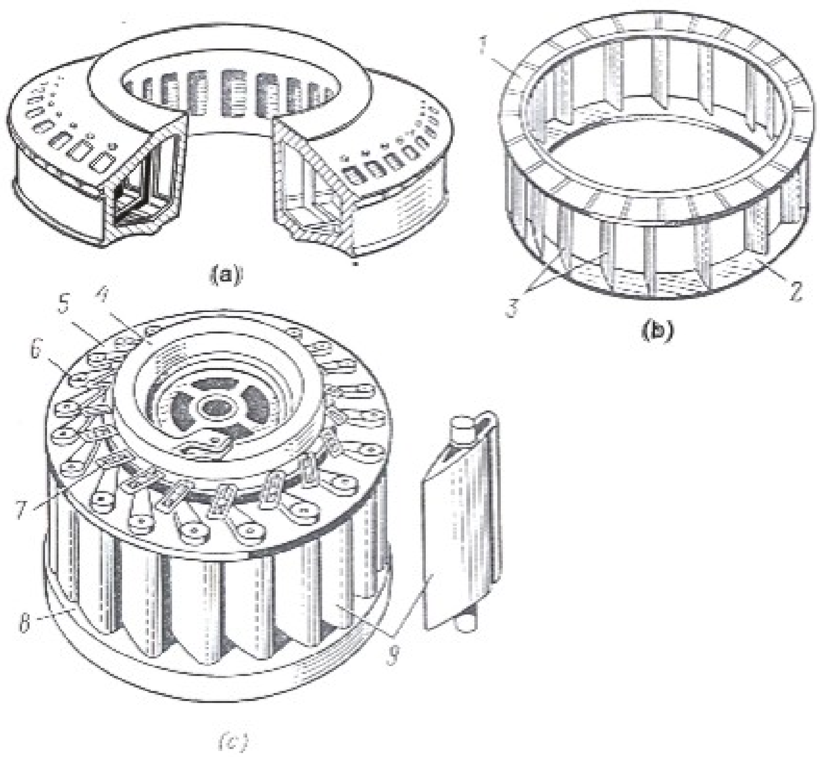
1. The wheel passages are not completely filled with water since a jet emanating from the penstock nozzle strikes the buckets of the runner
2. The water acting on the vanes or buckets located at the wheel periphery is under atmospheric pressure
3. The water impacts on the runner at one point or at a few discrete points, depending upon the number of nozzles

4. Energy applied to the wheel is completely kinetic

The components of the different types of turbines are discussed in the following paragraphs.

Reaction turbines

A turbine has stationary as well as rotating members. The stationary components of a reaction turbine with vertical axis are shown in Figure 2.



1=Upper circular plate of stay ring ; 2=Lower circular plate of stay ring; 3=Stay vanes; 4=Operating ring
5=Upper ring of wicket gates; 6=Levers; 7=Slotted guides ; 8= Lower ring of wicket gates; 9= Gates

FIGURE 2. Stationary components of a reaction turbine
(a) Cover ; (b) Stay ring ; (c) Wicket gates

The purpose of each part is as follows:

1. Stay ring, which forms the outer support of a turbine case. It resists heavy loads imposed by the equipment and the concrete of the power house structure. It consists of an upper and lower circular plates joined by 10 to 16 stay vanes of streamline shape. The stay rings of some turbines may have no lower plate.
2. Wicket gates, which are provided inside the stay rings regulate the discharge and direct the water flow towards the runner by their angle. Also, the wicket gates serve to start

up and shutdown the turbine and to control its power output and rotational speed. Wicket gates also have an upper and lower rings resting in recesses provided in the circular plates of the stay ring. The rings of the wicket gate are joined together by 20 to 32 blades (or gates) of streamline shape.

3. Operating ring, to which the gates are connected by slotted guides and levers rigidly fastened to their top pins. The function of the operating ring is to open or close the gates which can rotate about their axes.

4. Turbine cover, The operating ring is installed in a recess provided in the turbine cover which, in turn, rests in a recess made in the upper ring of the wicket gates. The turbine cover gives support to the turbine bearing and separates the water passageway through the turbine from the dry turbine pit.

The rotating or movable parts of a reaction turbine comprises of the runner and the shaft. The shape of such part differs for different types of reaction turbines, as described below:

1. Axial flow reaction turbines

Figure 3 shows the Kaplan type of adjustable blade axial flow turbine, where the stream of water flows in the direction of the axis of the turbine.

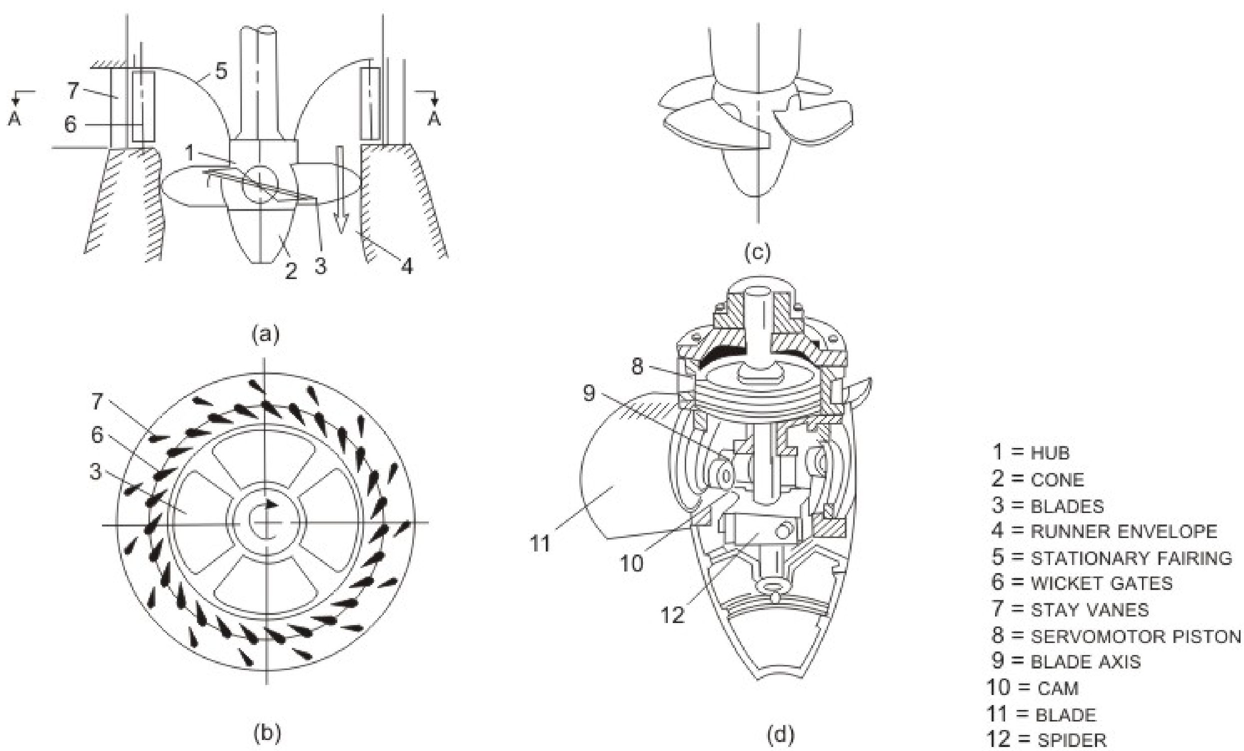
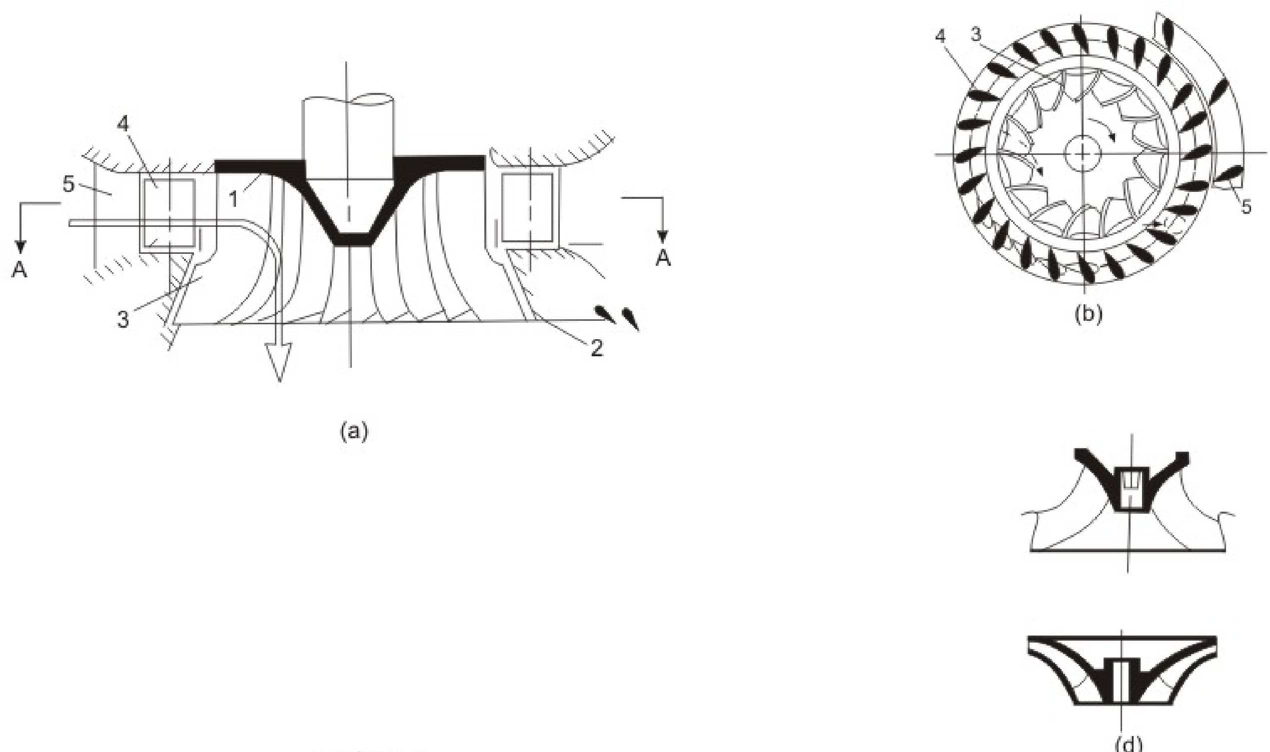


FIGURE. 3 Adjustable - blade propeller (Kaplan) turbine
(a) Cross-sectional elevation ;
(b) Horizontal section A-A through runner ;
(c) General appearance of runner ;
(d) Section through blade adjustment mechanism

The axial flow turbine resembles the propeller of a ship and hence is also termed as the propeller turbine. The runner of such a turbine has three to eight blades, cantilever-mounted on a spherical or cylindrical hub equipped with a cone (Figure 3c). In a fixed blade propeller turbine, the blades are rigidly fastened to the hub at a permanent angle. In the adjustable-blade propeller (or Kaplan) turbine, the blade angles can be adjusted by an hydraulic operated piston located within the hub (Figure 3d). The advantage of having an adjustable-blade for a propeller turbine, as for the Kaplan turbine, is that the fixed blade wheels show peak efficiency at only a small range of load. If load is slightly higher or smaller than this range, its efficiency drastically decreases. On the other hand, the blade adjustable turbines can be operated in such a way that their efficiency remains high over a very large range of load.

2. Radial flow reaction turbines

Figure 4 shows the runner of a Francis turbine, where the flow of water from the stationary part into the turbine runner is in a radial direction.



LEGEAND
1 = CROWN ; 2 = BAND ; 3 = BUCKETS ; 4 = WICKET ; 5 = STAY VANES

FIGURE 4. Francis turbine ; (a) Cross sectional elevation ; (b) Horizontal section A-A through runner ; (c) General appearance of runner ; (d) Runners for various heads

Once inside the runners, the flow direction changes and gets directed downward. The runners of a Francis turbine are attached at their upper ends to a conical crown and the lower ends to a circular band. The shrouded buckets make an angle with the radial planes and bend at the bottom. The number of blades usually ranges from 14 to 19 (Figure 4c). As with the propeller turbine, the runner shape depends upon the head, as shown in Figure 4d.

3. Diagonal flow reaction turbines

Figure 5 shows the runner of a Deriaz turbine, which is the adjustable-blade modification of a diagonal turbine.

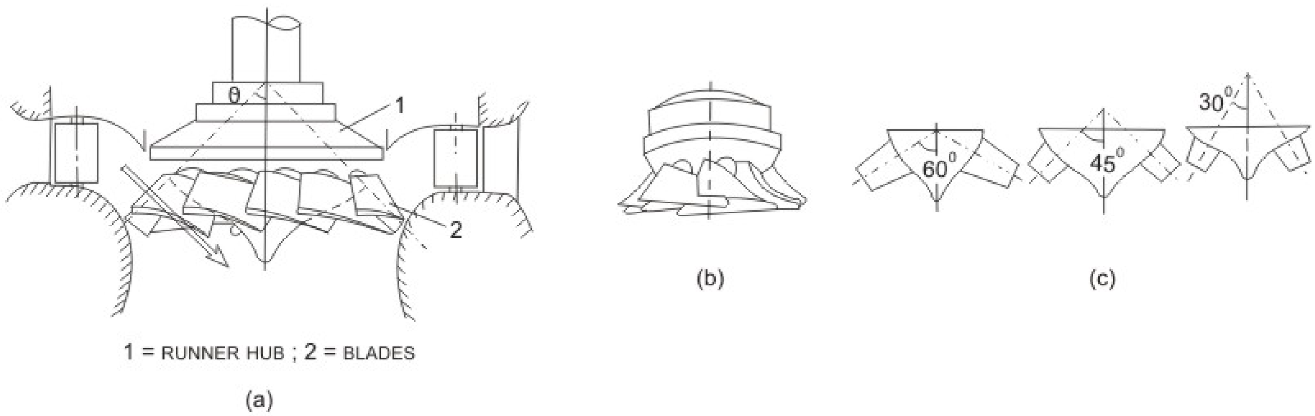


FIGURE 5. Adjustable - blade diagonal (Deriaz) turbine
(a) Cross sectional elevation
(b) General appearance of runner
(c) Runners for various heads

Here, the flow of water against the turbine blades is neither axial nor radial, but at an angle. The angle θ made by the blade axes with the main shaft decreases with increasing head and ranges between 30^0 and 60^0. As with Kaplan turbines, the mechanism for adjusting the blade angles is located within the hub.

Reaction turbines with the turbo-generator axis placed vertically, as described before, causes the stream of water to change by 90^0 from the point where it leaves the stationary part and the water enters the turbine runner. Again, the water turns by 90^0 as it leaves the runner and enters the draft tube before finally joining the tail water. This turning of flow twice – results in a loss of energy. Another inconvenience is that vertical turbo generators require extra head room in low-head power plants and thus becomes inconvenient to place there. Under these circumstances, a horizontal axial-flow (propeller type) arrangement of reaction turbine has been often adopted like the bulb type turbine (Figure 6). It has the generator in series with the turbine runner at a submerged elevation and enclosed in a streamlined water tight housing located in the water passageway on either upstream or downstream side of the runner.

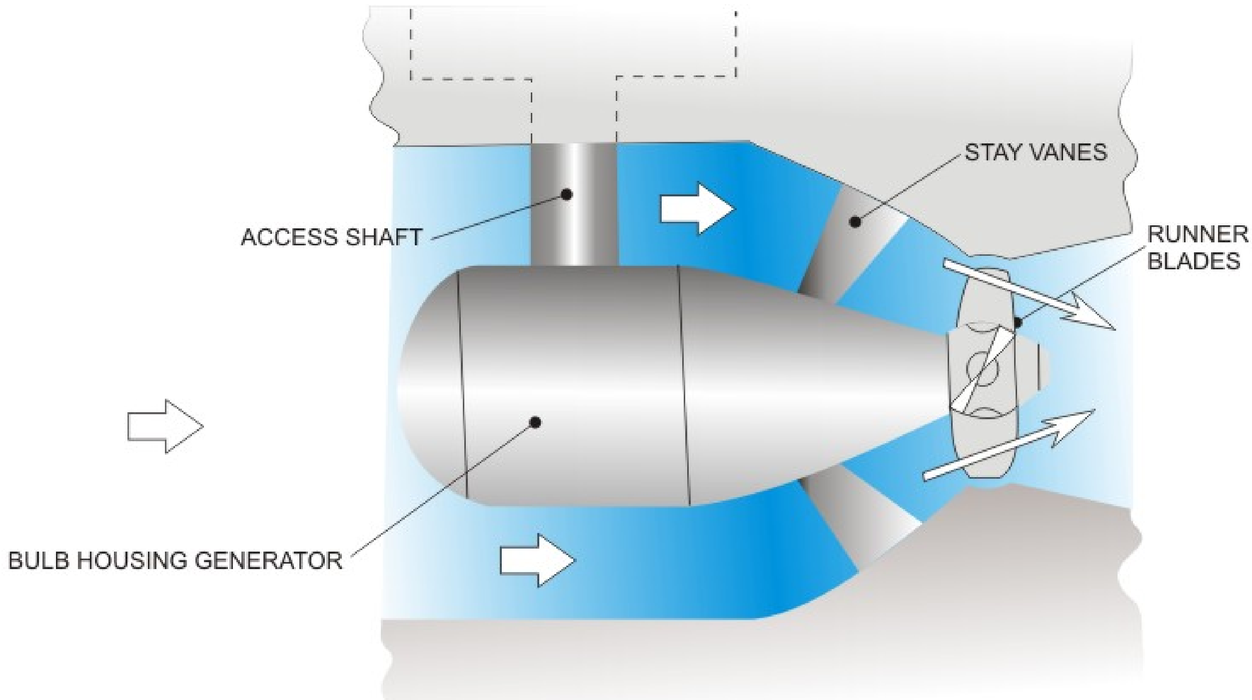


FIGURE 6. Axial - flow bull turbine

The S-type tubular turbine (coupled to an open-air generator) is also favoured for very low head range and designed as a low capacity unit (Figure 7). The blades of the turbine can be fixed (propeller) or movable (Kaplan).

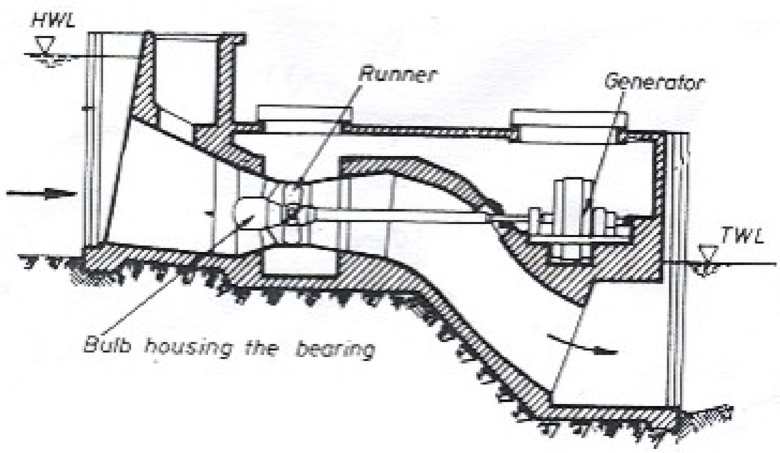
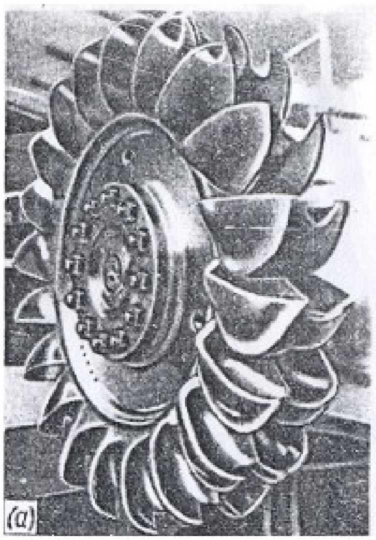
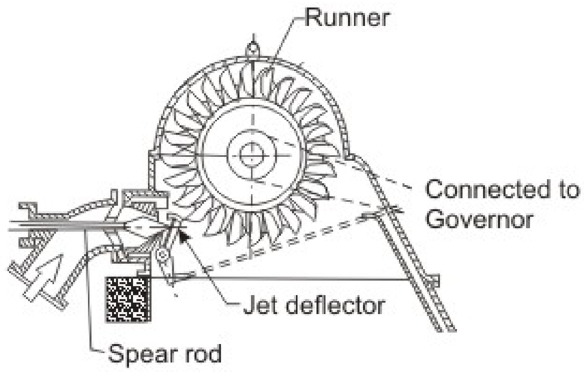


FIGURE 7. Low capacity S-type tubular turbine

Impulse turbines

As mentioned above, the Pelton turbine is an impulse turbine that is used commonly in hydro power projects world wide for very high head installations. The runner of a Pelton turbine (Figure 8) consists of a disk with buckets attached around its periphery. The buckets can be integrally cast with the disk or welded or bolted to the wheel centre. The buckets may vary in details of their construction but in general they are bowl-shaped and have a central dividing wall, or splitter, extending radially outward from the shaft. The runner is enclosed in a housing and is mounted on the same shaft with the rotor of the water-wheel generator, placed either vertically or, more frequently, horizontally. The water is supplied from the upper pool through penstocks which end with one or several tapering nozzles. The water jet escaping from the nozzles hit the dividing wall which splits it into two streams, and the bowl-shaped portions of the buckets turn the water back, imparting the full effect of the jet to the runner. The quantity of water through the nozzle is controlled with the help of a spear-valve. The used water flows down a conduit into the lower pool.





(b)

FIGURE 8. Pelton - type impulse turbine
(a) General view
(b) Section through water passage showing spear - type flow control vulve

5.3.2 Choice of a turbine

The water power designer has to make a choice on the type of turbine that can be adopted for a particular project. After the range of head to be handled by a turbine has been evaluated by stream flow analysis and the installed capacity determined from the analysis of the power-generating capacity of the proposed plant, the task of the designer is to choose an optimum turbine type and series, the number of power generating units, the runner diameter, rotational speed, and runner axis elevation. Knowing the total installation at the power station, the number of units can be decided. The capacity of the plant should be fixed as high as possible with adequate care on efficient running and low initial costs, and available transport and shipping facilities.

The following Bureau of Indian Standard Codes of practices may be referred to for the detailed design of hydraulic turbines and their selections:
1. IS:12837-1989 “Hydraulic turbines for medium and large power houses - guidelines for selection”
2. IS: 12800-1993 “ Guidelines for selection of turbines, preliminary dimensioning and layout of surface hydro-electric power houses”:

Part 1: Medium and large power houses
Part 2: Pumped storage power houses
Part 3: Small, mini and micro hydroelectric power houses

One of the important parameters of a turbine is the Specific Speed denoted as n_s, which and defined as the speed in r.p.m. at which a turbine of homologous design would operate, if the runner were to reduce to a size which would develop one metric horse power under one metre head. It is given by the following relation:

$$n_s = \frac{n\sqrt{P \times 1.358}}{H^{5/4}}$$

where

n_s = Specific speed of turbine in revolutions per minute(r.p.m.)
n = Rated speed of turbine in revolutions per minute
P = turbine output in kW, and
H = Rated head in metres.

Once the specific speed (n_s) is determined, the chart given in IS: 12837-1989 and reproduced in Figure 9 may be used to determine the type of turbine that may be adopted for the particular project.

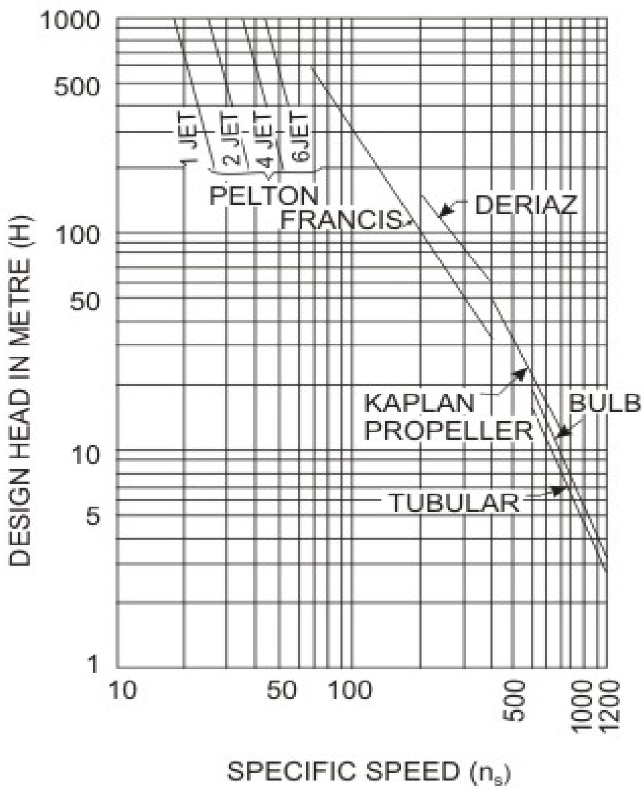


FIGURE 9. Chart for determining the selection of turbine

The type of turbine selected also depends upon techno-economic considerations of the generating equipment, power house cast and relative benefits of power generation. The factors given in the following table determine the type of turbine to be used depending upon site conditions.

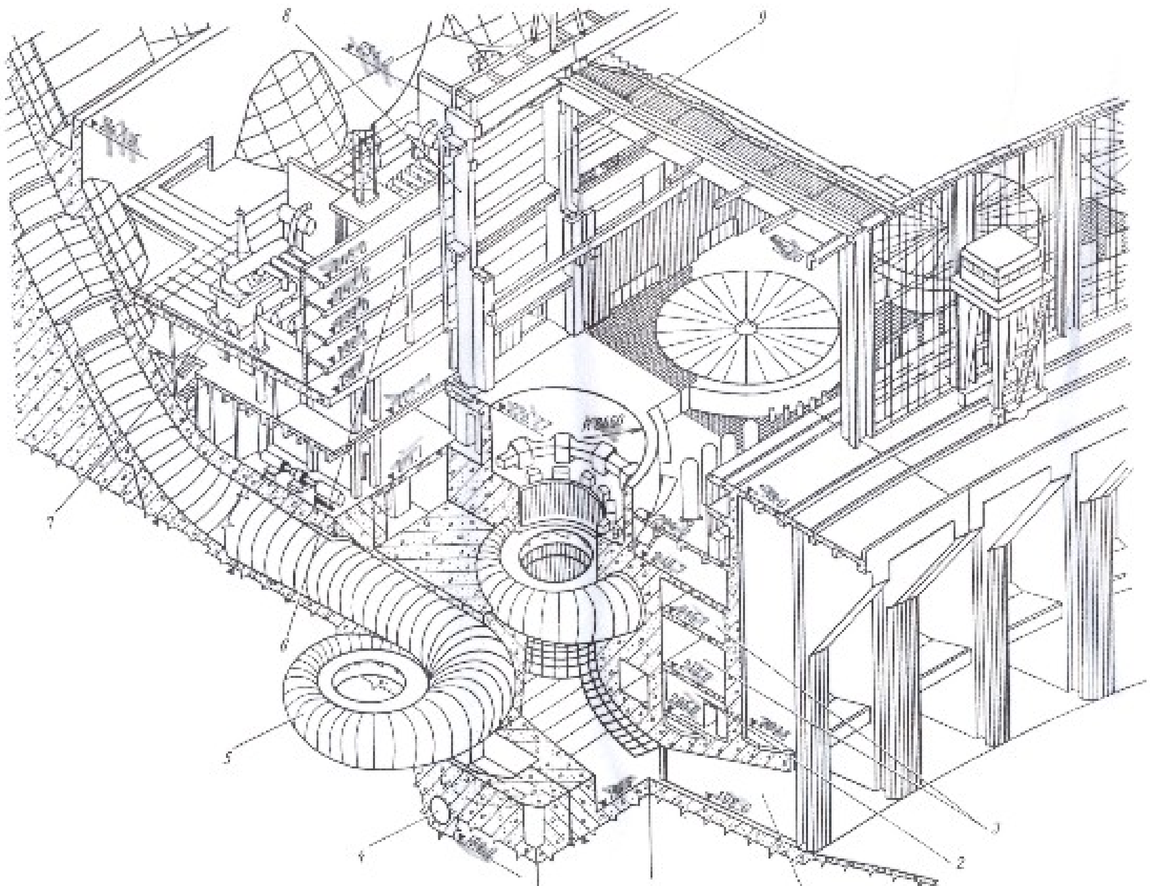
Type of Machine	Head variation percent of rated head (m)	Load variation percent of rated outlet	Specific speed (m-mhp)	Peak Efficiency in percent
Pelton	120 to 80	50 to 100	15 to 65	90
Francis	125 to 65	50 to 100	60 to 400	93
Deriaz	125 to 65	50 to 100	200 to400	92
Kaplan	125 to 65	40 to 100	300 to 800	92
Propeller	110 to 90	90 to 100	300 to 800	92
Bulb	125 to 65	40 to 100	600 to 1200	92

The following points may additionally be noted:

1. The performance of a turbine is ideal at the design head. Fall of efficiency in case of Pelton, Kaplan and Bulb turbines is much less in comparison to Francis-and Propeller types. Therefore in overlapping head ranges selection of type of turbine should consider the head variation existing at site.
2. Turbine efficiency varies with load. Fall of efficiency at part load for Francis and Propeller is much steeper in comparison to that for Kaplan and Pelton turbines. Therefore, necessity of operating turbines at part loads for longer time influences the choice of turbines in the overlapping head ranges. Thus in the head ranges where both Kaplan and Francis are suitable. The requirement of large pressure head and electrical load variation dictates, Kaplan turbine to be superior to Francis turbine from considerations of higher power generation on account of better overall efficiency. Similarly, in the overlapping head ranges where both Francis and Pelton could be used, Pelton has advantages over Francis in overall performance level when variation of load and head is higher.
3. Highest specific speed of turbine resulting in higher speed of rotation for generator with consequent reduction in cost of generator. This criteria is very important for selecting-type of turbine from cost consideration in the overlapping head ranges.

5.3.3 Apurtenant structures for reaction turbines

It was mentioned earlier that the power output of a reaction turbine depends upon the hydro-net pressure head, that is, the effective head available for power generation. This is calculated by deducting the losses in the water conductor system (including penstocks) from the gross head evaluated between the head water or reservoir level and the tail water level. Further, the entire water passage from the intake to the outlet is completely filled with water. In fact, there should not be any water nor the water pressure be so low that it turns into vapour, which may create cavitation damage to the turbine and the water passage. Hence, it is important to design the path of water from its distribution to the runner at the periphery of the stationary part to the existing passage.



1=Draft tube ; 2=De-watering-system pumps; 3=Process water-supply system; 4=Discharge gallery
5=Spcial case; 6=Cable gallery ; 7= Tran sformer; 8,9= Columns

FIGURE 10. Cut-out view through the water passage and powerhouse of a reaction turbine

In Figure 10, a three dimensional cut-out view through the power house of a reaction turbine has been shown. The turbine and generator units have been removed to give a better view of the water passage. It may be observed that the penstock, which is a uniform diameter tube shapes in to a spiraling pipe near the turbine that reduces in size towards its tail end. This section, called the Spiral or Scroll Casing, is provided to distribute the conveyed water all around the periphery of the turbine runner through the stay rings and wicket gates as uniformly as possible. At the bottom, the water passage bends up from vertical to a horizontal alignment, and this passage is known as the draft tube. The functions of a Draft Tube are two-fold: (a) Achieves the recovery of velocity head at runner outlet, which otherwise would have wasted as exit loss, and (b) Allows the turbine to be set at a higher elevation without losing advantage of the elevation difference. The second objective allows easier inspection and maintenance of the turbine runners.

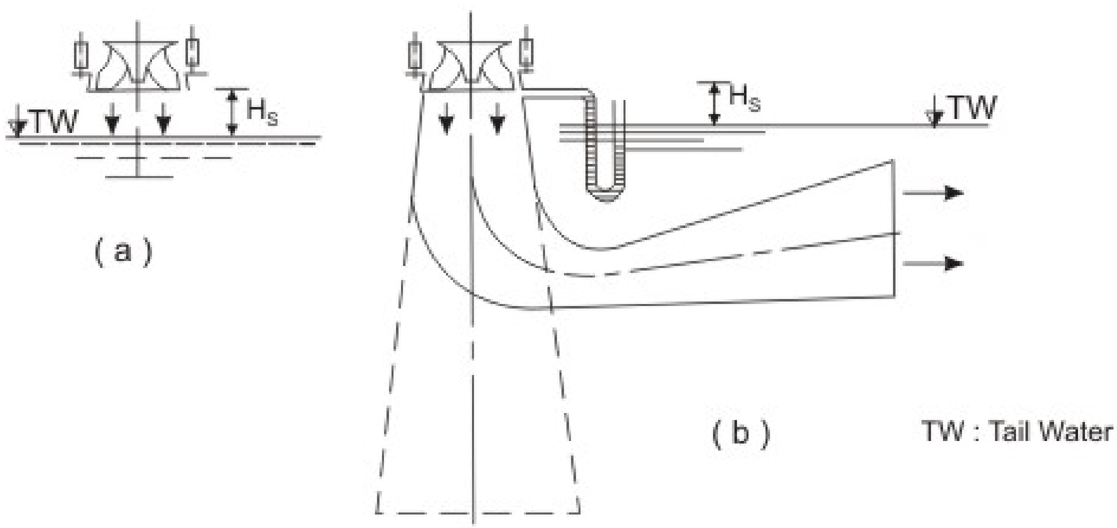


FIGURE 11. Principle of operation of a draft tube

Figure 11 explains the reason why a reaction turbine enjoys the energy available between the runner end and the tail water level (H_s) with the help of a draft tube (Figure11b). In the absence of a draft tube (Figure11a), the head difference (H_s) would have simply been lost. As shown in Figure11b, in the presence of a draft tube, therefore, there exits a sub-atmospheric pressure at the bottom of the runner. It may be noted that though Figure11 shows a typical Francis turbine in place, the draft tube is not very different in case of a propeller or Kaplan turbine, as shown in Figure 12.

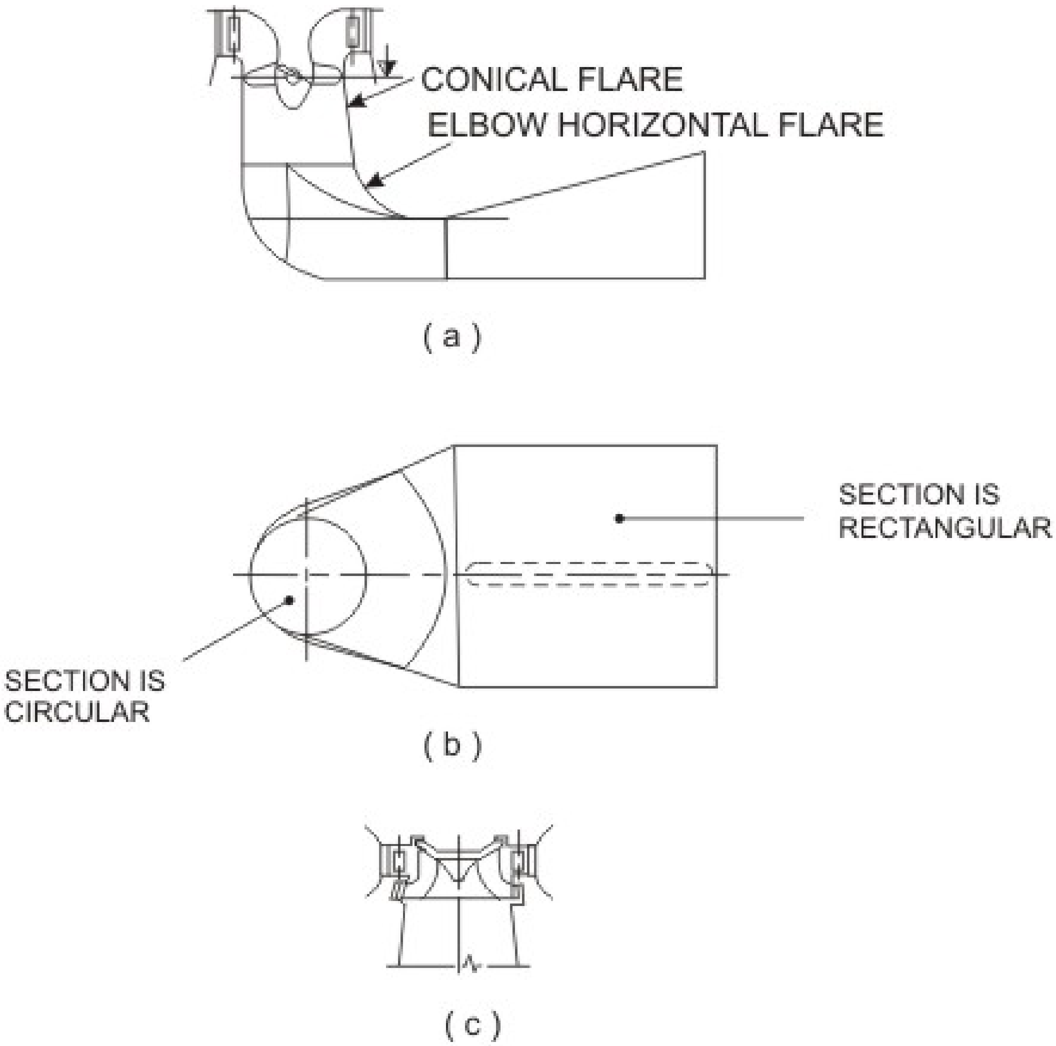


FIGURE 12. Standard draft tube shapes
(a) Vertical section through a Kaplan unit, showing draft tube
(b) Plan of a draft tube
(c) Section through a Francis unit & conical flare

It may be observed from the figure that in the 'Elbow-type' draft tube, the vertical section is circular but gradually changes to a rectangular section as it becomes horizontal. This is because at the end of the draft tube, there is usually a rectangular shaped gate that is sometimes required to shut off tail water during maintenance. However, the change in shape of the elbow increases the turbulent losses in the draft tube. The magnitude of this loss depends upon the magnitude of flow.

A sectional view through the water passage and turbine of the power house shown in Figure 10 would be as shown in Figure 11, which shows a typical Francis runner installed.

Of course, one may replace the runner with a Propeller or Kaplan unit without changing much of the inlet and exit water passages, the spiral casing and draft tubes.

Both the spiral casing and draft tubes have to be designed carefully, in order reduce the head loss as far as possible and also to avoid cavitations. The following Bureau of Indian Standards codes can be referred to for carrying out a detailed design of these units:

IS: 7418-1991 "Criteria for design of spiral casing (concrete and steel)"
IS: 5496-1969 "Guide for preliminary dimensioning and layout of elbow type draft tubes for surface
hydel power stations".

5.3.4 Power house layouts

Classifying by the way the water from the head water pond or reservoir is applied to the turbine, one may differentiate powerhouses as belonging to one of the following categories:

1. In-stream, as for low-head run-of river plants (Figures 13 and 14). Here, the intake head works are incorporated as a part of the power house.

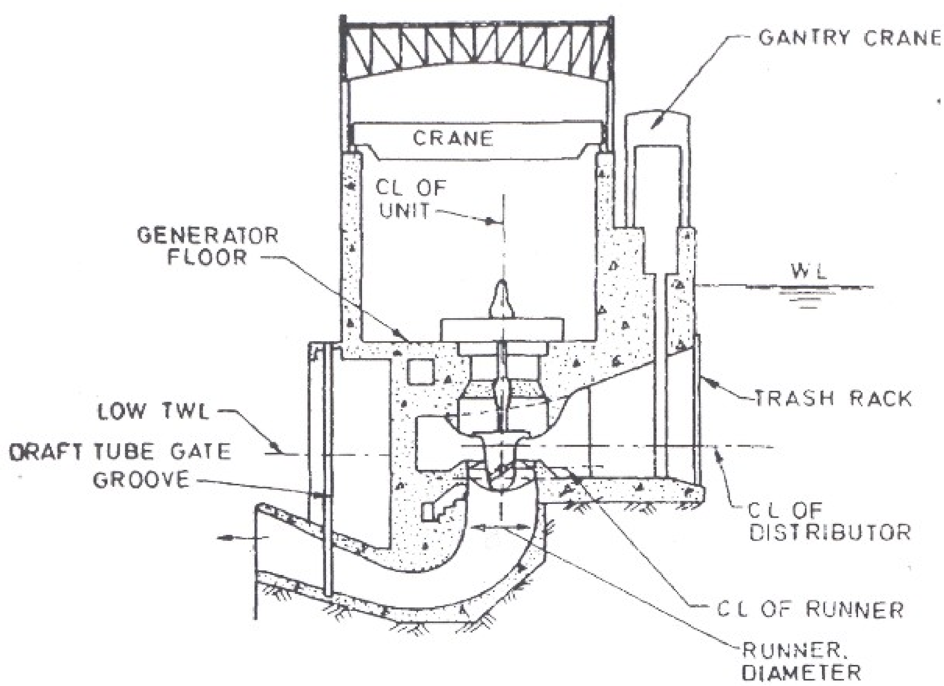


FIGURE 13 . Vertical section through a powerhouse Built as a structure integrated with intake and employing Kaplan turbine

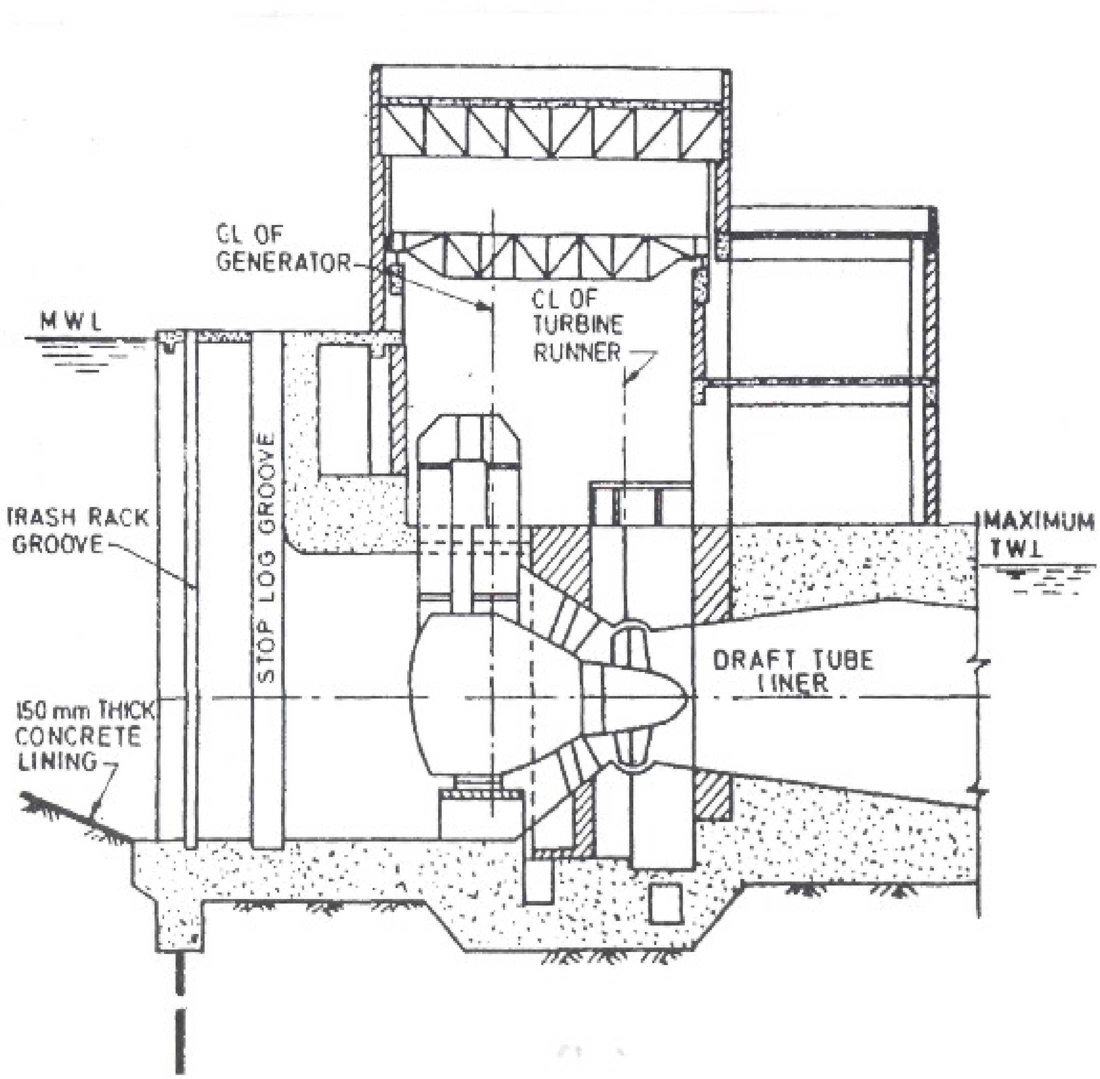
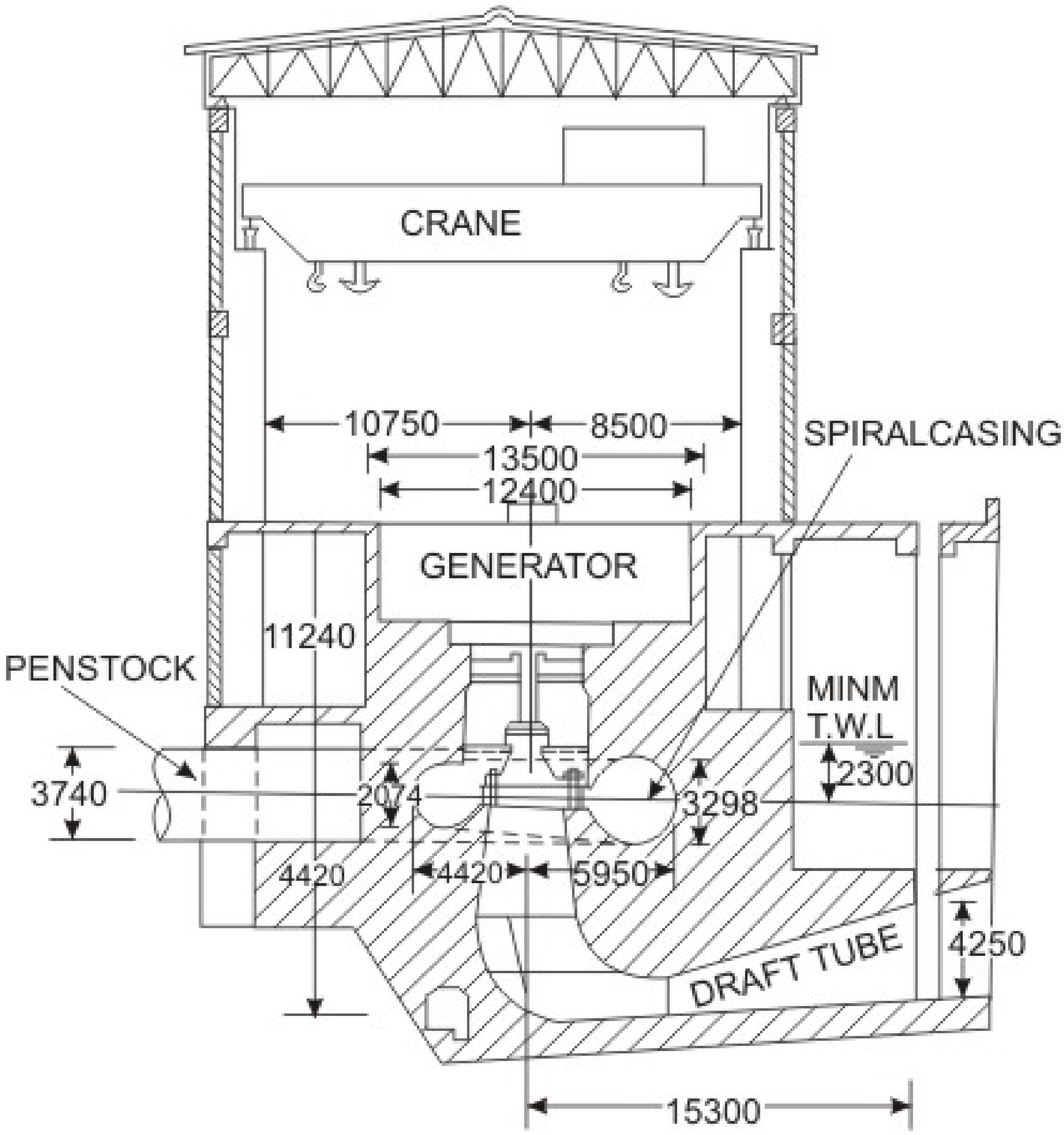


FIGURE 14 . Vertical section through a powerhouse built isolated Built as a structure integrated with intake intake and employing bulb turbine

2. Detached, as for turbines receiving water through an intake from the head water via a penstock (Figure 15).



(c)

All dimensions in milimetres.

FIGURE 15. Vertical section through a powerhouse built isolated from head works (intake) showing typical dimension in mm.The turbine unit may be either Kaplan or Francis

Power houses may also be classified as surface or underground types, depending upon its location with respect to the natural ground surface. An example of each may be cited for the powerhouses of the Sardar Sarovar, which actually consists of two sets, the first being the main river power house, which is underground (Figure 16) and the other at the intake of the canal, which is over ground (Figure 17).

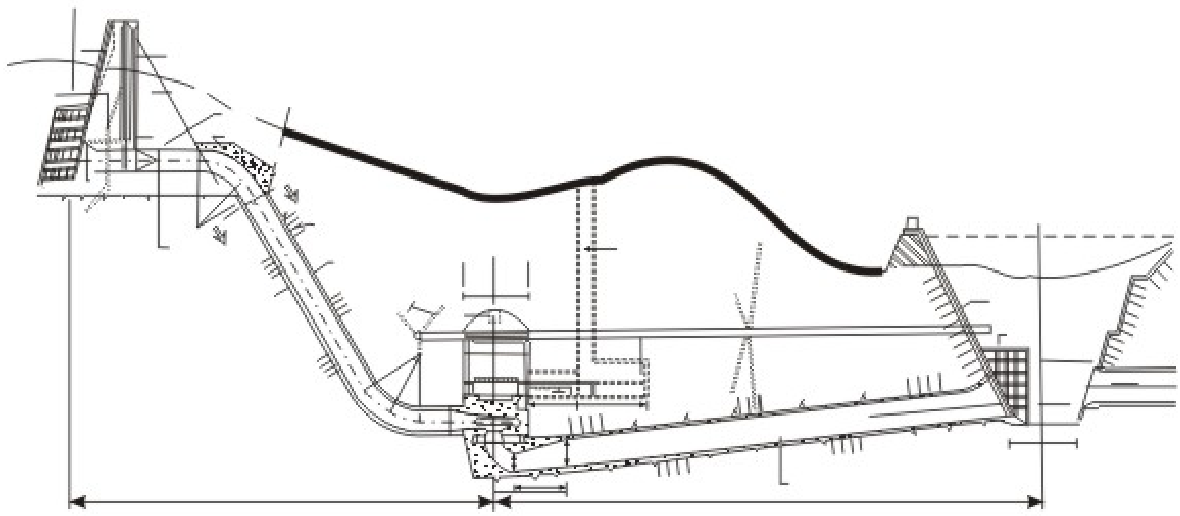


FIGURE 16. Section through riverbed powerhouse underground of Sardar Sarovar Dam project

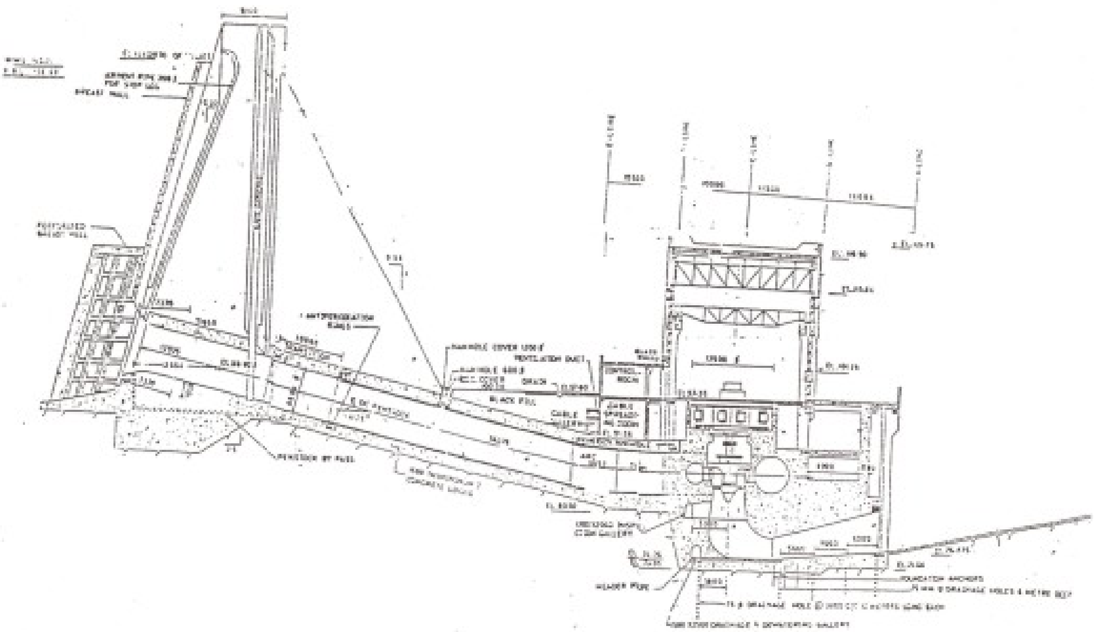
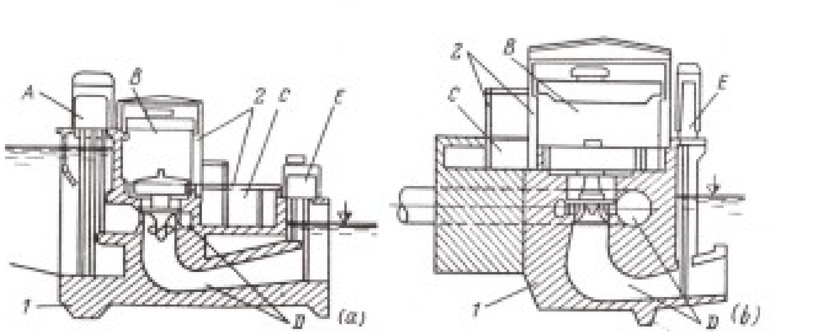


FIGURE 17. Section through canalhead power house surface of Sardar Sarovar Dam project

As such, there is no difference between the two as far as the flow pattern of water and hydraulic dimensions of the spiral case, runner, or draft tube are concerned. Though surface power stations have less space restriction, provision of a strong foundation has to be ensured. Further, they need to be blended to the surroundings to be aesthetically acceptable. The underground power houses, on the other hand, are found to be more economical than an equivalent surface power station due to two reasons. Firstly, about only half the amount of concrete is required compared to a surface power station and, secondly, the total length of water conveyance system may be reduced substantially. Further, since they are not exposed over ground, they are relatively safe from natural hazards like rock fall or man-made hazard like sabotage possibilities. Further, the underground power houses do not harm to the aesthetics of the surrounding landscape. Also, as compared with surface power station, underground stations are affected less by earthquakes.

Power houses, whether in-stream or isolated over ground or underground, use equipment for turbines and generators that have become standardized over the years. This has led to the layout of powerhouse to several basic structures with different functions as shown in Figure 18.



LEGEND:
1-SUBSTRUCTURE ; 2- SUPERSTRUCTURE ;A-JURKE-GATE ROOM
B-GENERATOR ROOM ; C- AUXILIARY ROOM; D-WATER PASSAGEWAYS;
E-DRAFT-TUBE GATE ROOM

FIGURE 18. Structural components of a powerhouse
(a) In-stream ; (b) Isolated

Usually, the power house can be divided into a Substructure (marked 1 in Figure 18) and the Superstructure (marked 2). The substructure contains water passageways (D) and serves as a hydraulic structure and the Superstructure houses the service rooms such as a intake-gate room (A), generator room (B), auxiliary room (C), and draft-tube gate room (E).

A typical plan of the machine hall for a large power house shows the units arranged in row with the service bay, central room and worked usually provided at one end (Figure 19).

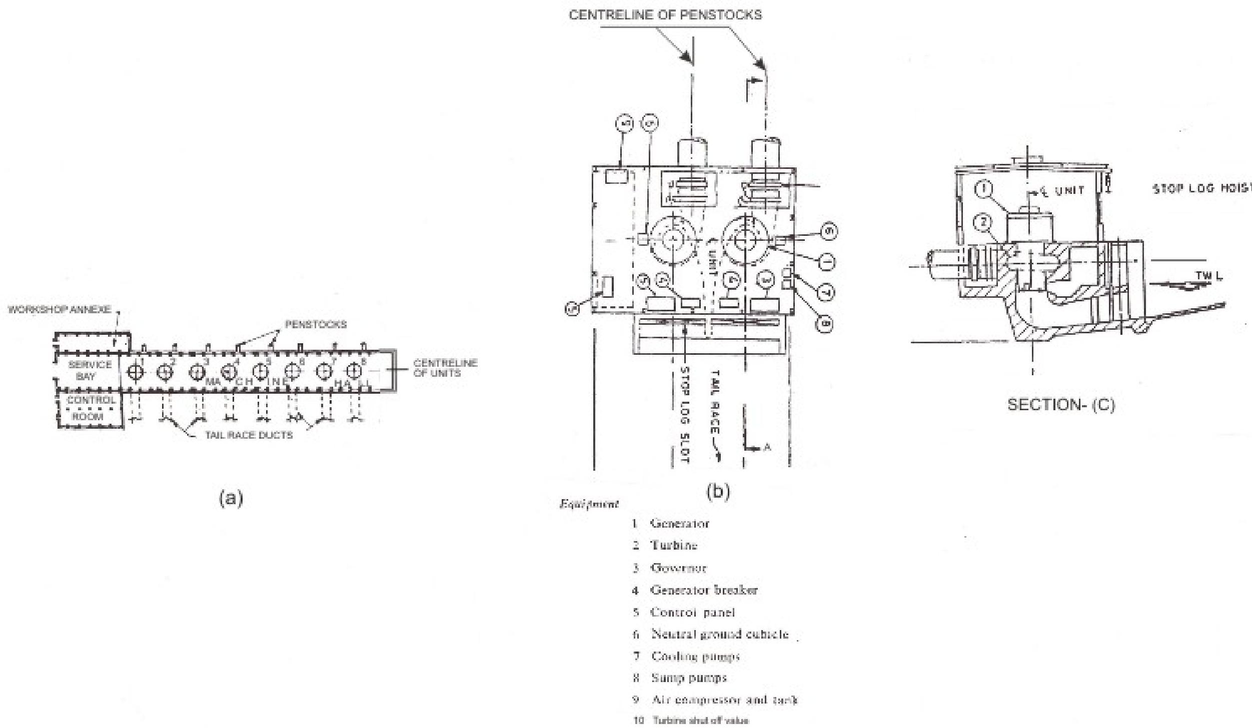


Figure 19. Typical layout of a large power home for Francis or Kaplan turbines
(a) General plan; (b) Detains of two units; (c) Cross section A-A

The length of a power house mainly depends upon the unit spacing, length of erection bay and the length required for the overhead crane to handle the last unit. The total length (L) of the power houses may be determined from the following formula:

$$L = N_0 \times (\text{unit spacing}) + L_s + K$$

where

N_0 = Number of units
L_s = Length of erection bay
K = Length required for overhead gantry crane to handle the last unit

The length of erection bay may be taken as 1.0 to 1.5 times the unit bay size as per erection requirement. The width of the power house depends on the dimensions of the spiral casing and the hydro generator. Superstructure columns which are required to carry the weight of the rails supporting the overhead gantry crane are usually spaced at about 2.0 to 2.5 metres.

The height of the power house from the bottom of the draft-tube to the centerline of spiral casing (H_1 in Figure 20) has to be determined from the dimensions of elbow type draft tube. The thickness of the concrete below the lowest point of draft-tube may be taken from 1.0 to 2.0m depending upon the type of foundation strata, backfill conditions and size of the power house. The height from the center line of the spiral casing up to the top of the generator (H_2 in Figure 20) can be determined from the height of the generator's stator frame and that of the load bearing bracket.

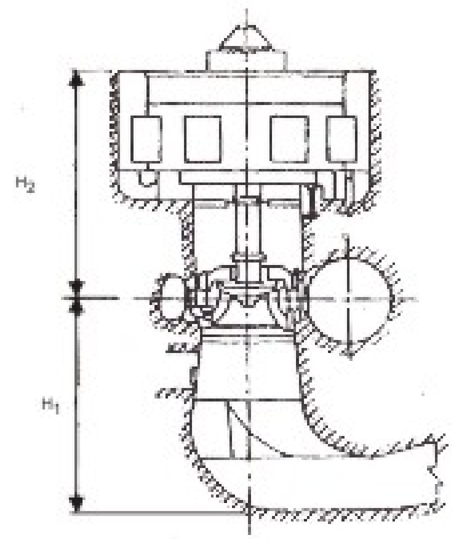
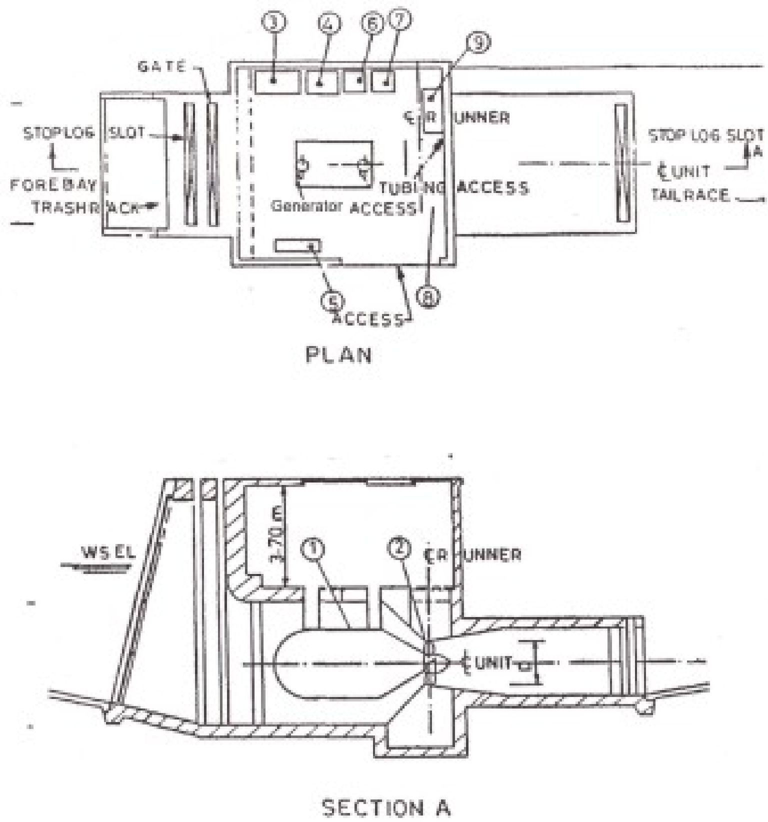


FIGURE 20. Cross section through generating unit showing relative heights of substructure divisions. H_1 : From bottom of draft tube to centreline of spiral case: H_2: From centreline of spiral case to top of generating unit

The height of the machine hall above the top bracket of the generator depends upon the overhead cranes hook level, corresponding crane rail level, and the clearance required between the ceiling and the top of the crane.

The layout of an in-stream power house with a bulb turbine is shown in Figure 21.



Equipment

1 Generator
2 Turbine
3 Governor
4 Generator breaker
5 Control panel
6 Neutral ground cubicle
7 Surge and protection cubicle
8 Sump pumps
9 Air compressor and tank

Figure 21. Layout of an in stream power house with bulb turbine

The detailed dimensions for the any of the above power houses may be done from the following Bureau of Indian Standard codes:

IS: 12800-1993 “Guidelines for selection of turbines, preliminary dimensioning and layout of surface hydro-electric power houses”
Part 1: Medium and large power houses
Part 2: Pumped storage power houses
Part 3: Small, mini and micro hydroelectric power houses

5.3.5 Structural design of power house

As indicated, a hydropower generating station essentially composes of a substructure and a superstructure, when viewed from a vertical section. In plan, each single turbo-generator unit or a few of these units are constructed monolithically as a block with vertical joints between two adjacent block.

Stability and analysis of forces

The stability of the entire power house largely depends upon the construction of the substructure foundation which is in direct contact with water. Primarily, the structure has to be checked for overturning, sliding and uplift pressures. The different expected loads for the substructure of two types of powerhouses-instream and dam based are shown in Figure 22(a) and (b) respectively.

FIGURE 22. Loads to be considered for the analysis of the substructive of a power house
(a) Instream power house; (b) Dam based power house

For the in-stream power house, the principal force here is the upstream (head water pressure) H_u. It is partially balanced by the downstream (tail water pressure) H_d. Sliding is resisted at the base by friction due to the weight of the structure (W) and the weight of water within the power house V_1 and V_2 on the upstream and downstream of the two gates considering the gates to be closed and the scroll case and draft tubes as empty. From the bottom of the structure, the uplift pressure (u) and the net base reaction pressure from the foundation (R) balance the vertical forces. Sliding and overturning checks for the stability of the powerhouse may be then done according to the laws of mechanics. The corresponding stresses calculated at the base may be checked against the safe bearing pressure of the foundation.

For powerhouses not built as an integral part of the headworks (intake), the stability check need not include that of sliding as there is no direct hydrostatic pressure to resist from the upstream over a large surface area as that for in-stream power houses. The upstream water pressure is now considered as that offered by the water within the penstock (P in Figure 22b). Since the substructure is located below the tail water level, the foundation may be considered saturated and the upstream and downstream pressures (H_u and H_d) may be considered same and they cancel each other. The uplift pressure may, therefore, be considered to be uniform throughout. The self weight of the structure (W) and the weight of water on the downstream of the structure (V) are balanced by the base reaction pressure (R).

Structural design and construction

The substructure concrete encasing the draft tube has to support the machinery load over the cavities and has to transmit the same to the foundation such that the pressure at the base remains within permissible bearing pressure of foundation. The substructure and the frames of the power house are sometimes constructed as one single stage of concreting, as shown in Figure 23. The substructure mass concrete portion housing the draft-tube has to be checked for loads and streams which may be done by a three dimensional stress analysis software package. Necessary reinforcement has to be provided according to the analysis.

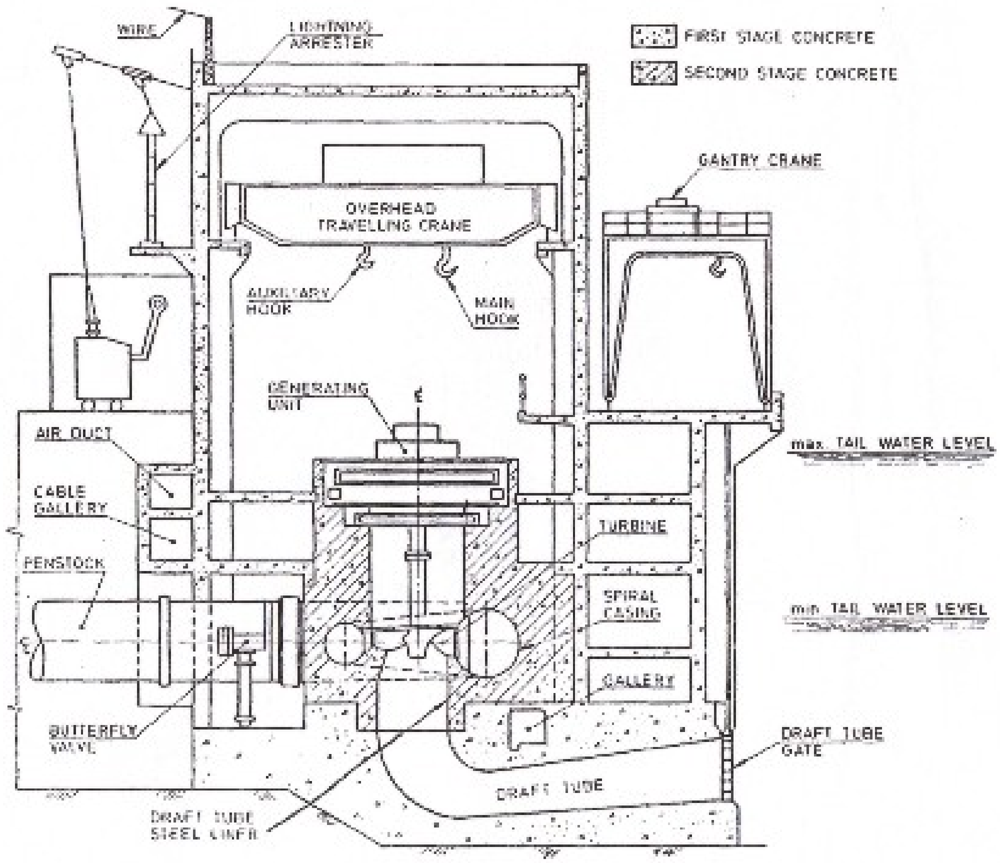


FIGURE 23. Sequence of construction of a power house

Above the substructure concrete encompassing the draft tube lies the concrete housing the scroll case which supports the load of an annular concrete block that acts as the foundation for the heavy generator load. Both these concrete blocks (second stage concrete shown in Figure 23) to be analyzed for forces, static as well as dynamic, caused by the rotation and vibration of the turbines. Reinforcements have to be provided according to the stresses obtained from the analysis, preferably using a three dimensional stress analysis software package. The Bureau of Indian Standards code IS: 7207-1992 "Criteria for design of generator foundation for hydroelectric power stations" may be referred to for guidance on the respective analysis.

The superstructure of a power house consisting the structure associated with the machine hall of the generating units, other places for electrical and auxiliary equipment, and structures to support cranes for servicing the turbo generators during installation and repair. For analyzing the superstructure of a surface power station, the Bureau of Indian Standards code IS: 4247 (Part2)-1992 “Code of practice for structural design of surface hydroelectric power station” may be referred to for further details. For an underground power house, the superstructure components remain much the same, except the roof and walls, which are then taken care of by the power house cavity itself.

Module
6

MANAGEMENT OF WATER RESOURCES

LESSON 1

RIVER TRAINING AND RIVERBANK PROTECTION WORKS

Instructional objectives:

1. The need for river training
2. Guide bunds for restricting the flow path of river waterway
3. Afflux bunds, approach embankments, groynes ,spurs, etc. for river waterway control
4. The issue of riverbank failure and lateral migration of rivers
5. Different modes of bank failure
6. Techniques for bank stabilization

6.1.0 Introduction

For constructing a hydraulic structure across a river, a water resources engineer must also consider the effect of the structure on the hydraulics of the river and the best ways to train the river such that the structure performs satisfactorily and also there is no significant damage to the riverine environment. For example, if a barrage is located within a river, then its length may span from end to end of the river width or could be smaller, if the waterway is so calculated. In the latter case, that is, when the length of a barrage is smaller than the width of a river, then certain auxiliary structures in the form of embankments have to be constructed as shown in Figure 1, known as River Training Works. At times, people residing very close to the flood zones of a river may have to be protected from the river's fury. This is done by providing embankments along the river sides to prevent the river water from spilling over to the inhabited areas.

In order to limit the movement of the bank of a meandering river, certain structures are constructed on the riverbank, which are called riverbank protection works. Sometimes, an embankment like structure, called a Groyne or a Spur, is constructed at right angles to the riverbank and projected into the river for attracting or deflecting the flow of the river towards or away from the riverbank.

This chapter discusses the layout and design of these River Training and Riverbank Protection Works, which can together be termed as aspects of River Engineering. Of course, river engineering includes much more, like dredging to keep the pathway of ships in a river navigable, or techniques of setting up jetties for ships to berth, but they are not discussed in this lesson.

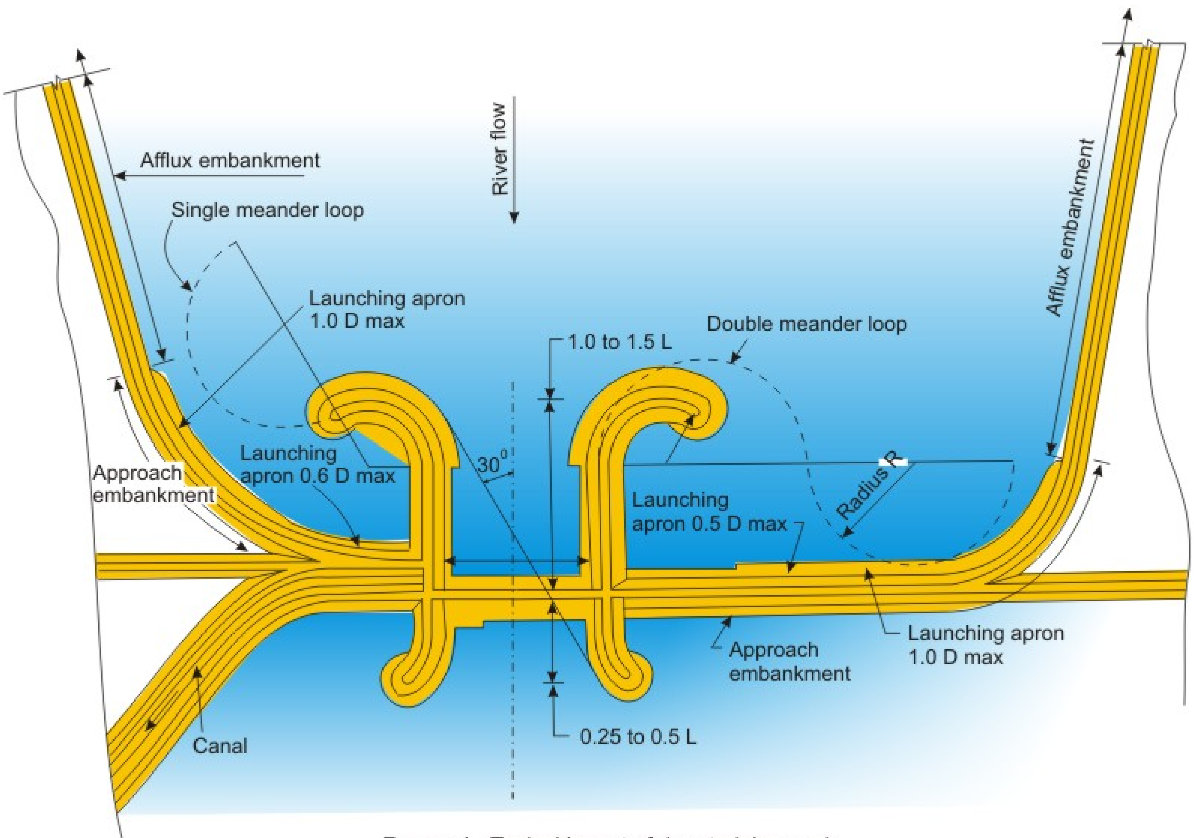


FIGURE 1. Typical layout of river training work.

6.1.1 Guide bunds or banks

Alluvial rivers in flood plains spread over a very large area during floods and it would be very costly to provide bridges or any other structure across the entire natural spread. It is necessary to narrow down and restrict its course to flow axially through the diversion structure. Guide bunds are provided for this purpose of guiding the river flow past the diversion structure without causing damage to it and its approaches. They are constructed on either or both on the upstream and downstream of the structure and on one or both the flanks as required.

Classification of Guide Bunds

Guide bunds can be classified according to their form in plan as (i) divergent, (ii) convergent, and (iii) parallel and according to their geometrical shape as straight and elliptical with circular or multi-radii curved head. These are shown in Figure 2.

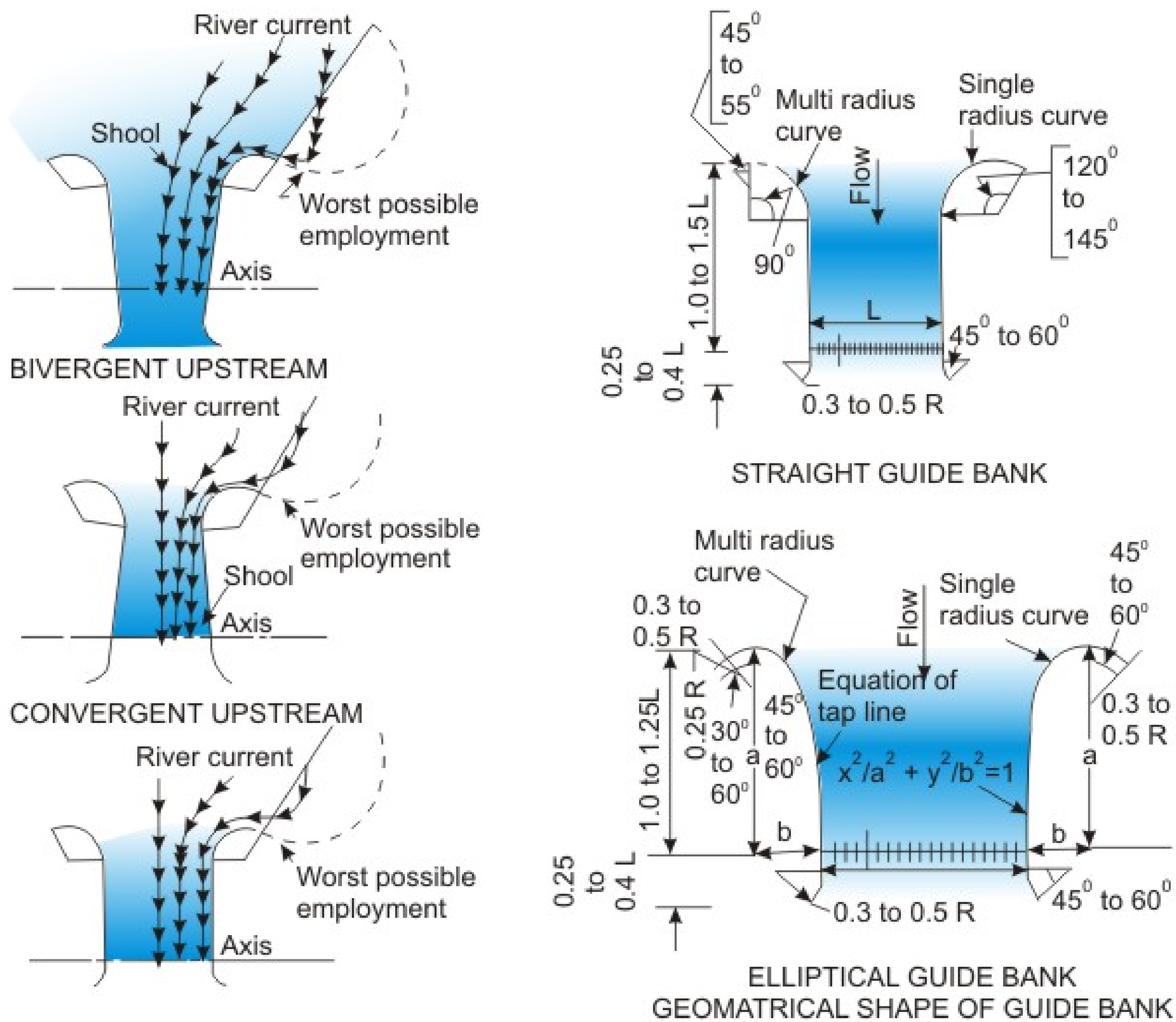


FIGURE 2. Types of guide banks and typical dimensions.

In the case of divergent guide bunds, the approach embankment gets relatively less protection under worst possible embayment and hence divergent guide bunds require a longer length for the same degree of protection as would be provided by parallel guide bunds. They also induce oblique flow on to the diversion structure and give rise to tendency of shoal formation in the centre due to larger waterway between curved heads. However, in the case of oblique approaching flow, it becomes obligatory to provide divergent guide bunds to keep the flow active in the spans adjacent to them.

The convergent guide bunds have the disadvantage of excessive attack and heavy scour at the head and shoaling all along the shank rendering the end bays inactive.

Parallel guide bunds with suitable curved head have been found to give uniform flow from the head of guide bunds to the axis of the diversion structure.

In the case of elliptical guide bunds, due to gradual change in the curvature, the flow is found to hug the bunds all along their lengths whereas in the case of straight guide bunds, separation of flow is found to occur after the curved head, leading to obliquity of flow. Elliptical guide bunds have also been found to provide better control on development and extension of meander loop towards the approach embankment.

Length of Guide Bunds

The length of the guide bund on the upstream is generally kept as 1.0 to 1.5L where L is the width between the abutments of the diversion structure. In order to avoid heavy river action on the guide bunds, it is desirable to limit the obliquity of flow to the river axis not more than 30^0 as indicated in Figure 1. The length of the downstream guide bund is kept as 0.25L to 0.4L.

For wide alluvial belt, the length of guide bunds is decided from two important considerations, viz. the maximum obliquity of the current and the permissible limit to which the main channel of the river can be allowed to flow near the approach embankment in the event of the river developing excessive embayment behind the training works. The radius of the worst possible loop has to be ascertained from the data of the acute loops formed by the river during the past. Where river survey is not available, the radius of the worst loop can be determined by dividing the radius of the average loop worked out from the available surveys of the river by 2.5 for rivers having a maximum discharge up to 5000 cumecs and by 2.0 for a maximum discharge above 5000 cumecs.

Curved head and tail of Guide Bunds

The upstream curved head guides the flow smoothly and axially to the diversion structure keeping the end spans active. The radius of the curved head should be kept as small as possible consistent with the proper functioning of the guide bund. The downstream curved tail provides a smooth exit of flow from the structure.

From the hydraulic model tests conducted for a number of projects over the past years, it has been found that a radius of the curved head equal to 0.4 to 0.5 times the width of the diversion structure between the abutments usually provides satisfactory performance. The minimum and maximum values could be 150 m and 600 m respectively. However, the exact values are to be ascertained from model tests. The radius of the curved tail generally ranges from 0.3 to 0.5 times the radius of the curved head.

According to the river curvature, the angle of sweep of curved upstream head ranges from 120^0 to 145^0. The angle for the curved tail usually varies from 45^0 to 60^0.

In the case of elliptical guide bunds, the elliptical curve is provided upto the quadrant of the ellipse and is followed by multi-radii or single radius circular curve. In case of multi-radii curved head, the larger radius adjacent to the apex of the ellipse is generally kept as 0.3 to 0.5 times the radius of the curved head for straight guide bund with the angle of sweep varying from 45^0 to 60^0 and the smaller radius equivalent to 0.25 times the radius of curve head for straight guide bund with sweep angle of 30^0 to 40^0.

Design of guide bunds

After fixing up the layout of the guide bunds in accordance with the guidelines mentioned in the foregoing paragraphs, the details of the guide bund sections have to be worked out. The various dimensions worked out are top width, free board, side slopes, size of stone for pitching, thickness of pitching, filters and launching apron. The guide lines for the same are given below.

Top width of guide bund

At the formation level, the width of the shank of guide bunds is generally kept 6 to 9 m to permit carriage of material and vehicles for inspection. At the nose of the guide bunds, the width is increased suitably in a bulb shape to enable the vehicles to take turn and also for stacking reserve of stone to be dumped in places whenever the bunds are threatened by the flow.

Free board for Guide Bund

A free board of 1 to 1.5 m above the following mentioned two water levels has to be provided and the higher value adopted as the top level of the upstream guide bund:

(i) Highest flood level for 1 in 500 years flood

(ii) Affluxed water level in the rear portion of the guide bank calculated after adding velocity head to HFL corresponding to the design flood (1in 100 year frequency) at the upstream nose of the guide bank.

On the downstream side also, a free board of 1 to 1.5 m above the highest flood level for 1 in 500 years flood is to be adopted.

Side slopes of guide bund

The side slopes of guide bund have to be fixed from stability considerations of the bund which depend on the material of which the bund is made and also its height. Generally the side slopes of the guide bund vary from 2:1 to 3:1 (H:V).

Size of stone for pitching

The sloping surface of the guide bund on the water side has to withstand erosive action of flow. This is achieved by pitching the slope manually with stones. The size and weight of the stones can be approximately determined from the curves given in Figure 3. It is desirable to place the stones over filters so that fines do not escape through the interstices of the pitching. For average velocities up to 2 m/sec, burnt clay brick on edge can be used as pitching material. For an average velocity upto 3.5 m/sec, pitching of stone weighing from 40 to 70 kg (0.3 to 0.4 m in diameter) and for higher velocities, cement concrete blocks of depth equal to the thickness of pitching can be used. On the rear side, turfing of the slope is normally found to be adequate.

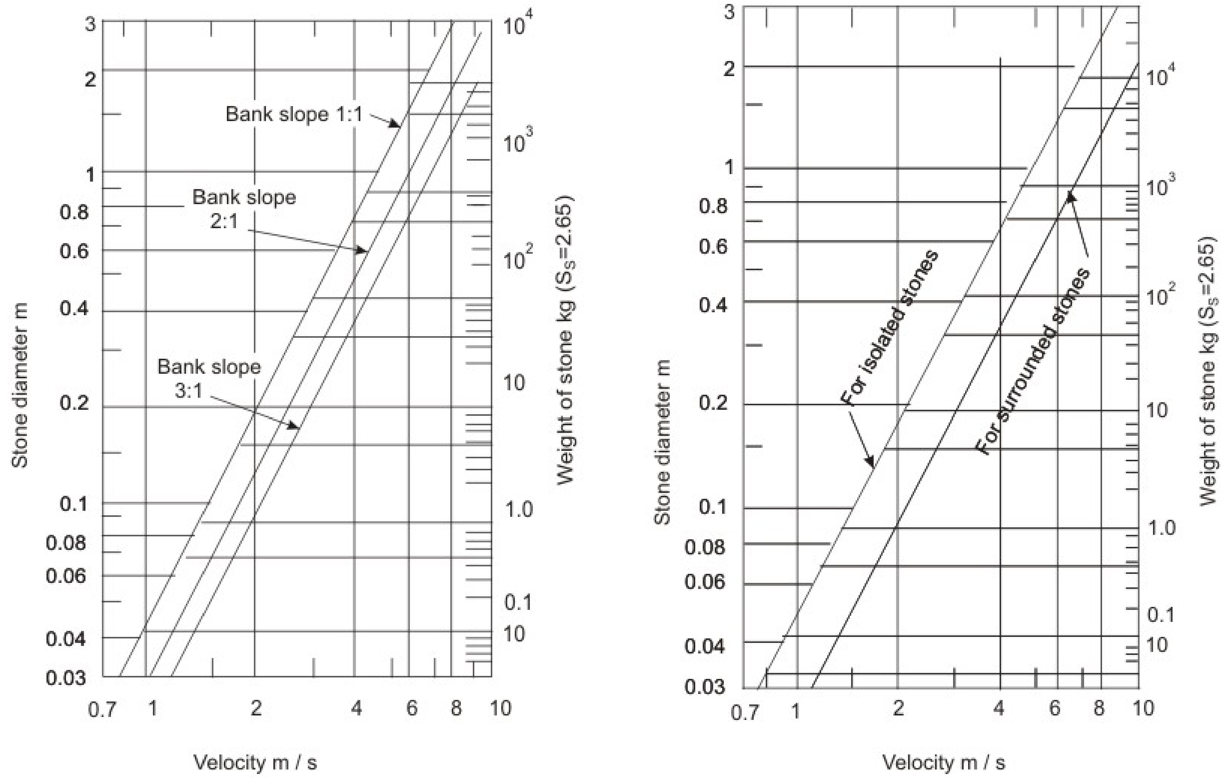


Figure 3. Graph for determining pitching and stone aprons.

Thickness of Pitching

The thickness of pitching is to be kept equal to the size of the stone for pitching determined. However, it should not be less than 0.25m. wherever the velocities are high for which the size of stone is greater than 0.4 m, cement concrete blocks of thickness 0.4 to 0.5 or 0.6 m may be used.

Provision of filter

It is always desirable to provide an inverted (graded) filter below the pitching stones to avoid the finer bund materials getting out through the interstices. The thickness of the filter may be 20 to 30 cm. Filter has to satisfy the criteria with respect to the next lower size and with respect to the base material:

(i) For uniform grain size filter,

$$R_{50} = \frac{D_{50} \text{ of filter material}}{D_{50} \text{ of base material}} = 5 \text{ to } 10$$

(ii) For graded material of sub-rounded particles,

$$R_{50} = \frac{D_{50}\ of\ filter\ material}{D_{50}\ of\ base\ material} = 12\ to\ 58$$

$$R_{15} = \frac{D_{15}\ of\ filter\ material}{D_{15}\ of\ base\ material} = 12\ to\ 40$$

Launching apron

Just as launching apron is provided for the main structure both on the upstream and downstream it has to be provided for guide bunds also in the bed in continuation of the pitching. The different aspects to be looked into are the size of the stones, depth of scour, thickness, slope of launched apron, shape and size of launching apron.

The required size of stone for the apron can be obtained from the curves. In case of non-availability of required size of stones, cement concrete blocks or stone sausages, prepared with 4 mm GI wire in double knots and closely knit and securely tied, may be used.

The scour depths to be adopted in the calculations for the launching apron would be different along the length of the guide bund from upstream to downstream, as given in the following table. The value of R, that is the normal depth of scour below High Flood Level may be determined according to Lacey's scour relations.

Location	Maximum scour depth to be adopted
Upstream curved head of Guide bund	2.5 R
Straight reach of guide bund to nose of downstream Guide bund	1.5 R

While calculating the scour values, the discharge corresponding to 50 to 100 years frequency may be adopted. However, after construction and operation of the diversion structure, the portions of the guide bund coming under attack of the river flow should be carefully inspected and strengthened as and when necessary.

The thickness of apron of the guide bund should be about 25 to 50 percent more than that required for the pitching. While the slope of the launched apron for calculation of the quantity can be taken as 2:1 for loose boulders or stones, it may be taken as 1:5:1 for c.c blocks or stone sausages.

From the behaviour of the guide bunds of previously constructed diversion structures, it has been observed that shallow and wide aprons launch evenly if the scour takes place rapidly. If the scour is gradual, the effect of the width on the launching of apron is marginal. Generally a width of 1.5 R has been found to be satisfactory. For the shank or straight portions of the guide bunds, the thickness of the apron may be kept uniform at 1.5 T where T is the thickness of the stone pitching. To cover a wider area, for the curved head, the thickness is increased from 1.5 to 2.25 T with suitable transition over a length of L_1 equal to one fourth of the radius of the curved head and provided in the shank portion only. On the rear side of the curved head and nose of the guide bund, the apron should be turned and ended in a length equal to about one fourth of the respective radius.

6.1.2 Afflux bund

Afflux bunds extend from the abutments of guide bunds (usually) or approach bunds as the case may be. The upstream afflux bunds are connected to grounds with levels higher than the afflux highest flood level or existing flood embankments, if any. The downstream afflux bunds, if provided, are taken to such a length as would be necessary to protect the canal/approach bunds from the high floods.

Afflux bunds are provided on upstream and downstream to afford flood protection to low lying areas as a result of floods due to afflux created by the construction of bridge/structure and to check outflanking the structure.

Layout of afflux bund

The alignment of the afflux bund on the upstream usually follows the alluvial belt edge of the river if the edges are not far off. In case the edges are far off, it can be aligned in alluvial belt, but it has to be ensured that the marginal embankment is aligned away from the zone of high velocity flow. Since the rivers change their course, it is not necessary that a particular alignment safe for a particular flow condition may be safe for a changed river condition. Hence the alignment satisfactory and safe for a particular flow condition (constructed initially) has to be constantly reviewed after every flood and modified, if necessary.

Top width of afflux bund

Generally the top width of the afflux bund is kept as 6 to 9 m at formation level.

Free board for afflux bund

The top level of the afflux bund is fixed by providing free board of 1 to 1.5 m over the affluxed highest flood level for a flood of 1 in 500 years frequency.

Slope pitching and launching apron

Generally the afflux bunds are constructed away from the main channel of the river. Hence they are not usually subjected to strong river currents. In such cases, provision of slope pitching and launching apron are not considered necessary. However, it is desirable to provide a vegetal cover or turfing. In reaches where strong river currents are likely to attack the afflux bunds, the slopes may be pitched as for the guide bunds.

A typical layout and section of afflux bund are shown in Figure 4.

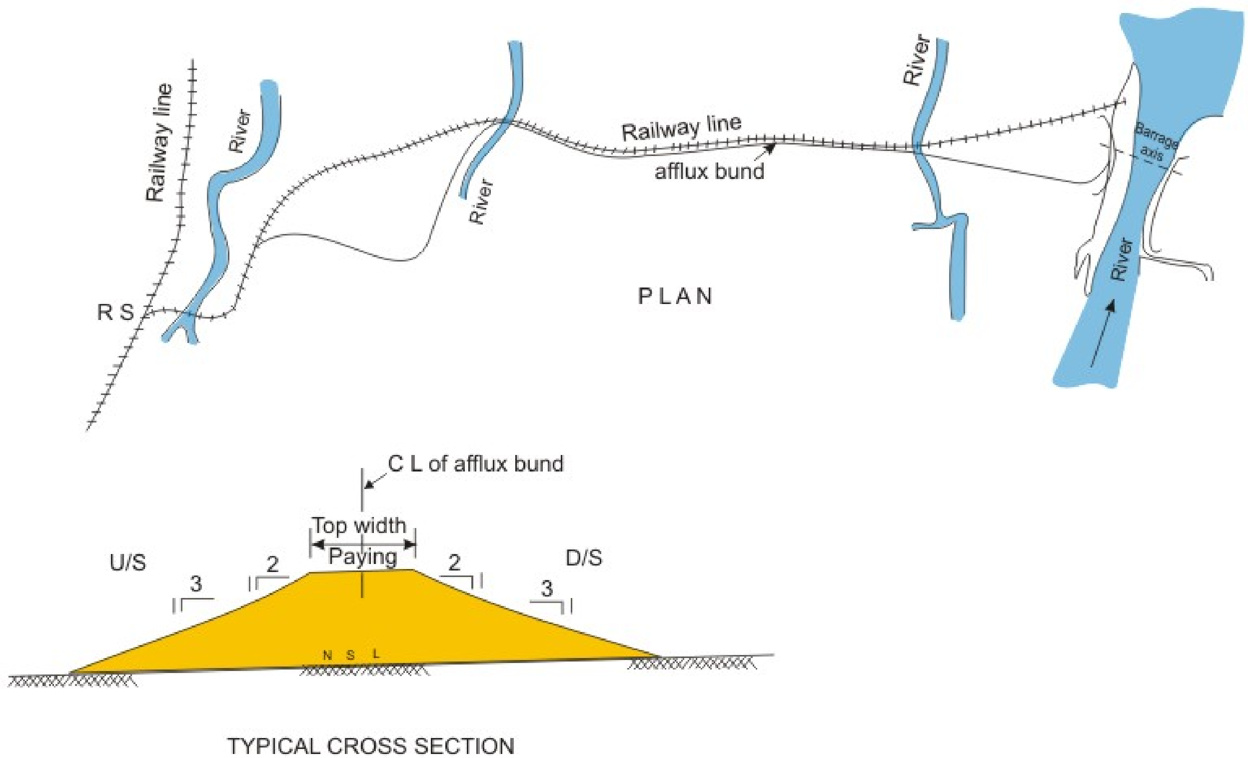


FIGURE 4. Typical layout and section of an afflux bund.

6.1.3 Approach embankment

Where the width of the river is very wide in an alluvial plain, the diversion structure is constructed with a restricted waterway for economy as well as better flow conditions. The un-bridged width of the river is blocked by means of embankments called Approach embankments or tie bunds.

Layout of approach embankment

In case of alluvial plains, the river forms either a single loop or a double loop depending upon the distance between the guide bunds and the alluvial belt edges. Hence the approach embankments on both the flanks should be aligned in line with the axis of the diversion structure up to a point beyond the range of worst anticipated loop. Sometimes the approach embankments may be only on one flank depending on the river configuration.

Top Width of approach embankments

The top width of the approach embankment is usually kept as 6 to 9m at formation level.

Free Board of approach embankment

Free board for approach embankment may be provided similar to that for guide bunds.

Side slopes of approach embankment

The side slopes of the approach embankment have to be fixed from stability considerations of the bund which depend on the material of which the bund is made and also its height. Generally the side slopes of the guide bund vary from 2:1 to 3:1 (H:V).

Size of stone for pitching

Velocities for 40 percent of the design discharge would be estimated and the size of stone for pitching would be determined as for guide bunds discussed in Section 6.1.1.

Thickness of pitching

The Guide lines for determining the thickness of pitching would be the same as for guide bunds in Section 6.1.1. The velocities would be estimated for 40 percent of the design discharge.

Provision of filter

Generally filters are not provided below the pitching stones in the case of approach embankments. However, if the section of embankment is heavy, filter may be provided as mentioned for guide bunds discussed in Section 6.1.1.

Launching apron

The provisions of size of stone, thickness of apron and slope of launched apron would be similar to those of guide bunds mentioned in above paragraphs. But the depth of scour for the approach embankment may be taken as 0.5 to 1.0 D_{max} and beyond that the width may be increased to 1.0 D_{max} with suitable transition in the former reach.

6.1.4 Groynes or Spurs

Groynes or spurs are constructed transverse to the river flow extending from the bank into the river. This form of river training works perform one or more functions such as training the river along the desired course to reduce the concentration of flow at the point of attack, creating a slack flow for silting up the area in the vicinity and protecting the bank by keeping the flow away from it.

Classification of Groynes or spurs

Groynes or spurs are classified according to (i) the method and materials of construction (ii) the height of spur with respect to water level (iii) function to be performed and (iv) special types which include the following:
These are

(i) Permeable or impermeable
(ii) Submerged or non-submerged
(iii) Attracting, deflecting repelling and sedimenting and
(iv) T-shaped (Denehey), hockey (or Burma) type, kinked type, etc.

The different types of spurs are shown in Figure 5.

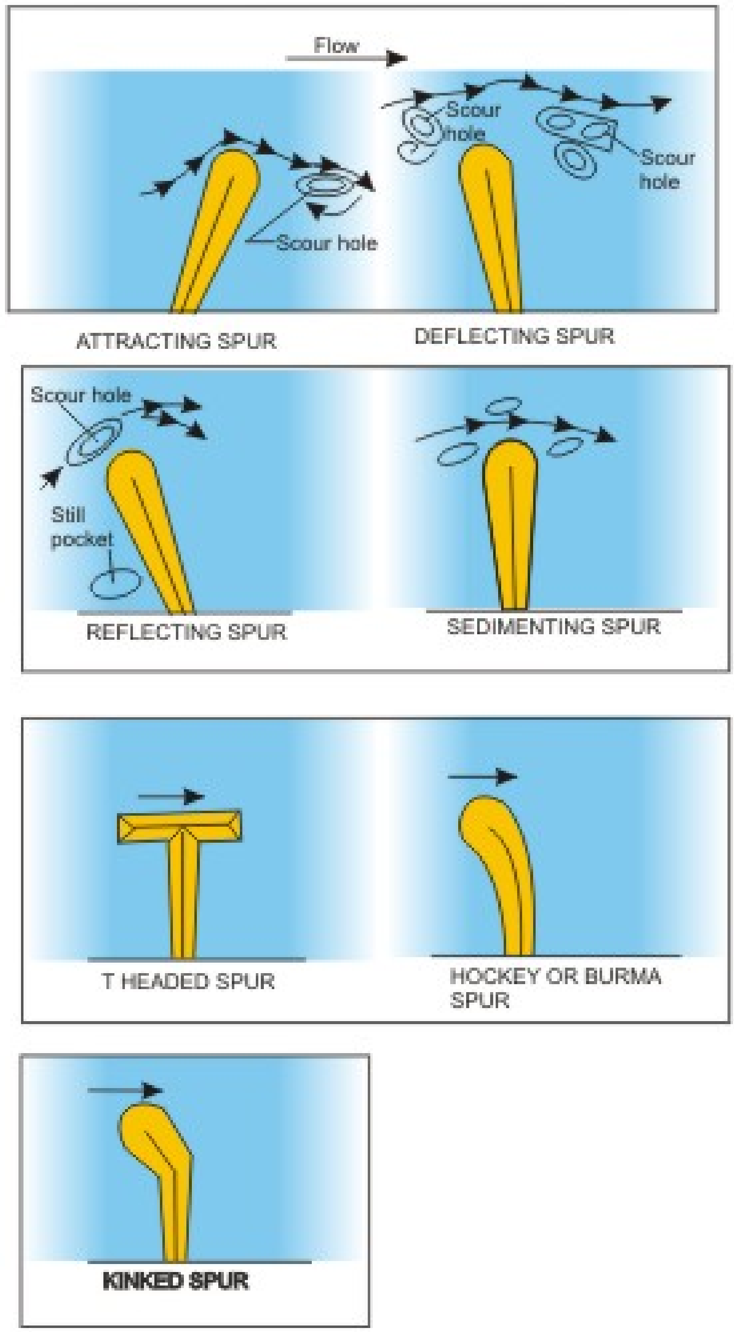


FIGURE 5. Different types of spur.

Impermeable spurs do not permit appreciable flow through them whereas permeable ones permit restricted flow through them. Impermeable spurs may be constructed of a core of sand or sand and gravel or soil as available in the river bed and protected on the sides and top by a strong armour of stone pitching or concrete blocks. They are also constructed of balli crates packed with stone inside a wire screen or rubble masonry.

While the section has to be designed according to the materials used and the velocity of flow the head of the spur has to have special protection.

Permeable spurs usually consist of timber stakes or piles driven for depths slightly below the anticipated deepest scour and joined together to form a framework by other timber pieces and the space in between filled up with brush wood or branches of trees. The toe of the spur would be protected by a mattress of stones or other material. As the permeable spurs slow down the current, silt deposition is induced. These spurs, being temporary in nature, are susceptible to damage by floating debris. In bouldery or gravelly beds, the spurs would have to be put up by weighing down timber beams at the base by stones or concrete blocks and the other parts of the frame would then be tied to the beams at the base.

Layout of groynes or spurs

Groynes are much more effective when constructed in series as they create a pool of nearly still water between them which resists the current and gradually accumulates silt forming a permanent bank line in course of time. The repelling spurs are constructed with an inclination upstream which varies from 10^0 to 30^0 to the line normal to the bank. In the T-shaped groynes, a greater length of the cross groyne projects upstream and a smaller portion downstream of the main groyne.

Length of Groynes

The length of groynes depends upon the position of the original bank line and the designed normal line of the trained river channel. In easily erodible rivers, too long groynes are liable to damage and failure. Hence, it would be better to construct shorter ones in the beginning and extend them gradually as silting between them proceeds. Shorter and temporary spurs constructed between long ones are helpful in inducing silt deposition.

Spacing of Groynes

Each groyne can protect only a certain length and so the primary factor governing the spacing between adjacent groynes is their lengths. Generally, a spacing of 2 to 2.5 times the length of groynes at convex banks and equal to the length at concave banks is adopted. Attempts to economise in cost by adopting wider spacings with a view to insert intermediate groynes at a later date may not give the desired results as the training of river would not be satisfactory and maintenance may pose problems and extra expenditure. T-shaped groynes are generally placed 800 m apart with the T-heads on a regular curved or straight line.

Design of groynes or spurs

The design of groynes or spurs include the fixation of top width, free board, side slopes, size of stone for pitching, thickness of pitching, filter and launching apron.

Top width of spur

The top width of the spur is kept as 3 to 6 m at formation level.

Free board

The top level of the spur is to be worked out by giving a free board of 1 to 1.5 m above the highest flood level for 1 in 500 year flood or the anticipated highest flood level upstream of the spur, whichever is more.

Side slopes

The slopes of the upstream shank and nose is generally kept not steeper than 2:1 the downstream slope varies from 1.5 : 1 to 2:1.

Size of stone for pitching

The guide lines for determining the size of stone for pitching for guide bunds hold good for spurs also.

Thickness of pitching

The thickness of pitching for spurs may be determined from the formula $T = 0.06\ Q^{1/3}$ where Q is the design discharge in cumecs. The thickness of stone need not be provided the same through-out the entire length of the spur. It can be progressively reduced from the nose.

Provision of filters

Provision of filter satisfying the filter criteria has to be made below the pitching at nose and on the upstream face for a length of 30 to 4 m for the next 30 to 45m from the nose. The thickness of the same may be 20 to 30cm. The thickness of filter for the next 30 to 45m on the upstream face may be reduced to about 15 cm and beyond that, it can be omitted.

A typical layout of a spur is shown is Figure 6.

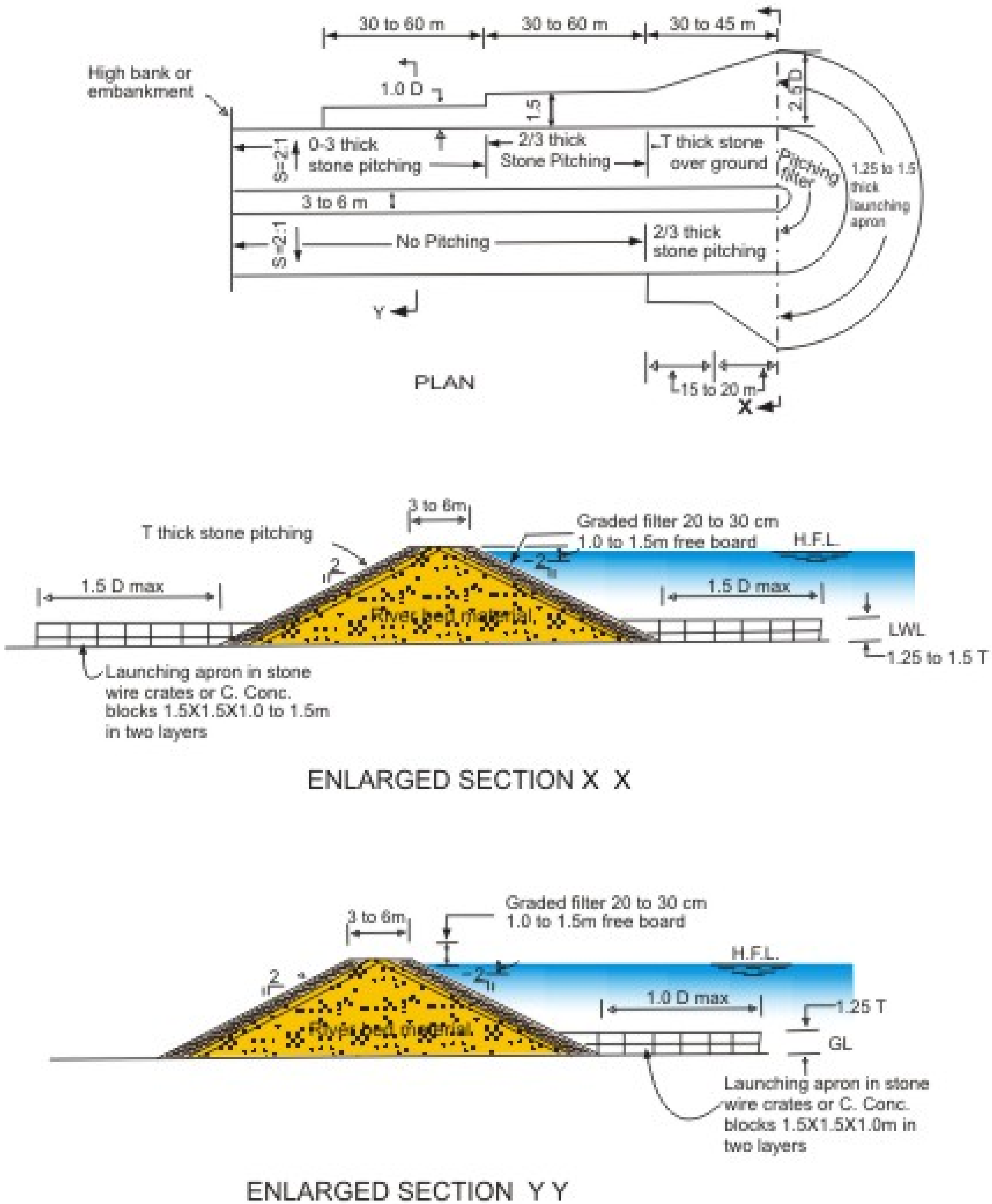


FIGURE 6. Typical layout and section of spur.

6.1.5 Cut-offs

Cut-offs as river training works are to be carefully planned and executed in meandering rivers. The cut-off is artificially induced with a pilot channel to divert the river from a curved flow which may be endangering valuable land or property or to straighten its approach to a work or for any other purpose. As the cut-off shortens the length of the river, it is likely to cause disturbance of regime upstream and downstream till readjustment is made. A pilot cut spreads out the period of readjustment and makes the process gradual. Model tests come in handy in finalising this form of river training works wherever needed.

A typical instance of a cutoff is shown in Figure 7.

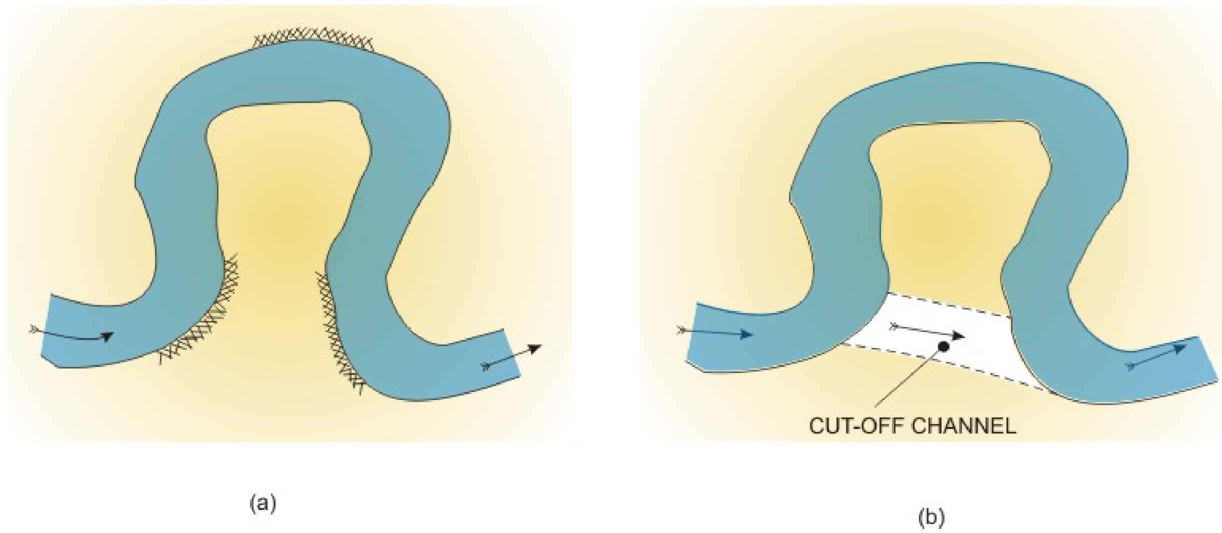


FIGURE 7.a Meandering river with possible threat to bank erosion (marked X-X...)
b An engineered cut-off channel.

6.1.6 Marginal embankments

These are earthen embankments, also known as levees, which are constructed in the flood plains of a river and run parallel to the river bank along its length. The aim of providing these embankments is to confine the river flood water within the cross section available between the embankments. The flood water of a river is thus not allowed to spill over to the flood plains, as normally would had been (Figure 8). This kind of protection against flooding has been provided for most of the rivers of India that are flood prone with low banks and have extensive flood plains in the last century. This may be apparent from the maps of any riverine area, as shown typically in Figure 9.

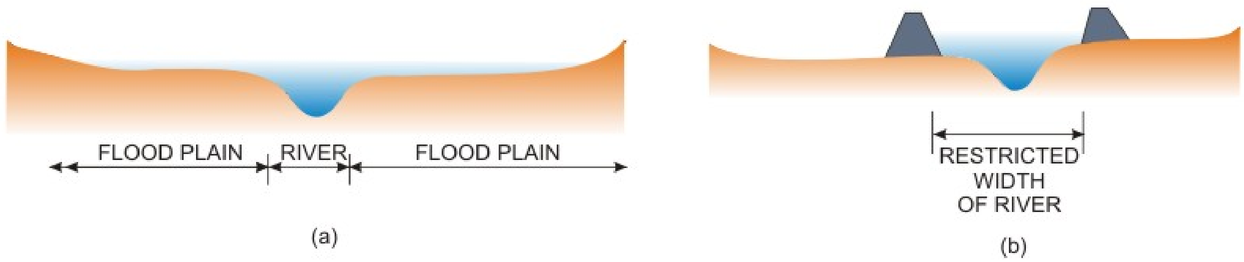


FIGURE 8. a A river in its original state during flood.
b A river embanked , during flood.

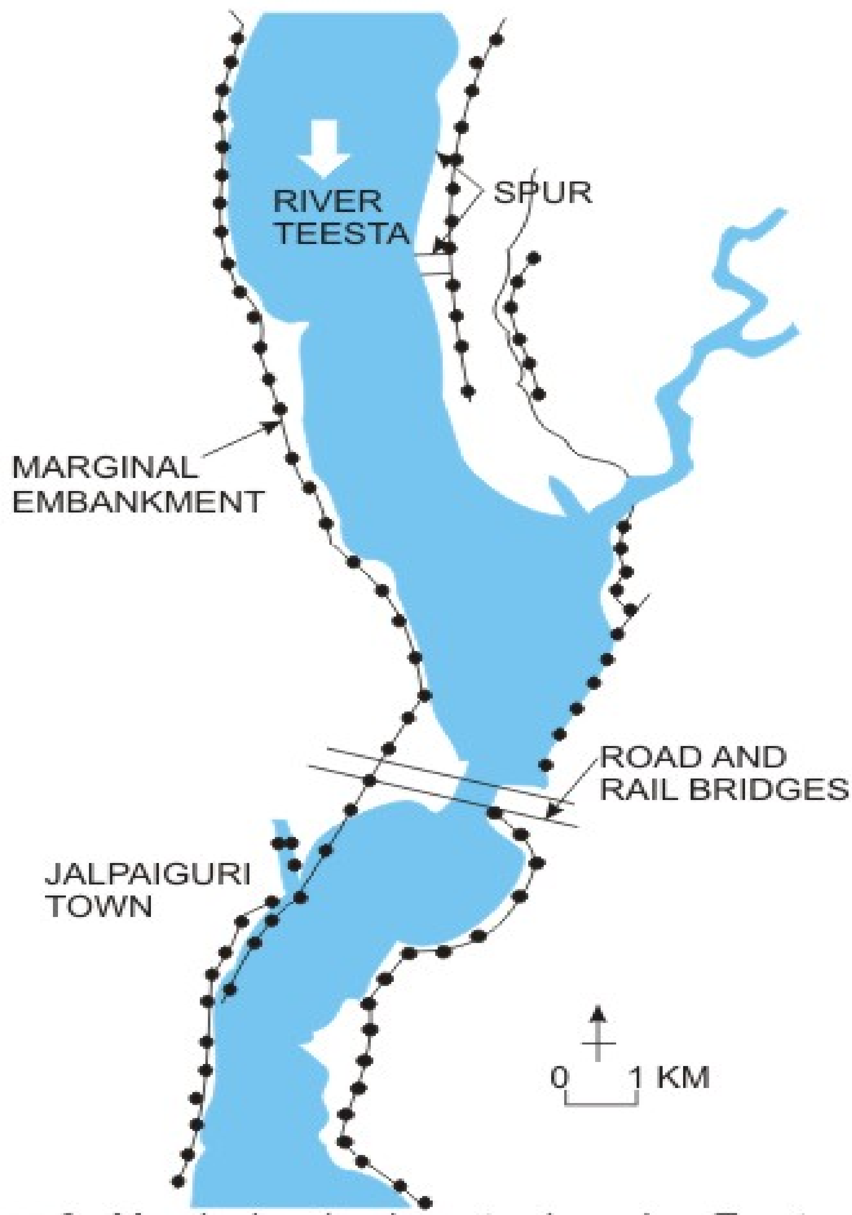


FIGURE 9. Marginal embankments along river Teesta.

However, the ill effects of providing these embankments have become quite apparent now, the most serious of which is the gradual rise of the river bed level over the years due to deposition of sediments. Normally, a river in its original unrestricted shape deposits silt along with flood water not only in its riverbed, but also on its flood plains (Figure 10a). However, as soon a river is confined by marginal embankments, the subsequent deposits of sediment can only take place over the river bed, thus raising the bed elevation (Figure 10b). Instance of such phenomena has been reported for such bis rivers like the Mississippi in the USA. In India, embanking of the Mahanadi river near Cuttack or the Ganga near Patna or the Teesta near Talpaiguri have all caused the river bed level to have gone up alarmingly**.** As a consequence, during floods, the river water level flows alarmingly high (Figure 10c), and the residents of the nearly towns are always under the threat of flooding.

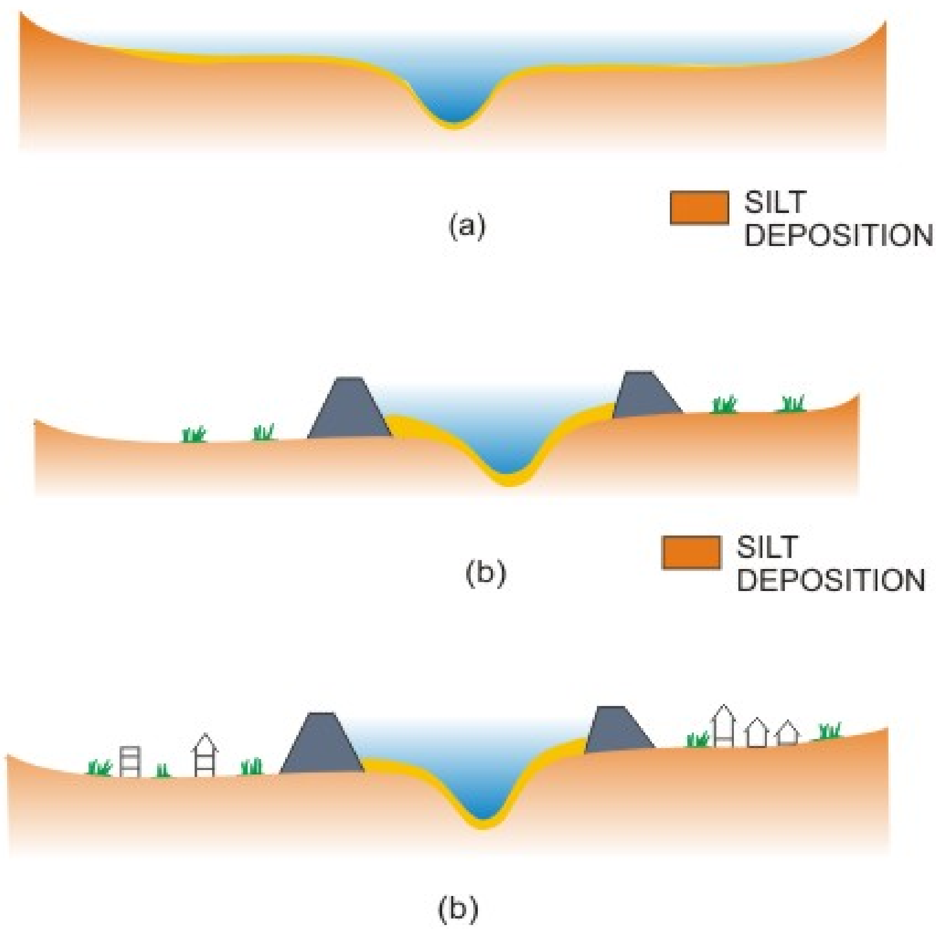


FIGURE 10. (a) Silt deposition spread over river and flood plains in original state
(b) Silt deposition restricted to river bed between embankments
(c) High water level during flood threaten adjacent habitation

In view of the seriousness of this issue, marginal embankments are now being discouraged. The flood plains rightly belong to the river, and forceful occupation of these by artificial means like constructing embankments is harmful to the river as well as human society itself, who had engineered the constructions.

6.1.7 Riverbank protection works

This aspect of river engineering considers methods and techniques for protecting the banks of rivers from collapsing. Hence, certain structural interventions are required to be implemented, which are termed as the riverbank protection works or alternately as bank stabilization structures. Generally these are simple to construct though, the specific hydraulic and geomorphic process associated with these structures are quite complex and challenging. Hence, the type of bank protection work has to be in accordance with the conditions of the specific site – a method suitable for one location of a river may not be so far another location of the same river or at another river. For a proper appreciation of the techniques of bank stabilization, one has to have an awareness about fluvial geomorphology and channel processes, a brief account of which has been discussed in Module 2.

Nevertheless, geomorphic analyses of initial morphological response to system disturbance provides a simple qualitative method for predicting the channel response to an altered condition. Another complicating factor in assessing the cause and effect of system instability is that very rarely is the instability a result of a single factor. In a watershed where numerous alterations (dams, levees, channelization, land use changes, etc.) have occurred, the channel morphology will reflect the integration of all these factors. Unfortunately, it is extremely difficult and often impossible to sort out the precise contributions of each of these components to the system instability. The interaction of these individual factors coupled with the potential for complex response makes assessing the channel stability and recommending channel improvement features, such as bank protection, extremely difficult. There are numerous qualitative and quantitative procedures that are available. Regardless of the procedure used, the designer should always recognize the limitations of the procedure, and the inherent uncertainties with respect to predicting the behavior of complex river systems.

Local instability

Local instability is a term that refers to bank erosion that is not symptomatic of a dis-equilibrium condition in the watershed (i.e., system instability) but results from site specific factors and processes. Perhaps the most common form of local instability is bank erosion along the concave bank in a meander bend which is occurring as part of the natural meander process. Local instability does not imply that bank erosion in a channel system is occurring at only one location or that the consequences of this erosion are minimal. As discussed earlier, erosion can occur along the banks of a river in dynamic equilibrium. In these instances the local erosion problems are amenable to local protection works such as bank stabilization measures. However, local instability can also exist in channels where severe system instability exists. In these situations the local erosion problems will probably be accelerated due to the system instability, and a more comprehensive treatment plan will be necessary. Local instability of a riverbank may be due to either streambank erosion or erosion due to meander bends. These are explained below.

Streambank erosion and failure processes

The terms streambank erosion and streambank failure are often used to describe the removal of bank material. Erosion generally refers to the hydraulic process where individual soil particles at the bank's surface are carried away by the tractive force of the flowing water. The tractive force increases as the water velocity and depth of flow increase. Therefore, the erosive forces are generally greater at higher flows. Streambank failure differs from erosion in that a relatively large section of bank fails and slides into the channel. Streambank failure is often considered to be a geotechnical process. The important processes responsible for bank erosion are described in the following section.

Meander Bend Erosion

Depending upon the academic training of the individual, streambank erosion may be considered as either a hydraulic or a geotechnical process. However, in most instances the bank retreat is the result of the combination of both hydraulic and geotechnical processes. The material may be removed grain by grain if the banks are non-cohesive (sands and gravels), or in aggregates (large clumps) if the banks are composed of more cohesive material (silts and clays). This erosion of the bed and bank material increases the height and angle of the streambank which increases the susceptibility of the banks to mass failure under gravity. Once mass failure occurs, the bank material will come to rest along the bank toe. The failed bank material may be in the form of a completely disaggregated slough deposit or as an almost intact block, depending upon the type of bank material, the degree of root binding, and the type of failure. If the failed material is not removed by subsequent flows, then it may increase the stability of the bank by forming a buttress at the bank toe. This may be thought of as a natural form of toe protection, particularly if vegetation becomes established. However, if this material is removed by the flow, then the stability of the banks will be again reduced and the failure process may be repeated.

As noted above, erosion in meander bends is probably the most common process responsible for local bank retreat and, consequently, is the most frequent reason for initiating a bank stabilization program. A key element in stabilization of an eroding meander bend is an understanding of the location and severity of erosion in the bend, both of which will vary with stage and plan form geometry.

As streamflow moves through a bend, the velocity (and tractive force) along the outer bank increases. In some cases, the tractive force may be twice that in a straight reach just upstream or downstream of the bend. Consequently, erosion in bends is generally much greater than in straighter reaches. The tractive force is also greater in bends with short radius than those with larger ones. The severity of bank erosion also changes with stage. At low flows, the main thread of current tends to follow the concave bank alignment. However, as flow increases, the streamlines tend to cut across the convex bar to be concentrated against the concave bank below the apex of the bend. Because of this process, meanders tend to move downstream, and the zone of maximum erosion is usually in the downstream portion of the bend level due to the flow impingement at the higher flows.

6.1.8 Erosional forces for bank failure

It is not quite easy to identify the processes responsible for the bank erosion. In this section, the primary causes of bank erosion are described briefly.

Parallel flow erosion is the detachment and removal of intact grains or aggregates of grains from the bank face by flow along the bank. Evidence includes: observation of high flow velocities close to the bank; near bank scouring of the bed; under-cutting of the toe/lower bank relative to the bank top; a fresh, ragged appearance to the bank face; absence of surficial bank vegetation.

Impinging flow erosion is detachment and removal of grains or aggregates of grains by flow attacking the bank at a steep angle to the long-stream direction. Impinging flow occurs in braided channels where braid-bars direct the flow strongly against the bank, in tight meander bends where the radius of curvature of the outer bank is less than that of the channel centerline, and at other locations where an in-stream obstruction deflects and disrupts the orderly flow of water. Evidence includes: observation of high flow velocities approaching the bank at an acute angle to the bank; near-bank scouring of the bed; under-cutting of the toe/lower bank relative to the bank top; a fresh, ragged appearance to the bank face; absence of surficial bank vegetation.

Piping is caused by groundwater seeping out of the bank face. Grains are detached and entrained by the seepage flow (also termed sapping) and may be transported away from the bank face by surface runoff generated by the seepage, if there is sufficient volume of flow. Piping is especially likely in high banks backed by the valley side, a terrace, or some other high ground. In these locations the high head of water can cause large seepage pressures to occur. Evidence includes: Pronounced seep lines, especially along sand layers or lenses in the bank; pipe shaped cavities in the bank; notches in the bank associated with seepage zones and layers; run-out deposits of eroded material on the lower bank.

Freeze/thaw is caused by sub-zero temperatures which promote freezing of the bank material. Ice wedging cleaves apart blocks of soil. Needle-ice formation loosens and detaches grains and cleaves apart blocks of soil. Needle-ice formation loosens and detaches grains and crumbs at the bank face. Freeze/thaw activity seriously weakens the bank and increases its erodibility. Evidence includes: periods of below freezing temperatures in the river valley; a loose, crumbling surface layer of soil on the bank;

loosened crumbs accumulated at the foot of the bank after a frost event; jumbled blocks of loosened bank material.

Sheet erosion is the removal of a surface layer of soil by non-channelized surface run-off. It results from surface water draining over the bank edge, especially where the riparian and bank vegetation has been destroyed by encroachment of human activities. Evidence includes: Surface water drainage down the bank; lack of vegetation cover, fresh appearance to the soil surface; eroded debris accumulated on the lower bank/toe area.

Rilling and gullying occurs when there is sufficient uncontrolled surface run-off over the bank to utilize channelized erosion. This is especially likely where flood plain drainage has been concentrated (often unintentionally) by human activity. Typical locations might be near buildings and parking lots, stock access points and along stream-side paths. Evidence includes: a corrugated appearance to the bank surface due to closely spaced rills; larger gullied channels incised into the bank face; headward erosion of small tributary gullies into the flood plain surface; and eroded material accumulated on the lower bank/toe in the form of alluvial cones and fans.

Wind waves cause velocity and shear stresses to increase and generate rapid water level fluctuations at the bank. They cause measurable erosion only on large rivers with long fetches which allow the build up of significant waves. Evidence includes: a large channel width or a long, straight channel with an acute angle between eroding bank and longstream direction; a wave-cut notch just above normal low water plane; a wave-cut platform of run-up beach around normal low-water plane.

Vessel forces can generate bank erosion in a number of ways. The most obvious way is through the generation of surface waves at the bow and stern which run up against the bank in a similar fashion to wind waves. In the case of large vessels and/or high speeds these waves may be very damaging. If the size of the vessel is large compared to the dimensions of the channel hydrodynamic effects produce surges and drawdown in the flow. These rapid changes in water level can loosen and erode material on the banks through generating rapid pore water pressure fluctuations. If the vessels are relatively close to the bank, propeller wash can erode material and re-suspend sediments on the bank below the water surface. Finally, mooring vessels along the bank may involve mechanical damage by the hull. Evidence includes: use of river for navigation; large vessels moving close to the bank; high speeds and observation of significant vessel-induced waves and surges; a wave-cut notch just above the normal low-water plane; a wave-cut platform or “spending” beach around normal low-water plane.

Retreat of river bank often involves geotechnical bank failures as well as direct erosion by the flow. Such failures are often termed as “bank sloughing” or “caving”. Examples of different modes of geotechnical stream bank failure are explained in the next section.

6.1.9 Different modes of bank failure

This section summarizes the different ways by which a river bank collapses by classifying them using geotechnical terminologies.

Soil/rock fall occurs only on a steep bank where grains, grain assemblages or blocks fall into the channel. Such failures are found on steep, eroding banks of low operational cohesion. Soil and rock falls often occur when a stream undercuts the toe of a sand, gravel or deeply weathered rock bank. Evidence includes: very steep banks; debris falling into the channel; failure masses broken into small blocks; no rotation or sliding failures.

Shallow slide is a shallow seated failure along a plane somewhat parallel to the ground surface. Such failure are common on bank of low cohesion. Shallow slides often occur as secondary failures are common on banks of low cohesion. Shallow slides often occur as secondary failures following rotational slips and/or slab failures. Evidence includes: weakly cohesive bank materials; thin slide layers relative to their area; planar failure surface; no rotation or toppling of failure mass.

Rotational slip is the most widely recognised type of mass failure mode. A deep seated failure along a curved surface results in bank-tilting of the failed mass toward the bank. Such failures are common in high, strongly cohesive banks with slope angles below about 60^0. Evidence includes: banks formed in cohesive soils; high, but not especially steep, banks; deep seated, curved failures scars; back-tilting of the top of failure blocks towards intact bank; arcuate shape to intact bank line behind failure mass.

Slab-type block failure is sliding and forward toppling of a deep seated mass into the channel. Often there are deep tension cracks in the bank behind the failure block. Slab failures occur in cohesive banks with steep bank angles greater than about 60^0. Such banks are often the result of toe scour and under-cutting of the bank by parallel and impinging flow erosion.
Evidence includes: cohesive bank materials; steep bank angles; deep seated failure surface with a planar lower slope and nearly vertical upper slope; deep tension cracks behind the bank-line; forward tilting of failure mass into channel; planner shape to intact bank-line behind failure mass.

Cantilever failure is the collapse of an overhanging block into the channel. Such failures occur in composite and layered banks where a strongly cohesive layer is underlain by a less resistance one. Under-mining by flow erosion, piping, wave action and/or pop-out failure leaves an overhang which collapses by a beam, shear or tensile failure. Often the upper layer is held together by plant roots. Evidence includes: composite or layered bank stratigraphy; cohesive layer underlain by less resistant layer; under-mining; overhanging bank blocks; failed blocks on the lower bank and at the bank toe.

Pop-out failure results from saturation and strong seepage in the lower half of a steep, cohesive bank. A slab of material in the lower half of the steep bank face falls out, leaving an alcove-shaped cavity. The over-hanging roof of the alcove subsequently collapses as a cantilever failure. Evidence includes: cohesive bank materials; steep bank face with seepage area low in the bank; alcove shaped cavities in bank face.

Piping failure is the collapse of part of the bank due to high groundwater seepage pressures and rates of flow. Such are an extension of the piping erosion process described previously, to the point that there is complete loss of strength in the seepage layer. Sections of bank disintegrate and are entrained by the seepage flow (sapping). They may be transported away from the bank face by surface run-off generated by the seepage, if there is sufficient volume of flow. Evidence includes: pronounced seep lines, especially along sand layers or lenses in the bank; pipe shaped cavities in the bank; notches in the bank associated with seepage zones; run-out deposits of eroded material on the lower bank or beach. Note that the effects of piping failure can easily be mistaken for those of wave vessel force erosion.

Dry granular flow describes the flow-type failure of a dry, granular bank material. When a noncohesive bank at close to the angle of repose is undercut, increasing the local bank angle above the friction angle. A carpet of grains rolls, slides and bounces down the bank in a layer up to a few grains thick. Evidence includes: noncohesive bank materials; bank angle close to the angle of repose; undercutting; toe accumulation of loose grains in cones and fans.

Wet earth flow failure is the loss of strength of a section of bank due to saturation. Such failures occur when water-logging of the bank increases its weight and decreases its strength to the point that the soil flows as a highly viscous liquid. This may occur following heavy and prolonged precipitation, snow-melt or rapid drawdown in the channel. Evidence includes: sections of bank which have failed at very low angles; areas of formerly flowing soil that have been preserved when the soil dried out; basal accumulation of soil showing delta-like patterns and structures.

6.1.10 Techniques for bank stabilization

There could be two broad ways of stabilizing banks – firstly the direct methods of protecting the slope, and secondly the indirect way by providing structures that extend into the stream channels and redirect the flow so that hydraulic forces at the channel boundary are reduced to a non - erosive level.

Amongst the direct methods available for bank stabilization, the following broad categories are as follows:

- Self-adjusting armour made of stone or other materials
- Rigid armour
- Flexible mattress

The advantages of this type protection are that armoring the surface of the bank is a proven approach which can be precisely designed for a given situation, and which provides immediate and effective protection against erosion. Also, existing or potential problems from erosion by overbank drainage can be effectively addressed integrally with the design of the streambank armor work. Disadvantages for these types of bank protection include preparation of the bank slope is usually required, either for geotechnical stability or to provide a smooth surface for proper placement of the armor. This may result in high cost, environmental damage, and disturbance to adjacent structures. The extent of earthwork associated with an armor revetment will be especially significant if the existing channel alignment is to be modified either by excavation or by placing fill material in the channel. The following sections describe the three types of bank protection works.

As for the indirect methods for bank stabilization, these may be classified into the following categories.

- Dikes - Permeable or Impermeable
- Retards - Permeable or Impermeable
- Other flow deflectors, like Bendway weirs, Iowa vanes, etc.

The advantages of this type of protection are that little or no bank preparation is involved. This reduces costs of local environmental impacts, and simplifies land aquisition. However, the main disadvantage is that these are not very effective where geotechnical bank instability or erosion from overbank drainage are the main causes of bank erosion. Further the construction of these are not very effective where geotechnical bank instability or erosion from overbank drainage are the main causes of bank erosion. Further, the construction of these structures induce significant changes in flow alignment, channel geometry, roughness and other hydraulic factors, which have to be carefully checked to find out any adverse implication of the river's geomorphology. Some types of indirect protection may also pose safety hazard if the stream is used for recreation or navigation. Lastly, since indirect methods require structures to be constructed deep into the stream channel, their construction may become practically difficult, especially during high flows.

Details about these indirect methods of bank protection are not presented in this lesson, but may be obtained from references such as "The WES Stream Investigation and Streambank Stabilization Hand book", published by the U.S. Army Engineer Waterways Experiment Station (WES) in 1997.

6.1.11 Self-adjusting armour of stone or other material

Stone armour can be placed in four general configurations, the most common being a "riprap blanket". Other forms, known as "trenchfill", "longitudinal stone toe," and "windrow" (referred to in some regions as "falling apron"), canbe very useful in certain situations.

A stone armor usually consists of “graded” stone, which is a mixture of a wide range of stone sizes; the largest sizes resist hydraulic forces, and the smaller sizes add interlocking support and prevent loss of bank material through gaps between larger stones. Hand-placed stone in a smaller range of sizes is occasionally used. The various types of stone armours are discussed below:

Riprap Blanket

Riprap (Figure 11) should be blocky in shape rather than elongated, as more nearly cubical stones “nest” together best and are more resistant to movement. The stone should have sharp, clean edges at the intersections of relatively flat faces. Cobbles with rounded edges are less resistant to movement, although the drag force on a rounded stone is less than on sharp-edged cubical stones. As graded cobble interlock is less than that of equal-sized angular stones, the cobble mass is more likely to be eroded by channel flow. If used, the cobbles should be placed on flatter side slopes than angular stone and should be about 25 percent larger in diameter.

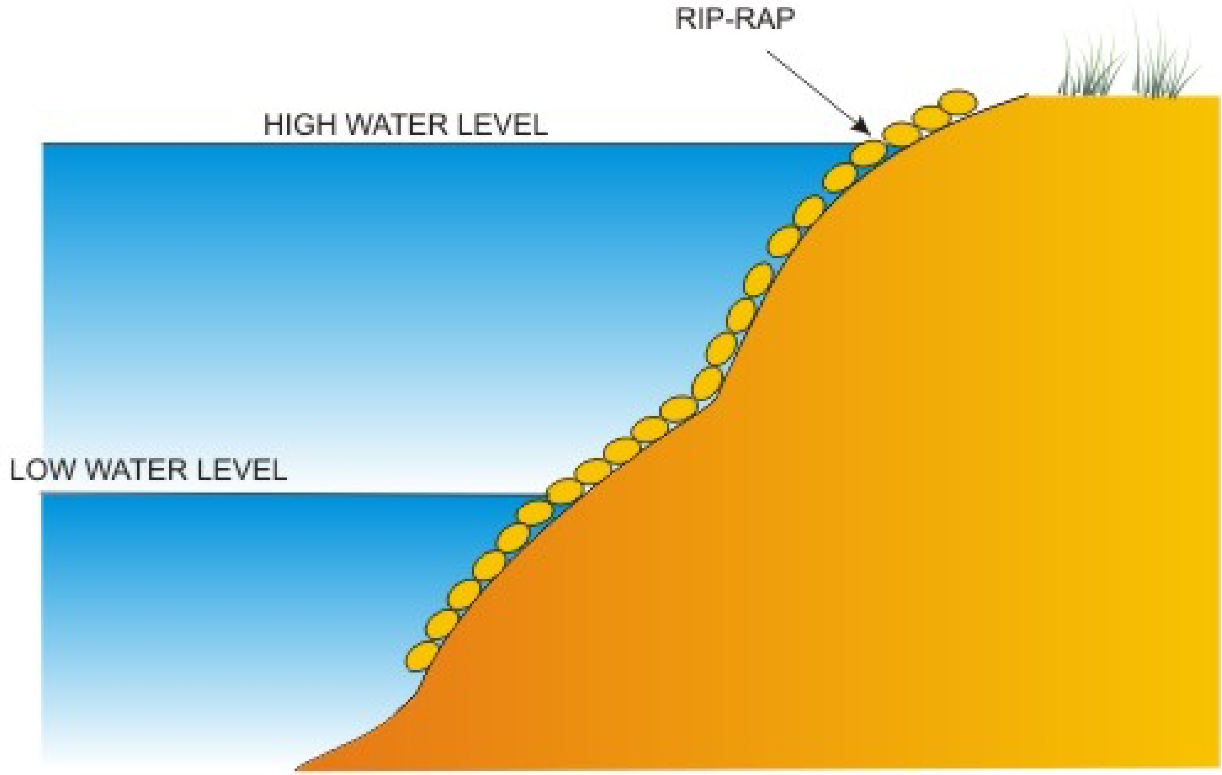


FIGURE 11. Typical river bank section shown protected by rip-rap blanket

The bed material and local scour characteristics determine the design of toe protection, which is essential for riprap revetment stability. The bank material and ground water conditions affect the need for filters between the riprap and underlying material. Construction quality control of both stone production and riprap placement is essential for successful bank protection. Riprap protection for flood-control channels and appurtenant structures should be designed so that any flood that could reasonably be expected to occur during the service life of the channel or structure would not cause damage exceeding nominal maintenance. While the procedures presented herein yield definite stone sizes, results should be used for guidance purposes and revised if appropriate, based on experience with specific project conditions.

Trenchfill

A trenchfill revetment, shown in Figure 12, is simply a standard stone armor revetment with a massive stone toe. It is normally constructed in an excavated trench behind the river bank, in anticipation that the river will complete the work by eroding to the revetment, causing the stone toe to launch down and armor the subsequent bank slope.

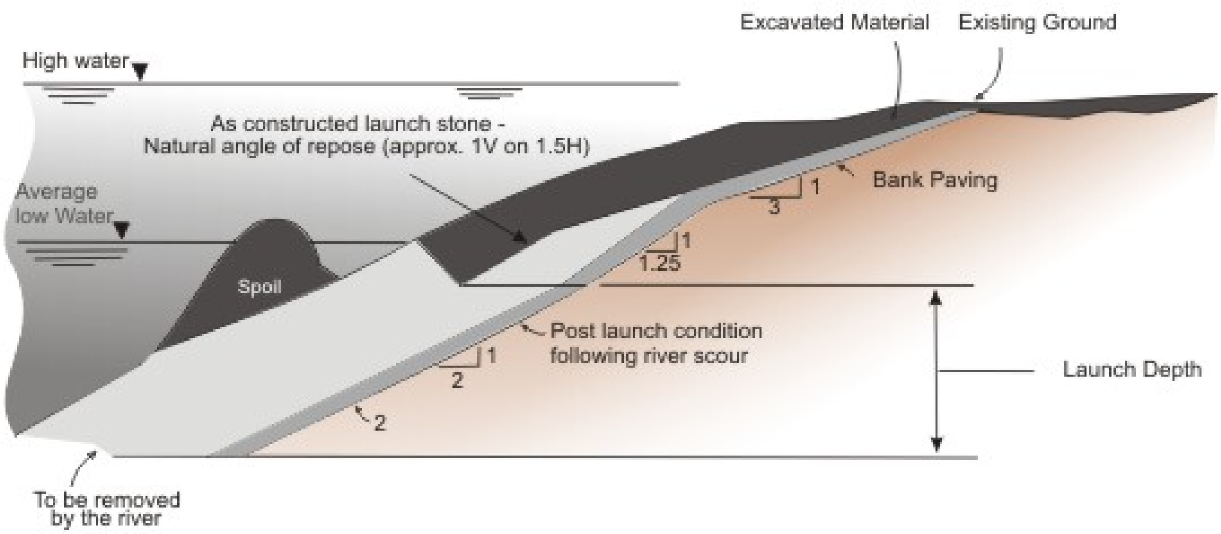


FIGURE 12. Typical river bank section shown protected by trenchfill

Material other than stone, such as broken soil-cement, has been used successfully and may be less costly than stone, but careful design of the soil/cement mixture, and careful monitoring of the material mixing, breaking, and placing operation is required.

Windrow

A windrow revetment (Figure 13) is simply an extreme variation of a trenchfill revetment. A window revetment consists of rock placed on the floodplain surface landward from the existing bankline at a pre-determined location, beyond which additional erosion is to be prevented.

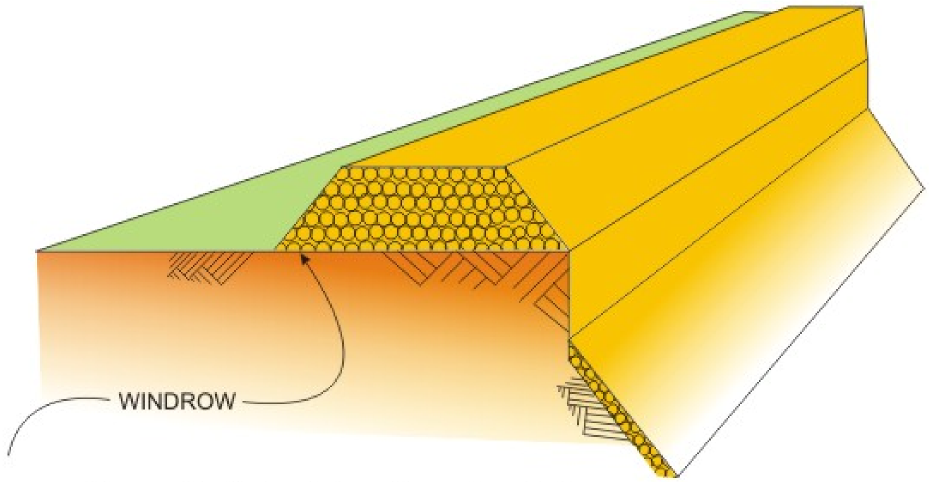


FIGURE 13. Typical view of wondrous laid along a riverbank

Longitudinal stone toe

Longitudinal stone toe (Figure 14) is another form of a window revetment, with the stone placed along the existing streambed rather than on top bank. The longitudinal stone toe is placed with the crown well below top bank, and either against the eroding bankline or a distance riverward of the high bank. Typical crown elevations may vary but are commonly between 1/3 and 2/3 of the height to top bank.

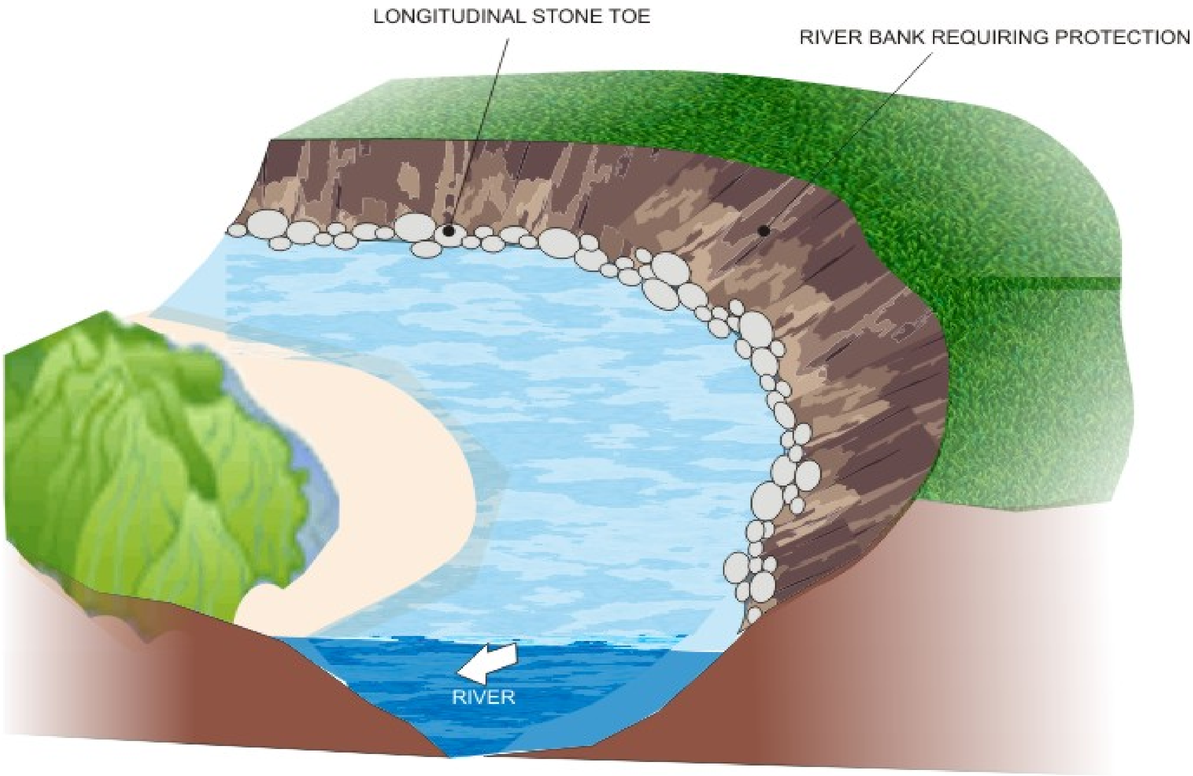


FIGURE 14. Atypical view of logitudinal stone toe laid along a riverbank

The success of longitudinal stone toe protection is based on the premise that as the toe of the bank is stabilized, upper bank failure will continue until a stable slope is attained and the bank is stabilized. This stability is usually assisted by the establishment of vegetation along the bank.

Concrete blocks

These are armour revetment for bank stabilization consisting of loose concrete blocks. Concrete blocks fastened together forms a kind of flexible mattress that is discussed in Section 6.1.13.

A wide variety of block shapes and placement techniques can be used. Some have evolved from engineering analyses, some from observation and empiricism, and some from improvisation using readily available materials. Blocks designed specifically for bank armor are commercially available. Forms for casting concrete blocks locally are often available from distributors, and may be an economical alternative to purchasing and transporting precast blocks.

A fabric or granular underlayment ("filter") is often required for riverbank protection by concrete blocks. Successful performance of the underlayment is more critical than with a riprap armour. In areas of high turbulence or waves, displacement of one block can lead to successive displacement of adjacent blocks. If blocks are cast on-site, delays from inclement weather may be a problem. At sites that are subject to theft or vandalism, blocks of an attractive size and shape may suffer serious attrition.

Sacks

Sacks as an armor material can be considered to be artificial "rocks" of uniform size and shape. The sacks may be made of paper, burlap, or a synthetic material. The fill material may be soil or aggregate of various types, with or without cement. Sacks can be placed on a steeper slope than stone. Materials are often available locally. The hydraulic roughness is low, and they form a walkable surface. The "cobblestone" effect may be more aesthetic than some other materials. As far concrete blocks, a fabric or granular filter is usually required.

Soil-cement blocks

Soil is mixed well with sufficient cement to provide a durable bond between soil particles. The resulting monolith is broken into blocks of various sizes, which are used to armor the bank. Besides the general characteristics of adjustability to bank irregularities and self-healing properties, soil-cement blocks allow the utilization of locally available materials. However, soil-cement blocks have a lower specific weight than riprap, and obtaining acceptable gradation and durability are highly dependent on closely controlled construction operations. Construction operations are adversely affected by wet or cold weather.

6.1.12 Rigid armour

Rigid armour is an erosion-resistant material which has little or no flexibility to conform to bank irregularities occurring after construction. Typically, the armour is placed directly on the bank slope in a fluid or chemically reactive state, then hardens.

The most common rigid armours are:

- Asphalt
- Concrete
- Grouted riprap (or other grouted armour material), and
- Soil-cement

The main advantages for a rigid armour are: The most common rigid armours will withstand high velocities, have low hydraulic roughness, and prevent infiltration of water into the channel bank. They are practically immune to vandalism, damage from debris, corrosion, and many other destructive agents. The most common rigid armours are easily traversed by pedestrians.

However, a rigid armour requires careful design and quality control during construction, and unfavourable weather conditions can cause construction delays. Provision for draining groundwater and preventing the builtup of excess positive pore water pressures, in the form of a filter or subsurface drains, must usually be provided for impermeable armours, which may significantly increase the cost of the project. Most rigid armours are difficult or impossible to construct underwater, although this difficulty can be alleviated for concrete by using one of the commercially available fabric mattresses. Asphalt has been placed underwater in some mattresses. Rigid armour, being inflexible, is susceptible to breaching if the bank material subsides or heaves. Increased wave runup on a smooth rigid armour may be a concern for some projects.

Typical applications of rigid armour in the form of concrete, asphalt, or grouted riprap is often considered for use in situations where high velocities or extreme turbulence make adjustable armour ineffective or very expensive. Typical uses are in conjunction with hydraulic structures or in artificial channels on steep slopes. Rigid armour may be the preferred alternative in flood control or drainage channels where low boundary roughness is mandatory, or in water supply channels where prevention of water loss due to infiltration into the bank is important. It is suitable for bank slopes which must be easily traversed by pedestrians or recreational users, if the slope is not too steep for safety. Rigid armour is sometimes the least costly alternative, typically where adjustable armour is not available locally, especially if a geotechnical analysis of the bank material indicates that elaborate subsurface drainage work is not necessary.

The important types of rigid armour are discussed in the following paragraphs.

Asphalt

Asphalt mixes with a high sand content are sometimes used to retain some permeability to relieve hydrostatic pressure. However, these mixes have been reported to become more brittle and less permeable upon long exposure to the elements, and weathering may result in a slow loss of thickness.

Concrete

On slopes above water, concrete can be placed in the conventional manner with forms, or can be pumped into fabric mattresses which serve as forms for a fine aggregate concrete. Prefabricated slabs would assume some of the characteristics of concrete block armour.

Grouted armour

Grouting of an armour layer with asphalt or concrete enables the armour to withstand higher flow velocities, provides a smooth surface for pedestrian or vehicle access, and reduces the hydraulic roughness of the armour. Grouting is also sometimes used with gabion armours or structures to increase the resistance of the gabions to corrosion and abrasion.

Soil-cement

Soil-cement will withstand relatively high velocities and is usually less expensive than concrete, asphalt, and grouted riprap. It is more durable than chemical stabilization, clay, and certainly ice, but usually somewhat less durable than concrete, asphalt, and grouted riprap, assuming that sound design and construction procedures are followed for all.

6.1.13 Flexible mattress

The basic concept of a flexible mattress is that material or objects which cannot resist erosive forces separately can be fastened together or placed in a flexible container to provide adequate resistance to erosive forces, while partially retaining the desirable characteristics of adjustable armour, especially that of flexibility.

The most common flexible mattress materials are:

- Concrete blocks;
- Fabric; and
- Gabions.

The advantages of this type of riverbank protection work includes its flexibility to adjust to scour or settlement and still remain in contact with the bed and bank is the most obvious shared trait. Most mattress materials which are sold under trade names share another advantage they are available in various configurations, thus can be applied to a variety of situations. Flexible mattresses can be placed underwater with a relatively high degree of confidence. If properly anchored to a geotechnically stable bank, they can be placed on steep slopes. They can be walked upon easily, thus are suitable for slopes used by pedestrians.

However, it must be kept in mind that mattress components are subject to deterioration from the elements and vandalism. The damage is often within acceptable limits through, and, since the various types are affected differently, identification of the hazards enables the designer to select an appropriate mattress for a given application. The construction of some types of mattress is labour intensive, and may require skills not commonly available. However, the labour intensive aspect may not be a disadvantage in all cases, and may be an advantage in some cases.

Typical applications of flexible mattresses are: This compromise between adjustable armour and rigid armour is most attractive when economical materials can be used for the mattress. In fact, the origin of some variations can be traced directly to creative use of local materials where no protective material of local origin was adequate to withstand the erosive forces in a given application, and where the most suitable method was the one which required the least amount of costly imported material, a requirement which is often met by a flexible mattress.

Some of the common forms of flexible mattresses are explained below:

Concrete block mattress

Mattresses provide a higher degree of safety from progressive failure of the armour due to displacement of individual blocks from hydraulic or geotechnical forces or vandalism. Placing of mattresses is more mechanized and less labour intensive than placing individual blocks. Precast concrete blocks can be formed into a flexible mattress in several ways: by fastening them to engineering fabric, by fastening them together with cable or synthetic rope, or by forming them in ingenious shapes which are then interlocked. All of these varieties are commercially available.

Fabric Mattress

Fabric mattresses made of synthetic material and filled with concrete grout, other cohesive mixtures, or sand are available from various manufactures. Tubular-shaped bags are also available; these can be filled and placed either parallel to the streambank as a bulkhead or perpendicular to the streambank as a dike, or can be used to fill scour holes or undermined slopes. A fabric mattress is relatively easy to place, and fill material is often available locally. Some designs have a low hydraulic roughness.

Gabion Mattress

A gabion mattress consists of a mesh container filled with cobbles or quarried stone. Several firms market the containers and furnish technical assistance. Spacialized equipment or accessories used on large jobs for efficiency, or on jobs requiring underwater placement.

A form of gabion which is a hybrid between flexible mattress and adjustable armour is the “sack” or “sausage,” which can be filled faster than mattress or box shapes, making it suitable for use in emergency situations. However, it makes less efficient use of material, and is less common than traditional mattress or boxes.

Vegetation (Fascine) Mattress

Wooden mattress is one of the oldest techniques of bank stabilization, even though it is seldom used now in developed regions. The mats may be made of poles, brush, or lumber. The material can be fastened together by weaving, binding, cabling, clamping, or spiking. The mattresses are sunk by ballasting with stone or other heavy materials. Some types of mat may be so buoyant that the ballast is a significant component of the protection, as well as a large part of the cost.

On navigable rivers during periods when current speed is slow enough that the mats can be safely maneuvered in tow, mats with sufficient buoyancy can be safely maneuvered in tow, mats with sufficient buoyancy can be assembled near the materials supply point or near a source of labour, then towed to the project site.

At least one marine construction firm has adapted modern technology to the construction of wooden mattress, while still retaining traditional skills for use where appropriate. They have also extended new technology to the point of developing synthetic materials for use in mattresses, in order to overcome some of the inherent problems of wood.

The main advantage of this type of bank protection is that its main raw material, that is, wood is usually available locally, and is a renewable resource. If inexpensive labour is available, a wooden mattress may be the least cost alternative. Wood is relatively durable when permanently submerged in freshwater. However, near-site availability of material is usually required for wooden mat to be competitive with other methods. Assembling and placing the mattresses are labour-intensive operations. Design and construction is surprisingly complex, requiring skills which have become rarer as other methods have become have more popular. At some instances, bamboo mattresses have proved very effective as fascine mattress.

Module
6
MANAGEMENT OF WATER RESOURCES

LESSON 2

DROUGHT AND FLOOD MANAGEMENT

Instructional objectives:

On completion of this lesson, the student shall have learnt:

1. The consequences of abnormal rainfall – drought and floods
2. Drought and flood affected regions of India
3. Definition of drought
4. Tackling drought through water management
5. Flood management measures

6.2.0 Introduction

Water is the essential ingredient of life. Unless it is in balanced quantity, any deficit or excess, may cause physiographic imbalance. Similarly, for an entire region, too, deficit or excess of the normal requirement of water may cause imbalance in the regions physical, social, or economic situation. This chapter discusses the two crucial adverse effects of water imbalance: Droughts and Floods.

Since the beginning of the existence of mankind, drought has affected human activity throughout the world. Historical records of drought confirm the fact that it has occurred in almost every part of the world at sometime or other. Examples galore in the history to show that drought is the chief cause of most famines throughout the history of mankind. Many civilisations have perished due to abnormally long persistent deficiency of rainfall. Syrian Desert is one such example.

In India, drought is a frequent natural calamity which finds in all the great epics of the country. One of the earliest droughts in India has been referred in 'Vayu Purana'. In Ramayana also, there is description of drought during the period of king Dasaratha. In Mahabharata, there is mention of serious drought during the reign of Emperor Mandhata of the race of Iksvakus. Written records also give evidence of occurrence of several famines like the one which occurred about 160 years before Mahabharata war during the reign of King Shantanu, the ruler of Hastinapur. During the reign of king Trishanku, father of famous King Harishchandra, a famine is said to have occurred. King Chandragupta Maurya's reign was also witness to a serious famine.

In pre-Independence period, the large catastrophic effect of frequent droughts and famine caught the attention of the then British rulers in the nineteenth century when a series of famine commissions and an Irrigation Commission were setup to go into the various aspects of the problem and to suggest suitable measures to mitigate the distress of the people. Indian Finance Commission in 1880 has mentioned occurrences of severe famine and drought conditions in the then north-west province and Punjab. In 1942-44, the great Bengal famine occurred.

Despite tremendous developments in almost every field, drought continues to torment our society constantly. Even such areas which normally have sufficient precipitation to meet various needs of the area, are confronted with occurrences of drought of shorter or longer duration at sometime or other. According to an estimate, about 108 million hectare, which works out to about one-third of the total geographical area of 329 million hectare of the country, are affected by drought. It has been estimated that number of people living in drought prone districts is around 263 million which is more than 26 percent of the total population of the country (as estimated in the year 2000). Hence, a major part of our country is in the grip of this natural calamity in spite of crores of rupees being spent by the Government on drought combating measures every year. In view of its impact on a wide spectrum of social concerns, a proper understanding and scientific study of drought is extremely essential so that suitable and effective drought proofing measures are formulated in order to minimize or eliminate the adverse impacts of drought on the economy of the country.

6.2.1 Identification of drought and flood affected areas of India

Drought

Irrigation Commission of India in 1972 identified 67 districts located in 8 states and having an area of 49.73 million hectare as drought-prone. Subsequently, the National Commission on Agriculture in 1976 identified a few more drought districts. The drought identification studies carried out by Central Water Commission during 1975-82 considered 99 districts for the study located in 13 states having an area of about 108 million hectares which included those identified by the above two Commissions. For the purpose of study, a smaller unit viz., taluka, was adopted instead of district as a whole and the number of drought affected talukas were identified as 315 out of a total of 725 talukas in the 99 districts. Thus, out of 108 million hectare, only 51.12 million hectare spread over 74 districts can be considered as drought prone areas.

The criteria adopted for the above study was “Drought is a situation occurring in an area when the annual rainfall is less than 75 percent of the normal in 20 percent of the years examined. Any taluka or equivalent unit where 30 percent or more of the cultivated areas are irrigated, is considered to have reached a stage which enables it to sustain a reasonably stable agriculture and to be reasonably protected against drought”.

Attempts are being made to assess as to how much area has been made drought proof (as per the criteria of 30% or more of the cultivated area brought under irrigation with the irrigation facilities) made available to the people. State Governments are to be emphasised to supply this information to the concerned agencies. But it is a fact that lot of area in the country has been converted from drought to normal since the launching of a systematic programme of irrigation development from first Five Year Plan. Had there been no Bhakra Dam, lot of areas of Punjab and Haryana would have been still reeling under the severity of drought. Similarly, Indira Gandhi Nahar Pariyojana has helped in transforming severe drought prone areas of Rajasthan to areas of abundant greenery and general prosperity.

To cover drought prone areas, storage reservoirs have been created to supply drinking water to far – off distant places e.g. water from Sutlej and Ravi rivers are distributed to dry lands of Thar desert and to villages and towns located even 500 to 800 km away from the source, throughout the year. Otherwise the population of these villages would have suffered due to acute water shortages.

Rabi Cultivation is a gift of storage reservoirs which supplements the production of cereals in a big way. But for this, the country would have had to depend on import of cereals from other countries to supplement its demand. No world market can support the food shortages of India even of the order of 50 percent of food requirement. Self help and natural resource generation is the only solution for sustaining the large population.

Floods

Floods have also been a cause of misery for the country since ages. This is because, major habitation clusters like towns and cities been located near rivers since the beginning of civilization. Of course, for most of the cases, they were located much above the high flood level of the river but once a while a heavy rain caused flooding of these places as had been perhaps for Pataliputra (ancient Patna) by River Ganga or Indraprastha (ancient Delhi) by River Yamuna.

According to the India National Commission on Irrigation and Drainage (INCID), 'Flood' is defined as a relatively high flow or stage in a river, marked by higher than the usual, causing inundation of low land or a body of water, rising, swelling and overflowing land that is not normally covered under water. Further, the damage due to flood, or Flood Damage, may be defined as the destruction or impairment, partial or complete of the value of floods and services or of lines resulting from the action of flood water and the silt and debris that they carry. Flood damages arise primarily due to the occupancy of flood plains, which rightfully belong to the river. This is because the flood plains, so to say, are the playgrounds of a river. The flood plains are the playground of the river. Width of these playgrounds may be roughly four to six times the waterway of dominant discharge for meandering rivers. On the one hand, flood plains provide attractive location for various human activities, notably agriculture and transportation such as in Gangetic alluvial plains in U.P., Bihar, W.Bengal, Bangladesh. With increased economic development activities, more and more of the flood plains are getting occupied.

Flood plain occupancy can be costly and in some cases may lead to disaster. Once in a while the river may overflow its banks and exact a heavy toll of property damages, income loss, and perhaps loss of life as well. In densely populated developing countries of South Asia, South East Asia and China, means of sustenance are already limited and the toll exacted by flood disasters in the flood plains is especially heavy.

The annual precipitation in India, which is the source of water causing floods, is estimated at 4,000 BCM including snowfall. Out of this, the seasonal rainfall in monsoon is of the order of 3,000 BCM. The flood problem in the country is mainly due to southwest monsoon during the months from June to October. The average annual rainfall of India is about 1170 mm, of precipitation takes place in about 15 days and less than 100 hours altogether in a year. The rainy days may be only about five in deserts to 150 in the North East.

The average annual flow of the rivers of India has been estimated to be about 1869 BCM. The Brahmaputra and the Ganga rivers contribute the major part of these flows. The rivers carry major portion of their flows during the southwest monsoon period when heavy and widespread rainfall occurs. It is mainly during this period that floods of varying intensities are experienced in one or the other part of the country, bringing in their wake considerable loss of life and property and disruption of communication network.

Flooding is caused by the inadequate capacity within the banks of the rivers to contain the high flows brought down from the upper catchment due to heavy rainfall. Areas having poor drainage characteristics get flooded by accumulation of water from heavy rainfall. Flooding is accentuated by erosion and silting of the river beds resulting in reduction of carrying capacity of river channel, earthquakes and landslides leading to changes in river courses, obstructions to flow, synchronization of floods in the main and tributary rivers and retardation due to tidal effects. Some parts of the country mainly coastal areas of Andhra Pradesh, Orissa, Tamil Nadu and West Bengal experiences cyclones which often are accompanied by heavy rainfall leading to flooding. There had been a recent case of flood due to a super cyclone combined with heavy rainfall during October 1999 in the coastal belt of Orissa in India.

6.2.2 Characteristics of flooding in specific regions of India

The rivers in India can be broadly divided into the following four regions for a study of flood problems:

1. Brahmaputra River Region
2. Ganga River Region
3. Northwest River Region
4. Central India and Deccan Region

The flood situation in each of these regions are described in the following paragraphs.

Brahmaputra River Region

This region consists of rivers Brahmaputra and Barak and their tributaries and covers the States of Assam, Arunachal Pradesh, Meghalaya, Mizoram, Northern parts of West Bengal, Manipur, Tripura and Nagaland. Catchments of these rivers receive very heavy rainfall ranging from 110 cm to 635 cm a year which occurs mostly during the months of May/June to September. As a result flood in this region are severe and quite frequent. Further, the rocks of the hills, where these rivers originate, are friable and susceptible to erosion and thereby cause exceptionally high silt charge in the rivers. In addition, the region is subject to severe and frequent earthquakes, which cause numerous landslides in the hills and upset the regime of the rivers. The predominant problems in this region are the flooding caused by the spilling of rivers over the banks, drainage congestion and tendency of some of the rivers to change their courses. In recent years, erosion along the banks of Brahmaputra has assumed serious proportions.

Considering the individual States in the region, main problems in Assam are inundation caused by spilling of the Brahmaputra and the Barak and their tributaries and also erosion along the Brahmaputra river. In northern parts of West Bengal, the Teesta, Torsa, Jaldhaka and Mahananda are in floods every year and inundate large areas. These rivers also carry considerable amount of silt and have a tendency to change their courses. Rivers in Manipur spill over their banks frequently. Lakes in this territory get filled up during monsoons and spread over large marginal areas. In Tripura, there are problems of spilling and erosion by rivers.

Ganga River Region

River Ganga and its numerous tributaries, of which some of the important ones are Yamuna, Sone, Ghagra, Gandak, Kosi and Mahananda, constitute this river region. It covers the States of Uttar Pradesh, Bihar, South and central parts of West Bengal, parts of Haryana, Himachal Pradesh, Rajasthan, Madhya Pradesh and Delhi. Normal annual rainfall of this region varies from about 60 cm to 190 cm of which more than 80% occurs during the southwest monsoons. Rainfall increases from west to east and from south to north.

Flood problem is mostly confined to the areas on the northern bank of Ganga River. Damage is caused by the northern tributaries of Ganga by spilling over their banks and changing their courses, inundation and erosion problems are confined to a relatively few places. In general, the flood problem increases from the west to the east and from south to north. In the Northwestern parts of the region, there is the problem of drainage congestion. Drainage problem also exists in the southern parts of West Bengal.

Flooding and erosion problems are serious in the States of Uttar Pradesh, Bihar and West Bengal. In Rajasthan and Madhya Pradesh, the problem is not so serious but in some of the recent years, these States have also experienced some incidents of heavy floods.

In Bihar, floods are largely confined to the rivers of North Bihar and are, more or less, an annual feature. Rivers such as Burhi Gandak, Bagmati, Kamala Balan, other smaller rivers of the Adhwra Group, Kosi in the lower reaches and Mahananda at the eastern end spill over their Ganga in some years causing considerable inundation of the marginal areas in Bihar. During last few years, erosion has also been taking place along Ganga and is now prominent on the right bank immediately downstream of the Mokamah bridge and in the vicinity of Mansi Railway Station on the left bank.

In Uttar Pradesh, flooding is frequent in the eastern districts, mainly due to spilling of Tapti, Sharada, Ghagra and Gandak. Problems of drainage congestion exists in the western and northwestern areas of Uttar Pradesh, particularly in Agra, Mathura and Meerut districts. Erosion is experienced in some places on the left bank of Ganga, on the right bank of Gharga and on the right bank of Gandak.

In Haryana, flooding takes places in the marginal areas along the Yamuna and the problem of poor drainage exists in some of the southwestern districts.

In south and central West Bengal, Mahananda, Bhagirathi, Ajoy, Damodar etc. cause flooding due to inadequate capacity of river channels and tidal effect. There is also the problem of erosion of the banks of rivers and on the left and right banks of Ganga upstream and downstream respectively of the Farakka barrage.

In Delhi, a small area along the banks of the Yamuna is subjected to flooding by river spills. In addition local drainage congestion is experienced in some of the developing colonies during heavy rains.

Northwest River Region

Main rivers in this region are the Sutlej, Beas, Ravi, Chenab and Jhelum, tributaries of Indus, all flowing from the Himalayas. These carry substantial discharges during monsoons and also large volumes of sediment. They changes their courses frequently and leave behind vast tracts of sandy waste. The region covers the States of Jammu and Kashmir, Punjab, parts of Himachal Pradesh, Haryana and Rajasthan.

Compared to Ganga and Brahmaputra River regions, flood problem is relatively less in this region. Major problem is that of inadequate surface which causes inundation and water logging over vast areas.

At present, the problems in the States of Haryana and Punjab are mostly of drainage congestion and water logging. Floods in parts of Rajasthan were rare in the past. Ghaggar River used to disappear in the sand dunes of Rajasthan after flowing through Punjab and Haryana. In recent years, it has become active in Rajasthan territory, occasionally submerging large areas.

Jhelum, floods occur periodically in Kashmir causing rise in the level of the Wullar Lake there by submerging marginal areas of the lake.

Central India and Deccan Region

Important rivers in this region are Narmada, Tapi, Mahanadi, Godavari, Krishna and Cauvery. These rivers have mostly well defined stable courses. They have adequate capacity within the natural banks to carry the flood discharge except in the delta area. The lower reaches of the important rivers on the east coast have been embanked, thus largely eliminating the flood problem.

This region covers all the southern States, namely Andhra Pradesh, Karnataka, Tamil Nadu and Kerala and the States of Orissa, Maharashtra, Gujarat and parts of Madhya Pradesh. The region does not have serious problem except for some of the rivers of Orissa state, namely Brahmani, Baitarni, and Subernarekha. The delta areas of Mahanadi, Godavari and Krishna rivers on the east coast periodically face flood and drainage problems in the wake of cyclonic storms.

Tapi and Narmada are occasionally in high floods affecting areas in the lower reaches in Gujarat.

The flood problem in Andhra Pradesh is confined to spilling by the smaller rivers and submergence of marginal areas along the Kolleru Lake. In addition, there is a drainage problem in the deltaic tracts of the coastal districts.

In Orissa, damage due to floods is caused by Mahanadi, Brahmani and Baitarni which have a common delta where the floodwaters intermingle and when in spate simultaneously cause considerable havoc. The problem is accentuate when the flood synchronizes with high tides. Silt deposited constantly by these rivers in the delta area raises the flood level and the rivers often over-flow their banks or break through new channels causing heavy damage. Lower reaches of Subernarekha are affected by floods and drainage congestion. Small rivers of Kerala when in high floods cause considerable damage occasionally.

Details of annual damage due to floods in any one of the years under consideration is taken as the area liable to flood in that State. Considering all such figures for all the States for the period from 1953 to 1978, Rashtriya Barh Ayog (National Commission on Floods) has assessed the total area liable to flood in the country as 40 m.ha. out of which 32m.ha area could be provided with reasonable degree of protection. The severity of the problem can be seen from the fact that this area constitutes one eighth of total geographical area of the country.

6.2.3 The concept of drought

Drought has many definitions, but mostly it originates from a deficiency of precipitation over an extended period of time, usually a season or more. This deficiency results in a water shortage for some activity, group, or environmental sector. Drought should be considered relative to some long term average condition of balance between precipitation and evapotranspiration (i.e., evaporation+transpiration) in a particular area, a condition often perceived as “normal”. It is also related to the timing (i.e., principal season of occurrence, delays in the start of the rainy season, occurrence of rains in relation to principal crop growth stages) and the effectiveness (i.e., rainfall intensity, number of rainfall events) of the rains. Other climatic factors such as high temperature, high wind, and low relative humidity are often associated with it in many regions of the world and can significantly aggravate its severity. There are four disciplinary definitions of drought, which are as follows:

Meteorological Drought

Meteorological drought is defined usually on the basis of the degree of dryness (in comparison to some “normal” or average amount) and the duration of the dry period. Definitions of meteorological drought must be considered as region specific since the atmospheric conditions that result in deficiencies of precipitation are highly variable from region to region. For example, some definitions of meteorological drought identify periods of drought on the basis of the number of days with precipitation less than some specified threshold.

Agricultural Drought

Agricultural drought links various characteristics of meteorological (or hydrological) drought to agricultural impacts. Focusing on precipitation shortages, differences between actual and potential evapotranspiration. Soil water deficits, reduced ground water or reservoir levels, and so forth. Plant water demand depends on prevailing weather conditions, biological characteristics of the specific plant, its stage of growth, and the physical and biological properties of the soil. A good definition of agricultural drought should be able to account for the variable susceptibility of crops during different stages of crop development, from emergence to maturity. Deficient topsoil moisture at planting may hinder germination, leading to low plant populations per hectare and a reduction of final yield. However, if topsoil moisture is sufficient for early growth requirements, deficiencies in subsoil moisture at this early stage may not affect final yield if subsoil moisture is replenished as the growing season progresses or if rainfall meets plant water needs.

Hydrological Drought

Hydrological drought is associated with the effects of periods of precipitation (including snowfall) shortfalls on surface or subsurface water supply (i.e., streamflow, reservoir and lake levels, ground water). The frequency and severity of hydrological drought is often defined on a watershed or river basin scale. Although all droughts originate with a deficiency of precipitation, hydrologists are more concerned with how this deficiency plays out through the hydrologic system. Hydrological droughts are usually out of phase with or lag the occurrence of meteorological and agricultural droughts. It takes longer for precipitation deficiencies to show up in components of the hydrological system such as soil moisture, streamflow, and ground water and reservoir levels. As a result, these impacts are out of phase with impacts in other economic sectors.

Socioeconomic Drought

Socioeconomic definitions of drought associate the supply and demand of some economic good with elements of meteorological, hydrological, and agricultural drought. It differs from the aforementioned types of drought because its occurrence depends on the time and space processes of supply a demand to identify or classify droughts. The supply of many economic goods, such as water, forage, food grains, fish, and hydroelectric power, depends on weather. Because of the natural variability of climate, water supply is ample in some years but unable to meet human and environmental needs in other years. Socioeconomic drought occurs when the demand for an economic goods exceeds supply as a result of a weather-related shortfall in water supply.

The sequence of impacts associated with meteorological, agricultural, and hydrological drought further emphasizes their differences. When drought begins, the agricultural sector is usually the first to be affected because of its heavy dependence on stored soil water. Soil water can be rapidly depleted during extended dry periods. If precipitation deficiencies continue, then people dependent on other sources of water will begin to feel the effects of the shortage.

6.2.4 Indices for drought monitoring

Drought indices are numbers on a certain scale, which defines drought quantitatively. The most commonly used indices world wide, which are based on a number of data on rainfall, snowpack, streamflow, and other water supply indicators, are discussed in the following paragraphs.

Percent of normal

Percent of normal precipitation is one of the simplest measurements of rainfall for a location. Analyses using the percent of normal are very effective when used for a single region or a single season. Percent of normal is also easily misunderstood and gives different indications of conditions, depending on the location and season. It is calculated by dividing actual precipitation by normal precipitation-typically considered to be a 30 scales range from a single month to a group of months representing a particular season, to an annual or water year. Normal precipitation for a specific location is considered to be 100 percent.

One of the disadvantages of using the percent of normal precipitation is that the mean, or average precipitation is often not the same as the median precipitation, which is the value exceeded by 50 percent of the precipitation occurrences in a long-term climate record. The reason for this is that precipitation on monthly or seasonal scales does not have a normal distribution. Use of the percent of normal comparison implies a normal distribution where the mean and median are considered to be the same.

Standardized Precipitation Index (SPI)

The understanding that a deficit of precipitation has different impacts on groundwater, reservoir storage, soil moisture, snowpack, and streamflow led scientists to develop the Standardized Precipitation Index (SPI). The SPI was designed to quantify the precipitation deficit for multiple time scales. These time scales reflect the impact of drought on the availability of the different water resources. Soil moisture conditions respond to precipitation anomalies on a relatively short scale. Groundwater, streamflow, and reservoir storage reflect the longer-term precipitation anomalies.

The SPI calculation for any location is based on the long-term precipitation record for a desired period. This long-term record is fitted to a probability distribution, which is then transformed into a normal distribution so that the mean SPI for the location and desired period is zero. Positive SPI values indicate greater than median precipitation, and negative values indicate less than median precipitation. Because the SPI is normalized, wetter and drier climates can be represented in the same way, and wet periods can also be monitored using the SPI.

Palmer Drought Severity Index (PDSI)

The PDSI is a meteorological drought index, and it responds to weather conditions that have been abnormally dry to abnormally wet. When conditions change from dry to normal or wet, for example, the drought measured by the PDSI ends without taking into account streamflow, lake and reservoir levels, and other longer-term hydrologic impacts. The PDSI is calculated based on precipitation and temperature data, as well as the local Available Water Content (AWC) of the soil. From the inputs, all the basic terms of the water balance equation can be determined, including evapotranspiration, soil recharge, runoff, and moisture loss from the surface layer. Human impacts on the water balance, such as irrigation, are not considered. The classification of a region according to this index is as follows:

4.0 or more	Extremely wet
3.0 to 3.99	Very wet
2.0 to 2.99	Moderately wet
1.0 to 1.99	Slightly wet
0.5 to 0.99	Incipient wet spell
0.49 to -0.49	Near normal

Crop Moisture Index (CMI)

The Crop Moisture Index (CMI) uses a meteorological approach to monitor week-to-week crop conditions. It was developed from procedures within the calculation of the PDSI. Whereas the PDSI monitors long-term meteorological wet and dry spells, the CMI was designed to evaluate short-term moisture conditions across major crop-producing regions. It is based on the mean temperature and total precipitation for each week within a climate division, as well as the CMI value from the previous week.

Because it is designed to monitor short-term moisture conditions affecting a developing crop, the CMI is not a good long-term drought monitoring tool. The CMI's rapid response to changing short-term conditions may provide misleading information about long-term conditions. For example, a beneficial rainfall during a drought may allow the CMI value to indicate adequate moisture conditions. For example, a beneficial rainfall during a drought may allow the CMI value to indicate adequate moisture conditions, while the long-term drought at that location persists. Another characteristic of the CMI that limits its use as a long-term drought monitoring tool is that the CMI typically begins and ends each growing season near zero. This limitation prevents the CMI from being used to monitor moisture conditions outside the general growing season, especially in droughts that extend over several years. The CMI also may not be applicable during seed germination at the beginning of a specific crop's growing season.

Surface Water Supply Index (SWSI)

The objective of the SWSI was to incorporate both hydrological and climatological features into a single index value resembling the Palmer Index for each major river basin in the state of Colorado in U.S.A. These values would be standardized to allow comparisons between basins. Four inputs are required within the SWSI: snowpack, streamflow, precipitation, and reservoir storage in the winter. During the summer months, streamflow replaces snowpack as a component within the SWSI equation.

The procedure to determine the SWSI for a particular basin is as follows: monthly data are collected and summed for all the precipitation stations, reservoirs, and snowpack/streamflow measuring stations over the basin. Each summed component is normalized using a frequency analysis gathered from a long-term data set. The probability of non-exceedence (the probability that subsequent sums of that component will not be greater than the current sum) is determined for each component based on the frequency analysis. This allows comparisons of the probabilities to be made between the components. Each component has a weight assigned to it depending on its typical contribution to the surface water within that basin, and these weighted components are summed to determine a SWSI value representing the entire basin. Like the Plamer Index, the SWSI is centered on zero and has a range between -4.2 and +4.2.

Reclamation Drought Index (RDI)

Like the SWSI, the RDI is calculated at a river basin level, and it incorporates the supply components of precipitation, snowpack, streamflow, and reservoir levels. The RDI differs from the SWSI in that it builds a temperature-based demand component and a duration into the index. The RDI is adaptable to each particular region and its main strength is its ability to account for both climate and water supply factors.

Deciles

Arranging monthly precipitation data into deciles is another drought-monitoring technique. It was developed to avoid some of the weakness within the “percent of normal” approach. The technique they developed divided the distribution of occurrences over a long-term precipitation record into tenths of the distribution. They called each of these categories a decile. The first decile is the rainfall amount not exceeded by the lowest 10% of the precipitation occurrences. The second decile is the precipitation amount not exceeded by the lowest 20% of occurrences. These deciles continue until the rainfall amount identified by the tenth decile is the largest precipitation amount within the long-term record. By definition, the fifth decile is the median, and it is the precipitation amount not exceeded by 50% of the occurrences over the period of record. The classification of a region according to deciles is as follows:

Deciles 1-2: lowest 20%	Much below normal
Deciles 3-4: next lowest 20%	Below normal
Deciles 5-6: middle 20%	Near normal
Deciles 7-8: next highest	Above normal
Deciles 9-10: highest	Much above normal

6.2.5 Tackling drought through water management

Mean annual rainfall over the country is around 119 centimeters, out of which about 80 percent rainfall occurs only during the 4 monsoon months of the year. However, this rainfall varies widely from region to region, season to season and year to year. While some of the regions of the country receive as much as 10,000 millimetres (mm) or more like the hills of Assam, a major part of Rajasthan gets only 100mm or even less. Low rainfall leads to arid conditions which persist almost throughout the year. Nearly 9 percent area of the country is arid and 40% is semi-arid (annual rainfall between 500 and 1000 mm). Because of large variability of rainfall both in space and time, semi-arid regions are subjected to the problems of drought. The problems of arid areas wherever one good crop is not possible in normal years is quite different from those of semi-arid areas where one good crop is normally expected but it is frequently lost due to scanty rainfall or due to variability of rainfall. Even normally high rainfall areas face failure of rains and consequent upsetting of human water requirements. Water conservation and water management measures are need of the day to achieve a strong and stable economic base, especially in the arid and drought prone areas of the country. There are no general solutions possible. They will have to be area specific, because of the hydrological peculiarities. It has also to be remembered that development of drought prone areas cannot be modelled on the lines of the development of other favourably placed areas. The pattern of development of the drought-prone areas will have to be quite different from that of the others.

Some of the methods that may be suggested as technical strategies to mitigate the adversities of drought are mentioned in the following paragraphs.

Creation of surface storage

Conventional approach to water conservation has been to go in for water development projects – creating reservoirs by building dams, big and small, and diversion canals – to supply water wherever and in whatever amounts desired. The total storage capacity of all the reservoirs (major, medium and minor) in the country has been assessed as 400 cubic kilometer. Central Water Commission is regularly monitoring the storage availability of 70 selected major and medium reservoirs with a storage capacity of about 131 cubic kilometer, out of which reservoirs with storage capacity of about 50 cubic kilometer are located in the drought prone areas. Comparing this with the overall utilisable potential of 690 cubic kilometer of surface water apparently shows that to overcome our water supply problems, we have to go in for creation of more storages. However, this will not solve the main problems raised due to large spatial and temporal variations in rainfall. One is that the overall figure of availability of water resources presents a misleading picture. In some regions there is scope of storage but so much of water is not needed. Many river basins like Cauvery, Sabarmati have already exhausted the available water resources. In many other basins, water is fast becoming scarce. The second problem is that building dams and canals has become an extremely costly proposition. This is partly due to the increase in the basic cost of construction and partly due to the necessity to tackle more complex projects involving difficult foundations etc.

Planning for less dependable yield

In India normally the drinking water supplies are planned for almost 100 percent dependability, hydro-power systems for 90 percent dependability and the irrigation systems for 75 percent dependability. However, for the drought areas, planning of average flows or 50 percent dependability has been recommended by many Commission and Committees to increase the availability of water mainly for the agricultural purposes. Minor irrigation tanks (i.e. which have culturable command area of 2,000 hectare or less) are already being planned for 50 percent dependability.

Prevention of evaporation losses from reservoirs

It is seen that shallow tanks having large surface areas located in the drought affected areas lose nearly half the water storage by evaporation in summer months. To save water in a critically water short region, an application of a layer of chemicals like cetyl, stearyl and fatty alcohol emulsions can effectively retard evaporation and savings in the field can be around 40 percent of the normal evaporation losses.

Adjustment in sanctioned water to a reservoir or its releases

The trend of reservoir filling or the ground water position for a water year gets fairly known by the middle of August. Re-adjustment of sanctions and releases have to be carefully carried out at this time keeping a close watch on the behaviour of the monsoon. The modern management techniques using probability analysis may help in assessing the situations of 'supply-variability' in the drought areas.

Reduction in conveyance losses

Reduction in conveyance losses in the conveyance system is an important facet of the water conservation techniques because losses due to seepage are found to vary widely in an irrigation system ranging from 35 percent to 45 percent of the diverted water. Lining of the canal system could be an appropriate step to conserve this precise resource in such a situation.

Considering the high degree of losses in dry summer months, running a canal system in the drought areas during the hot dry months will not be an economical proposition. As an alternative, it will be better to transport as much water during the wet monsoon months or later during the winter period thereafter and to store water in small tanks or ponds near the point of consumption for later use during the summer months. Similarly, practice usually adopted for releasing water through the river channel itself for transporting over long distances during the dry months should be discouraged. However, in cases where releases have to be made during summer months through the river channel instead of resorting to releasing continuous low flows over long periods it would be better to rush the requisite quantity in a small period and then hold it up in small storages near the points of consumption. Such rush systems are being successfully practiced in Maharashtra.

Equitable distribution

Many of the existing canal systems are not able to supply an adequate and equitable quantum of water to all the farmers in the command areas. A rotational system of supply of water if strictly implemented will not only meet the ends of equity but will also economise use of water. Lack of adequate control arrangements in the canal systems also adds to the problem of equitable distribution of water. Another important aspect of water management is the prevention of loss of water to drains during transit from outlet to field. This can be eliminated by farmers active participation in water distribution and maintenance of distribution network in good shape.

Maintenance of irrigation systems

Over the years, maintenance of irrigation systems has deteriorated mainly due to the fact that water rates charged are not sufficient for carrying out the maintenance for keeping the system fit and efficient. In some States, leave alone the operation and maintenance, the revenue collected from irrigation rates does not even cover the expenses incurred on collection of revenue. Whole range of activities covering operation, routine maintenance, special and major repairs, replacements etc. are now covered under " maintenance" and funds are allocated to it from non-plan. Due to shortages of funds and restrictions on non-plan activities, most of the allotted money under this is being spent on staff salaries. Unless adequate allocations are earmarked for maintenance of irrigation systems, gradual deterioration of the existing irrigation systems cannot be controlled.

Better irrigation practice

On farm irrigation practices prevailing in the country also result in wastage of water leading to poor irrigation efficiency. Most farmers still irrigate as their predecessors did hundreds of years ago by flooding or channeling water through parallel furrows. Absence of field channels for adopting to field irrigation adds to the problem. Simple measures like leveling of the fields so that water gets more evenly distributed can greatly improve the performance. Wastage due to absence of field channels and lack of field leveling are now being eliminated through the Command Area Development (CAD) programmes.

Irrigation scheduling

Better irrigation scheduling practices can also improve the irrigation efficiency. For example, it is now well established that water is required more at critical stages of crop growth and water stress during other period has negligible impact on yields. Addition waterings do not add proportionately more to the yield. Greater effort should be made to train farmers in the use of irrigation scheduling methods appropriate to their mode of production. Agricultural extension programmes could help spread the benefits of these water management techniques.

Cropping pattern

Better water management involves all stages i.e. from pre-project formulation to operation and maintenance. In the project formulation stage, a suitable cropping pattern in conformity with soil and climatic conditions taking into account the farmers preferences should be evolved. While designing the canal capacities, peak demand of water in critical periods by the high yielding varieties of crops should be kept in view.

Conjunctive use of surface and ground water

The concept of conjunctive use of surface and ground water resources is very essential especially in drought areas in order to increase the production per unit of water. The manner of using ground water and surface water varies considerably from region to region. Where ground water quality is not good, canal water can be mixed in suitable proportion. Conjunctive use makes possible same flexible of cropping pattern and multi-cropping in the canal command. For the proper water management, it is necessary to treat command areas as one composite unit and two resources should be judiciously managed to achieve optimization of benefits. Costs of exploiting the two sources vary considerably and efforts are necessary to lay uniform charges for providing irrigation to serve the area in an optimal manner and to achieve maximum food production. The concept of conjunctive use has been successfully implemented in various States. Conjunctive use of surface and ground water supplies needs careful planning on more scientific lines to achieve full benefits particularly in all drought management programmes. Suitable legislation is called for to regulate over-exploitation of ground water, which at present is developed and used on individual ownership basis.

Watershed development

Planning of watershed development involves an integrated approach upon physiographic and hydrologic characteristics which include construction of soil conservation works on crop lands; Construction of structures, like check dams, Nalla bunding, contour bunds, Gully plugging, percolation tanks, development of rainwater harvesting and construction of wells etc.

Ministry of Agriculture’s proposal of National Watershed Development Projects focuses on aspects from the angle of agriculture environment, forests and rural development and heavy investment is envisaged for macro level development. Pilot projects for Watershed Development in Rainfed areas in Andhra Pradesh, Karnataka, Madhya Pradesh and Maharashtra have already been implemented with World Bank assistance. This is a long term development, whereas watershed development at micro level will lead to quick results in increasing the water availability and leading to sustainable development. Presently, there are several externally aided projects sponsored by the Central Government and funded by the World Bank and other Organisations which are going on in various parts of the country. Some State Governments namely Andhra Pradesh, Karnataka, Madhya Pradesh, Maharashtra, Orissa and Rajasthan have also started watershed development programmes on their own, with some success as at Jhabua in Madhya Pradesh. In this connection, it is worth mentioning that watershed development in drought prone areas needs involvement of both Government and Non-Government agencies using Non-Government Organisations (NGOs) as an interface between the Government and the local village communities for revival, restoration and development of the watersheds. Examples of Ralegaon-Shindi, Adgaon, Rendhar, Sukhomajri, Tejpura, Nalgaon, Daltonganj, Sidhi, Jawaja and Alwar show that Voluntary organizations and Non-Government Organisations can play a major role in the watershed development and management.

Creation of large storages

While planning various projects particularly in the regions depending on rainfall, it is preferable to go in for large storages rather than a large number of small storages on the tributaries, since small tanks are particularly vulnerable to drought. This is also essential in view of the fact that about 80 percent of the river flow occurs only during the four monsoon months and this flow requires to be stored for irrigation and power generation. The present storage capacity of all the reservoirs including major, medium and minor schemes is 400 cubic kilometer as against the potential of 690 cubic kilometre which means the water scarcity problem may be solved to a great extent by creation of more storages.

Integrating small reservoirs with major reservoirs

Of late, there are persistent demands to abandon the schemes of large storages as it is feared that they cause environmental disaster leading to non-sustainable development of water resources. Instead, number of small reservoirs are being advocated to replace a single large reservoir. However, in many cases, a group of small schemes may not provide the same benefits as a large project can. It is, therefore, very important that minor schemes are integrated with the canal systems of major reservoirs.

Transfer of water from water excess basins to water-deficit basins

A permanent long term solution to drought problem may be found in the basic principles of transfer of water from surplus river basins to areas of deficit. For this purpose, it is essential to take an overall national view for the optimum utilisation of available water resources. With this aim in view, Ministry of Water Resources and Central Water Commission have formulated a national perspective plan for water resources development which consists of two components: Himalayan Rivers Development and Peninsular Rivers Development.

The national perspective of water resources development envisages construction of about 185 cubic kilometre of storages. These storages and the interlinks will enable additional utilisation of nearly 210 cubic kilometre of water for beneficial uses, enabling irrigation over an additional area of 35 million hectare, generation of 34 million kilowatts of hydro power and other multi-purpose benefits.

In order to give concrete shape to these proposals, Government of India has set up National Water Development Agency in 1982 which is carrying out prefeasibility/feasibility level studies of linking of various rivers both in Peninsular as well as Himalayan Rivers Development Components based on internationally accepted norms.

6.2.6 Flood protection measures in India over the years

In India, flood protection measures using embankments were in existence for centuries. This is evident from the old embankments constructed by private individuals for the protection of their lands.

The inadequacy of the individual efforts in the sphere of flood control led to governmental interest in the problem chiefly during the past century. As a result of this, a number of well-planned embankments were constructed on some of the rivers, which were causing recurrent flood damage. These measures were largely to give protection to the commanded areas of the canal systems in northern India, and the deltaic tracts of east flowing rivers in Orissa, Andhra Pradesh and Tamil Nadu.

In the fifties, the Damodar Valley Dams, viz., Tilaiya Dam (1953), Maithon Dam (1957) and Panchet Dam (1959) were constructed for multipurpose development of Damodar Valley, which included flood control also. A barrage at Durgapur downstream of the Maithon and Panchet Dams and an upstream dam at Tenughat were also constructed and are operating as a unified system for flood control apart from other major purposes of irrigation, power generation and water supply requirements. The system has helped to moderate the intensity of the floods in the lower valley considerably and the Damodar is no longer, a 'river of sorrow' of the pre-project era.

Kosi Flood Control Schemes (1959) have certainly helped in checking the movement of the river in westward direction and provided better protection to a large area of about 2.5 lakh hectares which used to be ravaged by floods. Large-scale economic development has come up in this area.

Similarly, storage reservoirs such as Hirakud, Ukai, Bhakra, Beas, Chambal Dam and Nagarjunsagar have either protected some areas from floods or have reduced their intensity considerably. In addition to these, Baigal reservoir in Uttar Pradesh, Rengali Dam and Bhimkund project in Orrisa, Multipurpose reservoirs on the Subernarekha for the benefit of Orissa, Bihar and West Bengal, Halai Dam in Madhya Pradesh and Mecharbali Dam in Karnataka have been useful for reduction of flood fury. The Kangasabati Reservoir in West Bengal also takes care of flood problem in the downstream.

A number of drainage schemes have been taken up in the States of Punjab, Haryana, Rajasthan and Gujarat. Such schemes have also benefited the States of Uttar Pradesh and West Bengal and water logged areas in Punjab and Haryana. The Krishna Godavari Delta Drainage Scheme in Andhra Pradesh has also resulted in positive developments.

Similarly, a number of schemes for channel improvements, raising of villages, anti-erosion and town protection works have been taken up towards protection from floods.

In addition to the above for tackling the flood problem in Ganga and its tributaries and to facilitate effective coordination of flood management among the Ganga Basin States, Government of India set up Ganga Flood Control Board and its Secretariat, namely Ganga Flood Control commission in April, 1972. The Ganga Flood Control Commission has prepared comprehensive plans for flood management for all the 23 river systems of Ganga by the year 1990. The Comprehensive Plans have been sent to State Governments for preparing detailed schemes based on actual ground surveys and investigations and implementation. Similarly, Government of India had set up Brahmaputra Board under an Act of Parliament (46 of 1980) in December, 1981. The objective of the Board is mainly to prepare a Master Plan for the control of floods and bank erosion and improvement of drainage in the Brahmaputra and Barak Valley. The Board has prepared Master Plans for Main Brahmaputra River (1986) and Barak Valley (1988) and 38 tributaries. These have been also forwarded to States for taking further action towards their implementation.

Flood management schemes are taken up by respective State Governments in their successive plans. It is estimated that a total of 16,199 km length of embankments and 32,003 km length of drainage channels were constructed; a total of 906 towns were protected and 4,721 villages were provided protection from floods upto the year 1997. From the above works, it is estimated that an area of 14.37m.ha has benefited.

The evolution of flood management policy in India can be summarized as below:

Before 1947	Main emphasis on flood embankment
1954	Policy statement in Parliament: Two document presents " Floods in India-problems and Remedies" and "The Floods in the Country" - absolute immunity from flood damage is not physically possible even in the distant future.
1957	High level committee on floods -Non structural measures were recommended
1964	Ministerial committee on flood control -Non structural measures were emphasized
1972	Ministers Committee on floods and flood relief -additional storage for flood moderation -Legislation to prevent encroachment of river -Restricting the costly anti erosion works to important locations
1980	Rashtriya Barh Ayog (National flood commission): A comprehensive report 207 recommendations covering entire gamut of flood problems CWC in 1987 reviewed implementation status of these recommendations and found not much progress. even now (in 2005) not much progress.
1987	National water policy -Basin master plans, watershed management catchment area treatment
1999	National commission for integrated water resources development -Efficient management of flood plains and other non structural measures -Performance of embankments to be evaluated -Flood forecasting network to be extended

It may be mentioned here that the organizations that are responsible for the management of flood in the country are the following.

State Flood Control Department	Planning and execution of flood management works
Central Water Commission	(River Management Wing) Coordination and guidance
Ganga Flood Control Commission	Master plans for flood control in 23 sub

	basins of Ganga have been prepared
Ganga Flood Control Commission	(1981) Survey and Investigation, Preparation of plans for flood control and bank erosion in Brahmaputra and Barak Valleys.

6.2.7 Flood management initiatives

Flood management activities can be broadly classified into four major groups:

i. Attempts to modify the flood
ii. Attempts to modify the susceptibility to flood damage
iii. Attempts to modify the loss burden
iv. Bearing the loss

All these measures for flood management can be classified as under:

- Structural measures
- Non-structural measures

Broadly, all measures taken up under the activity of "Modifying the flood" which are in the nature of physical measures are "Structural measures", while the others which are taken up as management tools without major construction activity are grouped as "Non-structural measures". These are explained in the subsequent sections.

6.2.8 Structural measures for flood mitigation

The general approach to tackle the problem of floods in the past has been in the form of physical measures with a view to prevent the flood waters from reaching potential damage centres. This approach had been extensively constructed in the Godavari, Krishna and Cauvery Deltas in South India and also in some areas of Indo-Gangetic plain.

The main thrust of the flood protection programme undertaken in India so far has been in the nature of taking structural measures like:

i) Embankments, flood walls, sea walls.
ii) Dams and reservoir
iii) Natural detention basin
iv) Channel improvement
v) Drainage improvement
vi) Diversion of flood waters

Embankments, flood walls, sea walls

The most common and generally economical form of protection to provide immediate relief from inundation is construction of embankment. The embankment system along the river is planned to restrict the river in its existing course and they are designed to avoid over-flowing of banks by increasing the channel capacity to pass the probable floods. Generally, these are constructed with easily available earth in the nearby area. The embankments of the pre-independence period and those came up after independence through plans have provided considerable protection to life and property of people living in flood plains.

As embankments prevent passage of river water into adjoining area even during high floods which otherwise could have been inundated by silt laden river water, the adjoining land is deprived of the fertilizing effect of silt, But there is no conclusive evidence to establish the so-called fertilizing effect of silt. It is also a fact that sometimes flood waters when spread over the flood plains adversely affect productivity of the land.

Also, in the case of embankments when constructed along the river banks, the flood wave movement becomes restricted which causes general increase in flood stages upstream of the embanked section. In alluvial reach, the embankments are continuously threatened by erosion. The progressive of bed level requires progressive rise of the embankment to ensure protection.

Embankments attract new settlements as a result of protection from floods offered by them. State Governments have been very slow on maintenance of these structures due to inadequacy of funds. Another problem is that due to unprecedented rains, if a breach occurs, the effect of such a flood to the settlements will be unexpected and devastating. Such breaches are reported from the States of Bihar, Assam, U.P. and West Bengal. Proper maintenance of the embankments involving the beneficiaries and educating the masses on the consequences of occupying the flood plains are necessary.

The benefits of embankments and embankment scheme in reducing distress and damage due to floods are very evident. The benefits achieved for the some case of the kosi project is as follows:

Kosi is a perennial river originating from Himalayas in Nepal whose three main streams viz. the Sun Kosi, the Arun Kosi and the Tamur Kosi meet at Triveni in Nepal to form Sapta Kosi or simply called as Kosi in the plains of Bihar. The notorious meandering behaviour of Kosi is apparent from the fact that it has changed its course for a width of about 112 km its lower reach in Bihar in a period of about 250 years, as a result of which the flood problem faced by people living in the area was acute. A barrage was constructed in 1963 for irrigation, power and regulated flow downstream. Flood embankments on both sides of the river were constructed arresting its unique translatory movements giving a great sense of security to the people of the area which is apparent from the fast changing outlook in the districts of Purnea, Saharsa and Darbhanga and, therefore, giving protection to an area of about two lakh ha in Bihar.

The flood embankments on both the banks of Kosi river which were mostly completed as early as 1957 have on an average protected 1.17 lakh acres of land in Darbhanga and 4.11 lakh acres in Saharsa districts of Bihar from the ravages of flooding. Kharif crops on this land were formerly badly damaged and even Rabi sowing was sometimes affected by standing water and excessive moisture. Severe damage was also caused to properties like building, orchards etc. Prevention of flooding in this area on account of the project has rendered following incidental benefits:

i) Construction of a network of metalled roads improved communication and there by addition to the convenience and prosperity of the people.
ii) Communications on the Mansi-Supaul and Saharsa-Purnea sections of the North Eastern Railway, which were formally cut off in the wet season, are now possible throughout the year. The longer running period will mean a rise in railway earnings from fares and freights. Villagers served by these lines will have better facilities for carrying their produce to markets.
iii) Replacement of improvised and temporary thatched houses of private individuals, commercial concerns and govt. Departments by pucca buildings reflecting a better standard of living and sense of security.
iv) Reclamation for cultivation of large areas of land which were formerly infested with 'Kans and Pater'
v) Opening of small-scale industries and even major factories like the sugar factory proposed at Banmankhi.
vi) Reduction of flood and waterlogging and consequential improvement in general health.
vii) Kosi barrage has opened up all weather communication between Saharsa-Purnea district on the bank and Darbhanga district on the other bank.
viii) A metalled road from Bathnaha to Bhimnagar had been constructed by the project. This road provides much-needed means of communication in this locality.

In addition to benefits to Bihar (India) from the project, Nepal is also drawing large benefits from it through protection from flood, stability of river in the upstream to some extent and better communication system at flood time apart from power and irrigation facility.

The Programme Evaluation Organisation of the Planning Commission undertook evaluation of the embankment projects with a view to assess the economic benefits of the projects. They also substantiated benefits from the project.

The embankments, however, have created some potential dangers to the zone inside embankments, which is liable to experience greater threats of floods than before. The area lying within 5 km of left embankment is submerged under water. Besides, the problem of water logging also needs to be separately addressed and necessary measures taken.

Dams and reservoir

Human issues involved in the case of dams and reservoirs are evacuation and resettlement of people in the reservoir area, environmental impacts due to developmental activities and increase in population etc. Consequences of dam failure-possible damages to life and property in the downstream and human encroachment on the flood plains due to the security provided by dams are also to be taken into account. While planning all the above works, impact due to the scheme on its surrounding and on the settlements in the downstream area need be taken into account.

Maximum attention is to be given while framing reservoir regulation policies so that optimum utilisation of water resources is possible and at the same time flood control and issues related to the people living in the downstream of the reservoir are taken care of.

Construction of dams and reservoirs is adopted as a major activity to control floods by storing flood water so that the stored water could be released subsequently when the flood has receded and the downstream river channel is in a position to contain the flow without causing floods. The main advantage reservoirs is that apart from moderating the flood peaks, the stored water can be used for multipurpose uses such as irrigation, power generation, industrial requirements and domestic uses etc. In the case of flood control reservoirs, proper reservoir regulation schedule can be worked out for optimum benefit from the project as a whole from the flood control point of view, and it is advisable that specific flood cushion is allocated in the reservoir although incidental benefit of flood control to some extent is available from any reservoir scheme.

6.2.9 Non-structural measures for flood management

The present trend to reduce the losses incurred by flooding is equally towards non-structural measures. This section examines some such techniques.

Flood plain management and zoning

Heavy encroachment of flood plains has been responsible for increasing trend of damage over the years. The basic concept of flood plain management is to regulate the land use in the flood plains in order to restrict the damage due to floods, while deriving maximum benefits from the same. This is done by determining the locations and the extent of areas likely to be affected by floods of different magnitudes/frequencies and to develop those areas in such a fashion that the resulting damage is minimum in case floods do occur. Flood plain zoning, therefore, aims at disseminating information on a wider basis so as to regulate indiscriminate and unplanned development in flood plains and is relevant both for unprotected as well as protected area. Flood plain zoning recognizes the basic fact that the flood plains are essentially the domain of the river, and as such all developmental activities in flood plains must be compatible with the flood risk involved.

The basic concept of flood plain zoning is to regulate the land use in the flood plains in order to restrict the damage by floods which are bound to occur from time to time. Flood plain zoning, therefore, aims at determining the locations and the extent of areas likely to be affected by floods of different magnitudes/frequencies and to develop those areas in such a fashion that the resulting damage is reduced to the minimum. It, therefore, places limitations on indiscriminate and unplanned development of both the unprotected as well as protected areas. In the former case, boundaries of restricted areas are established to prevent indiscriminate growth; while in the protected areas, only such categories of development can be allowed which will not involve unduly heavy damage in case of failure of the protection provided, while ensuring that the valuable flood plains are simultaneously put to development use.

Flood plain zoning is not only necessary in the case of floods caused by rivers but is also useful in reducing the damage caused by drainage congestion, particularly in urban areas, where on grounds of economy and other considerations, urban drainage system is not designed for the worst possible conditions and presupposes some damage during storms whose magnitude exceeds that for which the drainage system is designed.

The steps involved in implementation of flood plain zoning measures could be broadly indicated as follows:

i) Demarcation of areas liable to floods

ii) Preparation of detailed contour plans of such areas on a large scale (preferably 1:15,000) showing contours at an interval of 0.3 to 0.5 metres

iii) Fixation of reference river gauges and determination of areas likely to be inundated for different water levels and magnitudes of floods

iv) Demarcation of areas liable to flooding by floods of different frequencies like once in two years, ten, twenty, fifty and hundred years. Similarly, area likely to be affected on account of accumulated rainwater for different frequencies of rainfall like 5,10,25 and 50 years

v) Delineation of the types of use to which the flood plains can be put to in the light of (i) to (iv) above with indication of safeguards to be ensured.

The need for flood plain zoning has been recognized in the past also. As far back as 1973-74, Central Water Commission had prepared guidelines for flood plain zoning which were approved by the Central Board. Since the implementation of these guidelines needed statutory backing, CWC also prepared a model draft bill which was circulated in 1957 by the then Ministry of Irrigation, to all the States for enacting legislation.

The Rashtriya Barh Ayog (1980) in its report has recommended that Flood Plain Management measures should be undertaken, wherever necessary legislation enacted in other States. However, the response from States except Manipur has not been encouraging. Manipur enacted a legislation in Sept., 1978 which came into force in Dec., 1985.

One of the reasons advanced by the State Governments for non-implementation of flood plain zoning measures has been the non-availability of survey maps on suitably large scale to enable proper demarcation of flooded areas. To overcome this difficulty, Central Water Commission had initiated in 1978 programme for such surveys under the Central Sector through the Survey of India to assist the State Governments in the preparation of flood risk maps. These surveys cover areas along main Ganga, Yamuna, Ramganga, Roopnarayan, Jalangi and other flood prone rivers of West Bengal, tributaries of Brahmaputra like Burhi Dehing, Desang and Dikhoo, Sutlej and Ravi etc., which were taken up in a phased manner as per the priorities indicated by the States. However, the programme has now been discontinued at Central level. With the available data/maps, it should now be possible for the State Govts. to make a start and demarcate the zones for different flood frequencies.

This is an area where immediate attention of all the State Governments has to be attracted considering the extent of human issues involved. With the availability of remote sensing maps, preparation of flood risk maps has become easier. The need of the hour is to prepare flood risk maps for all the frequently flood prone river plains in the country, enact and enforce laws for implementing the zoning regulations, constant monitoring of the flood situation using remote sensing and to plan mitigating measures accordingly.

Flood Proofing

Flood proofing measures help greatly in the mitigation of distress and provide immediate relief to the population in flood prone areas. It is essentially a combination of structural change and emergency action, not involving any evacuation. The techniques adopted consist of providing raised platforms for flood shelter for men and cattle and raising the public utility installations above flood levels and other facilities to make various essential services flood proof so that the miseries of people can be reduced to minimum even when flooding occurs.

Flood forecasting and warning

Flood forecasting enables forewarning as to when the river is going to use its flood plain, to what extent and for how long. As per strategy of laying more emphasis on non-structural measures, Central Water Commission has established a nationwide flood forecasting and warning system. With reliable advance information/warning about impending floods, loss of human lives and moveable properties, human miseries can be reduced to a considerable extent. People and cattle can be shifted to safer places. Similarly, valuable moveable properties can be removed to safer places beyond area to be inundated. Large number of reservoir schemes to harness water resources for irrigation, power etc. were undertaken in the country during various plan periods. Realising the great potential of reservoirs in impounding floods and regulating the flows downstream for flood moderation, flood control has been sought to be achieved as one of the objectives in multi-purpose dams by providing flood cushion. Inflow forecasting for these reservoirs is very important for optimum reservoir operation.

Present flood forecasting network in India

Flood forecasting and flood warning in the country commenced in a small way in the year 1958 with the establishment of a unit in the Central Water Commission (CWC), New Delhi, for flood forecasting for river Yamuna at Delhi. This has by now grown to cover most of the flood prone interstate river basins in the country. This organization is presently responsible for issuing flood forecasts at 157 stations of which 132 stations are for river stage forecast and 25 for inflow forecast.

The flood forecasting system of CWC functions under Member (River Management). Seven field offices of Chief Engineers, ten offices of Superintending Engineers and nineteen divisional offices are in charge of management of the forecasting work. These offices are responsible for hydrological and hydro meteorological data collections such as gauge, discharge and rainfall data, their transmission from field stations to the central control rooms, formulation of forecasts and dissemination to various concerned Central/State Governments, Media and other users. Forecast about water level in river likely to be attained as a result of flood and volume of inflow to reservoir is issued by concerned officers everyday in the morning. The forecasts are also transmitted to CWC headquarters at New Delhi where daily bulletins are prepared for the country as a whole and sent to all concerned departments of the Government.

According to the present norms of the Central Water Commission, a forecast is considered to be reasonably accurate if the difference between forecast and corresponding observed level of the river lies within $\pm$ 15 cm. In case of inflow forecasts, variation within $\pm$ 20 percent is considered acceptable. On an average, about 6,000 flood forecasts are issued every year with a maximum of 7,943 forecasts issued during the year 1998. The forecasts issued by CWC have been consistently accurate as a result of which the flood forecasting and warning services have rendered immense benefit to the people in the flood prone areas.

Like the river stage forecasts, the inflow forecasts issued by CWC have also been consistently accurate. This has provided immense benefit to the authorities of concerned dams and barrages for systematic operation of the reservoirs for optimum utilization of the water resources and for the control of floods.

However, proper integration of the flood forecasting system with disaster mitigation works can go a long way to reduce flood damage and alleviate distress to the people affected by flood. Also, where the warning messages are utilized with other non-structural measures like flood plain zoning activity and flood fighting in a comprehensive manner, the outcome of the forecasting and warning system can become more effective.

Flood fighting

Flood fighting covers building temporary dykes along the river, dowel bunds on the banks, closing small breaches immediately, attending to scour, wave wash, sand boils etc. evacuating goods and equipment out of the reach of flood zone, protecting equipment with plastic sheets etc. When floods occur, the existing facilities for water supply and sewerage get disrupted affecting the health of the population. The inundation and deterioration of the quality of food grains, destruction of agricultural crops and health of livestock may lead to famine or at least nutritional deficiencies. Stagnant water becomes the breeding ground for mosquitoes affecting public health. Public health operations should ensure availability of supplies and equipment, co-ordination with other organisations engaged in disaster relief and procedure for immediate mobilisation of personnel to eliminate health hazards. Flood fighting measures normally involve:

- Strengthening of Central, State and District Flood Control Rooms.
- Evacuation of flood victims.
- Air dropping of food packers.
- Close review of flood relief measures.
- Release of emergency funds to local bodies and thence to the flood victims.
- Supply of food and other rations.
- First aid and health operations.
- Supply of essential commodities like Kerosene, oil, petrol etc.
- Plugging of breaches.

- Restoration of road/rail links.
- Restoration of tube wells and other agricultural machinery.
- Pumping out water from ponds and low-lying areas in cities/villages.
- Restoration of public assets such as roads, bridges, irrigation systems and structures, power installations, public buildings, municipal roads, sewerage and water supply schemes, paved streets and drains etc.
- Voluntary efforts by Red-Cross, Home-Guards, Panchayats, local people etc.

Most of the above activities are of immediate nature. Participation by voluntary organisations and local people are necessary for flood fighting. There is a need to give training to voluntary organisations and other non-governmental organisations in the field. All the above issues are directly aimed at reducing the losses due to flood to life and property.

Flood insurance

Flood insurance has several advantages as means of modifying loss burden. The insurance does not reduce the flood loss potential directly, but it provides a mechanism for spreading the loss over large number of individuals. It is advantageous both to the public and the Government.

So far, flood insurance has not been adopted widely in India. Though flood risk has been included in 'cover' issued by the General Insurance Companies in India, it is more popular in urban areas and big towns where damage due to inundation caused mostly by excessive rainfall is taken care of. The insurance companies have also not been able to arrive at different rates of insurance premium for different flood prone regions in the country. As such, they continue to charge uniform rate irrespective of the fact that property was located in high flood risk area compared to the other areas. The insurance companies are facted with a difficult choice. If they levy uniform rates in all areas, the people in flood prone areas would most likely take out the policy which may become too large a burden. If an attempt is made to charge rates proportionate to the flood risk, the premium may work out much more than what the property owner might be willing to pay. Another problem being faced by the insurance companies is regarding the assembling of basic data for working out a fair and equitable premium for all areas according to flood risk. The insurance cover works successfully for a class of people who are subjected to such risks more or less equally. In case of floods, the risk of loss even in areas liable to flood is not equal. For example, those owning land or property at lower elevation in a flood plain are subject to higher risk both in magnitude as well as in frequency. Therefore, those living at high levels would not be equally willing to obtain insurance cover, at the same rate. It is quite a difficult task to accurately adjust insurance premium. All these difficulties need to be sorted out and there is an urgent need to make insurance schemes attractive to insures and the insured.

Module 6

MANAGEMENT OF WATER RESOURCES

LESSON 3

REMOTE SENSING AND GIS FOR WATER RESOURCE MANAGEMENT

Instructional objectives:

On completion of this lesson, the student shall learn about:

1. The techniques of Remote Sensing and Geographic Information System (GIS)
2. Different types of remotely sensed images
3. Application of Remote Sensing in water resources engineering
4. Application of GIS in water resources engineering

6.3.0 Introduction

The term Remote Sensing is applied to the study of earth's features from images taken from space using satellites, or from nearer the earth using aircrafts. The technique of remote sensing has picked up in the past half a decade, largely due to the availability of digital computers, improved communication systems, digital imaging techniques and space technology. Remotely sensed data can be said to have its origin in photography, where the information about a target area is interpreted from photographs. Later this technique was extended to aeroplane - borne cameras giving rise to the science of aerial photography. This technique is still used, but largely the signal cameras have been replaced by Laser operated ones where the reflectance of a Laser beam projected from the bottom of the aircraft is sensed by electronic sensors.

In this chapter we shall discuss remote sensing using satellite as India has strived ahead in this field and made good use of satellite images. The satellite launching program of our country is one of the most ambitious in the world, and is still continuing to be so in the future as well. Amongst other fields, the Water Resources Engineers have benefited greatly by using satellite imaging techniques, some applications of which have been highlighted in this chapter.

The other topic that is discussed in this lesson is the Geographic Information System (GIS) that has wide applications in planning any spatially distributed projects. Fundamentally, a GIS is a map in an electronic form, representing any type of spatial features. Additionally, properties or attributes may be attached to the spatial features. Apart from its spatial data analysis capabilities, it provides an interface to remotely sensed images and field surveyed data. This technique has specifically benefited the Water Resources Engineers, which has been discussed in some detail.

6.3.1 Remote sensing through satellites

Remote sensing means assessing the characteristics of a place (usually meant as the surface of the earth) from a distance. Though this term was coined during the 1960's, similar technology had been practiced earlier like fitting a camera to a balloon and allowing it to float over the earth's surface taking pictures, which may then be developed and interpreted for specific purpose like geology, agriculture, forestry etc. Photogrammetry, that is, taking pictures of the land surface from a low flying aircraft and comparing subsequent pictures to obtain the terrain relief has been extensively used in the last century and many books have been written on the subject.

In satellite remote sensing, too, cameras are fitted to the orbiting satellite and are focussed towards the earth. However, the cameras are special in the sense that they are sensitive to other wavelengths of the electromagnetic spectrum as well. As may be observed from Figure1, the electromagnetic spectrum identifies the wavelength of the electromagnetic energy, of which the visible portion (or light) occupies only a small portion. Actually, electromagnetic energy refers to light, heat and radio waves. Ordinary camera or the human eye are sensitive only to the visible light. But the satellites are equipped with Electromagnetic Sensors that can sense other forms of electromagnetic radiations as well. This includes not only the Blue (0.4-0.5μm), Green (0.5-0.6μm) and Red (0.6-0.7μm) of the spectrum but also longer wavelength regions termed as the Infrared (IR) spectrum (0.7-1000μm), which can again be further subdivided into the following:

a) Photographic IR : 0.7-0.9μm
b) Very near IR : 0.7-1.0μm
c) Reflected/Near IR : 0.7-3.0μm
d) Thermal IR : 3.0-1000μm

Still longer wavelength is the microwave portion of the spectrum, which extends from 3000μm to 3m. The common remote sensing systems operate in one or more of the visible, reflected-infrared, thermal-infrared and microwave portions of the spectrum.

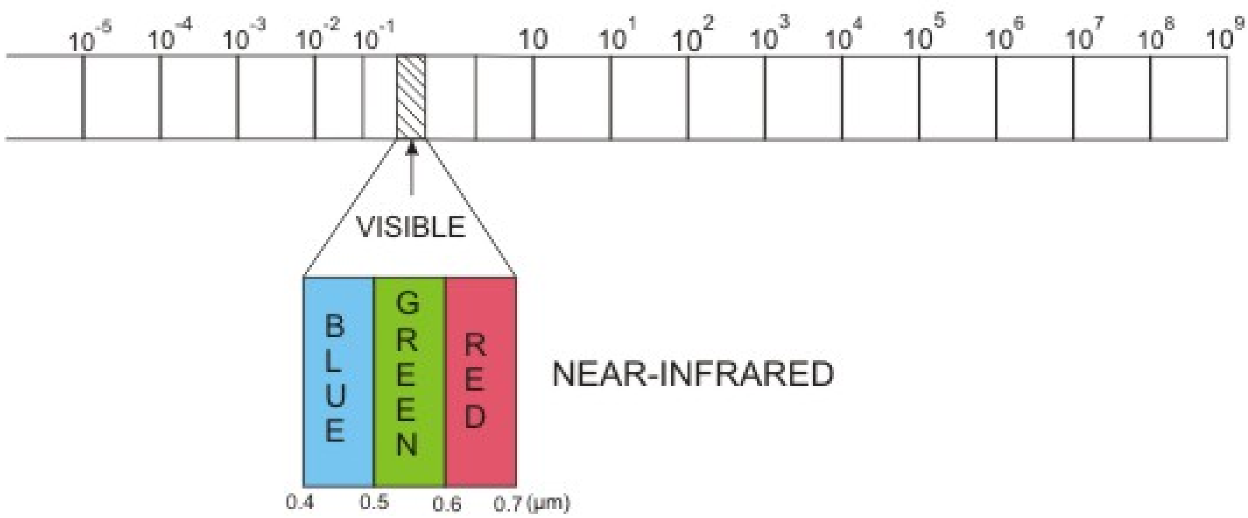


FIGURE 1. The energy spectrum

6.3.2 Interaction of electromagnetic radiation and earth

Electromagnetic energy of the sun incident on the earth's surface reaches fully upto the top of the atmosphere. However, as illustrated in Figure 2, not all of this energy reaches the surface of the earth, since part of the energy gets either scattered, absorbed or reflected by the atmosphere or cloud cover, if any. Only a part is transmitted upto the earth's surface. Specifically, it may be said that although the electromagnetic radiation reaching the top of the atmosphere contains all wavelengths emitted by the sun, only specific wave bands of energy can pass through the atmosphere. This is because the gaseous components of the atmosphere act as selective absorbers. Molecules of different gases present in the atmosphere absorb different wavelengths due to the specific arrangement of atoms within the molecule and their energy levels. The main gaseous component of the atmosphere is nitrogen, but it has no prominent absorption features. Oxygen, Ozone, Carbon Dioxide and Water Vapour, the other major components absorb electromagnetic wavelengths at certain specific wavelengths. The wavelengths at which electromagnetic radiation are partially or wholly transmitted through the atmosphere to reach the surface of the earth are known as atmospheric windows, as shown in Figure 3. Since these radiations reach the surface of the earth, they are useful for remote sensing as they would be reflected or absorbed by the features of the earth giving the typical signatures for the sensors in the satellite (or any other space borne device) to record. This is shown graphically in Figure 4.

The remote sensing system sensors are designed in such a way that can capture information for those wavelengths of electromagnetic radiation that occur within the atmospheric windows.

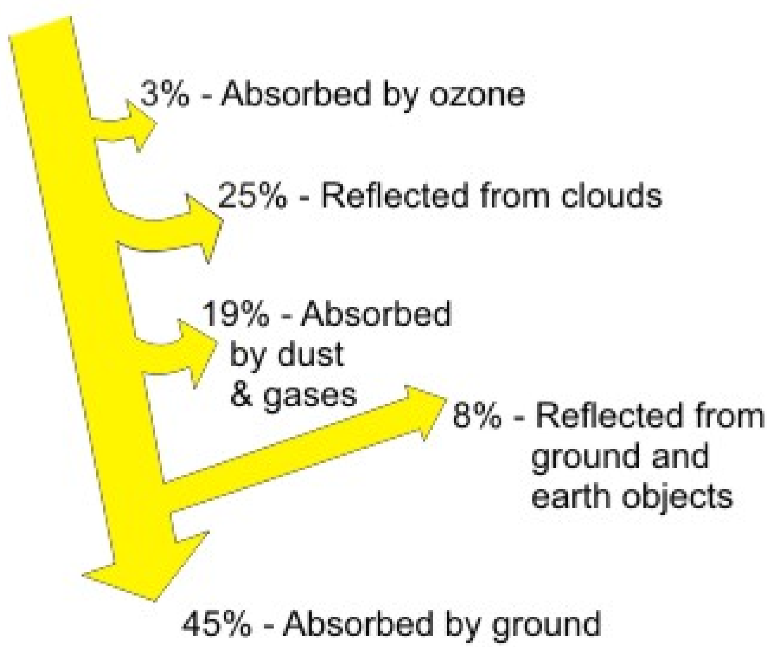


FIGURE 2. Fate of incoming solar radiation to earth

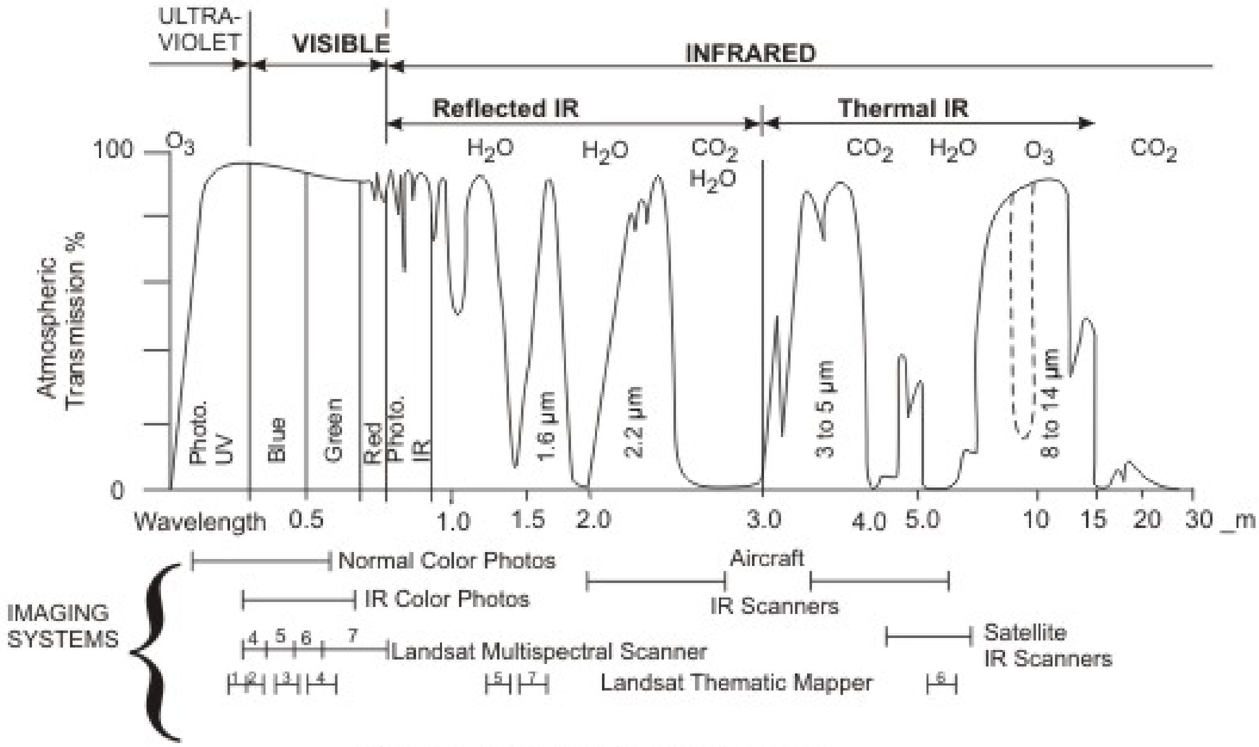


FIGURE 3. Atmospheric windows

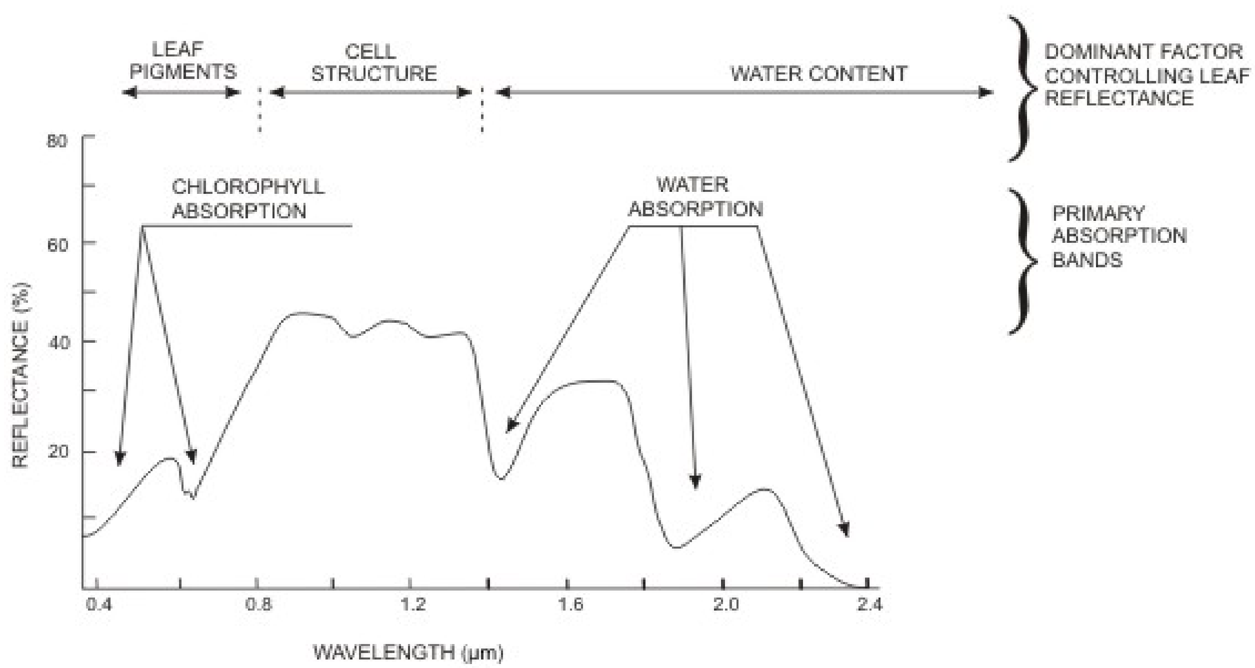


FIGURE 4. Spectral signature of vegetation

6.3.3 Interaction of electromagnetic radiation with a surface

When electromagnetic radiation strikes a surface, it may be reflected, scattered, absorbed or transmitted. These processes as not mutually exclusive: a beam of light may be partially reflected and partially absorbed. Which processes actually occur depends on the wavelength of the radiation, the angle at which the radiation intersects the surface and the roughness of the surface. Reflected radiation is returned from a surface at the same angle as it approached, the angle of incidence thus equals the angle of reflectance. Scattered radiation, however, leaves the surface in all directions. Whether or not incident energy's reflected or scattered is partly a function of the roughness variations of the surface compared to the wavelength of the incident radiation. If the ratio of roughness to wavelength is low (less than one), the radiation is reflected whereas, if the ratio is greater than one, the radiation is scattered. A surface which reflects all the incident energy is known as a Specular reflector whereas one which scatters all the energy equally is a Lambertian reflector. Real surfaces are neither fully specular nor fully lambertian. However, for remote sensing purposes, a Lambertian nature is better. A remotely sensed image of a fully specular surface gives a bright reflectance (or signature) for one position of the camera and dark image at other positions. If the surface is uniform lambertian, then the reflectance obtained for the surface will be same irrespective of the location of the camera because the radiation from the surface would be scattered equally in all directions. Most natural surfaces that are observed using remote sensing systems are approximately lambertian at visible and infrared wavelengths.

6.3.4 Interaction of electromagnetic radiation with earth surface features

From the general discussion on the nature of interaction of electromagnetic energy with any surface, we turn on to the earth features as these would be useful in Water Resources Engineering.

As observed from Figure 5, it is seen that a part of the electromagnetic energy reaches the earth's surface, a part of it gets absorbed by the body, a part gets transmitted within the body, and a part gets reflected from the surface of the body. The proportion of energy that is reflected, absorbed and transmitted varies with the particular earth feature, like whether it is vegetation, water, urban landscape, etc. Besides, the proportion of energy is also dependent on the wavelength of the electromagnetic spectrum that is interacting with the surface. Thus, for a particular feature, the proportion of energy that is reflected, absorbed or transmitted varies with the wavelength that is interacting.

This means that two different features may reflect equal proportion of energy in one wavelength range and may not be separately identified but for another wavelength range their difference reflectance may allow a sensor to distinguish between the two features. This variation in interaction of electromagnetic energy with any surface can be explained in the way we distinguish objects by separate colours. As we know, the wavelengths in the visible range of the spectrum strike all surfaces, but we observe different colours because each surface reflect only a particular wavelength and absorb the rest.

Most of the sensors in remote sensing systems also operate in the wavelength regions in which the reflected energy predominates and thus the reflectance property of surfaces is very important. Of course, the sensors do not capture only the reflected energy in the visible range of wavelength but different sensors are designed to capture the reflected energy in other ranges of wavelengths as well.

The reflectance characteristics of the different features of the earth surface may be quantified by measuring the portion of incident energy that is reflected by a surface. This reflected energy is measured as a function of the wavelength and is called Spectral Reflectance. Quantitatively this is defined as the ratio of the energy of the wavelength reflected from an object and the energy that is incident upon it.

Spectral reflectance of any object usually varies according to the wavelength of the electromagnetic radiation that it is reflecting. A graph showing the spectral reflectance of an object for various wavelength is known as a Spectral Reflectance Curve (Figure 6). The pattern of a Spectral Reflectance Curve gives an insight into the spectral characteristics of the object. It also helps in selecting the wavelength bands which may be suitable for identifying the object.

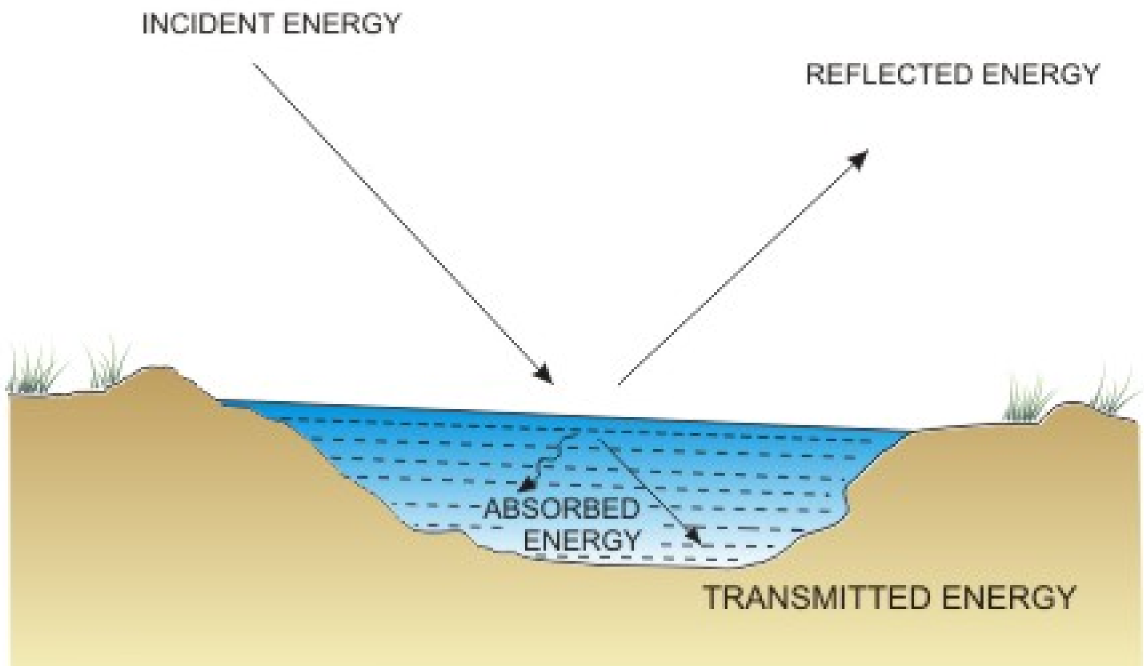


FIGURE 5. Fate of energy reaching an object

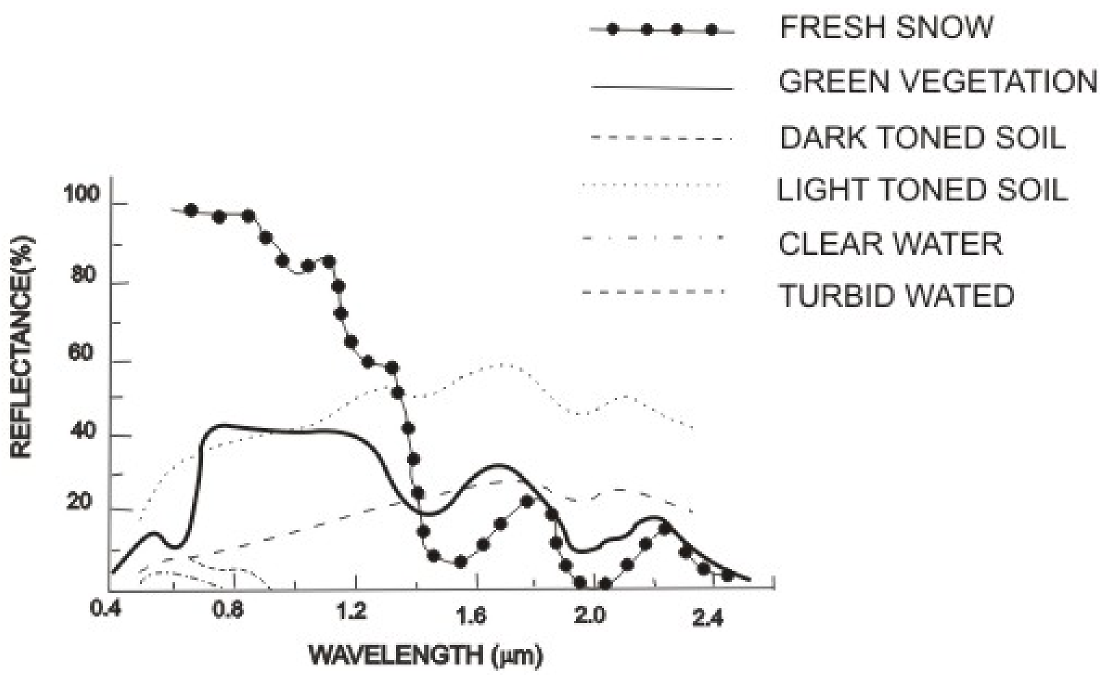


FIGURE 6. Spectral reflectance curves of vegetation, soil, water and snow

6.3.5 Spectral signature of earth features

For optimum use of remotely sensed data in various wavebands in identification and differentiation of different objects or features on earth, it is important to have a thorough knowledge and understanding of their spectral reflectance characteristics. Usually, the features that are classified through satellite remote sensing may be grouped into inanimate objects like soil, minerals, rock, water, etc. or animate object which is usually vegetation. Soil is a heterogeneous mixture of minerals, containing considerable amount of organic matter and often moisture. The proportion of these determine the spectral characteristics of the particular soil type. Rocks are assemblages of minerals and hence the reflectance spectrum of rocks is a composite of individual spectra of its constituent minerals. As for vegetation, the reflectance spectra vary according to the freshness of the leaves. Thus the characteristics of the reflectance of various earth features for different electromagnetic wavelength bands is used to identify different earth objects and are hence also known as Spectral Signatures. A study of the spectral reflectance characteristics of natural earth surface features shows that the broad features are normally separable. In the following paragraphs, we discuss the spectral signatures of certain typical earth features, natural and artificial.

Vegetation

The spectral signature or reflectance of healthy green vegetation is as given in Figure 6. In the visible range of electromagnetic wavelength spectrum, it has an absorption band in the blue and red parts because of the presence of chlorophyll. One may notice these at

0.45μm and 0.65μm. Even within the green part of the spectrum, only 10 to 15 percent of the incident light is reflected. The reflectance peak is seen to be at 0.54μm, which is in the green wavelength region.

The reflectance property of healthy vegetation is seen to be much larger (40 percent or more) in the infrared portion of the spectrum and is nearly constant from 0.7μm to about 1.3μm. In this range of electromagnetic spectrum, the reflectance variation is different for different plants and also between healthy vegetation and stressed vegetation. Hence, a reflectance measurement in this range permits one to discriminate between different species of vegetation, though this differentiation is not very apparent in the visible range of the spectrum.

Beyond 1.3μm, low spectral reflectance for vegetation is noticed at 1.4μm, 1.9μm and 2.7μm with intermediate peaks at about 1.7μm and 2.2μm.

Soil

The spectral signature of soil is simpler in soils compared to that by vegetation since all the incoming radiation is either reflected or absorbed due to very little transmittance. A typical reflectance curve for soil shows increase in wavelength in the visible and near-infrared regions (Figure 6).

The reflectance property of soil varies with soil moisture content, texture (that is, the relative content of sand silt and clay that makes up the soil), surface roughness, colour, content of organic matter, presence of sesquioxides, etc. In the visible portion of the spectrum, there is a distinct decrease in reflectance as moisture content increases, since more moisture in soil makes a soil appear darker causing less reflectance. Soil texture influences the spectral reflectance by the way of difference in moisture holding capacity and due to difference in the size of the particles. Soils with higher organic matter appears as light brown to grayish in colour. The reflectance characteristics in the visible region of the electromagnetic spectrum has been observed to be inversely proportional to the organic matter content. The presence of iron oxide in soil also significantly reduces the reflectance, at least in the visible wavelength.

Water

For water resources engineer, locating areal extent of water bodies like lakes, rivers, ponds, etc. from remotely sensed data is an important task. The spectral response from a water body is complex, as water in any quantity is a medium that is semi-transparent to electromagnetic radiation. Electromagnetic radiation incident on water may be absorbed, scattered and transmitted. The spectral response also varies according to the wavelength, the nature of the water surface (calm or wavy), the angle of illumination and observation of reflected radiation from the surface and bottom of shallow water bodies. Pure clear water has a relatively high reflectance in the visible wavelength bands between 0.4 and 0.6μm with virtually no reflectance in the near-infrared (0.7μm) and higher wavelengths (Figure 6). Thus clear water appears dark on an infrared image. Therefore, location and delineation of water bodies from remotely sensed data in the higher wave bands can be done very accurately.

Water containing heavy sediment load, as in the water in the estuary, has a turbid composition. The sediment suspended within the body of water tends to increase the reflectivity at longer wavelengths of the visible part of the spectrum, that is, in the yellow/red range.

Man-made structures

Sometimes it is required to identify artificial structures that is useful to an engineer. For example roads, paved surfaces, canals, and even dams and barrages can be identified from remotely sensed images by their reflectance characteristics. Many of these, especially linear features, are clearly discernible in the visible waveband of electromagnetic spectrum.

6.3.6 Remote sensing and imaging systems

The remotely sensed images are captured by sensors fitted to satellites (and at times below aircrafts) that work on two basic technologies (Figure 7). One of these, the Passive System, records the reflected electromagnetic energy of the earth, the source of the energy being the radiation of the Sun. The other, called the Active System, employs its self-generated pulses and records the reflected pulse. These two systems may be compared to taking photographs in sunlight and with flashlight respectively. The active remote sensing systems mostly use radars that emit radiation in the microwave band of the electromagnetic spectrum. This system is useful in cases where passive systems do not give sufficient information. For example, images of flood inundated areas are important to a Water Resources Engineer. However, most of these images taken by the passive systems are blocked by cloud cover since incidents of floods are most common during the monsoons and are almost coincident with heavy cloudy days. Radar based systems, on the other hand, are able to penetrate the cloud cover and give a clear picture of the flood inundation extent.

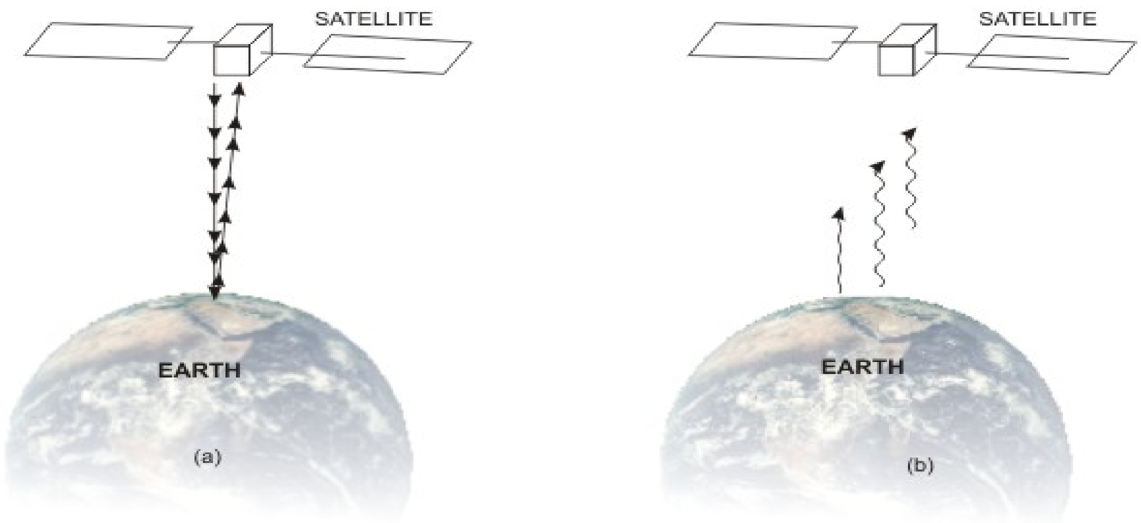
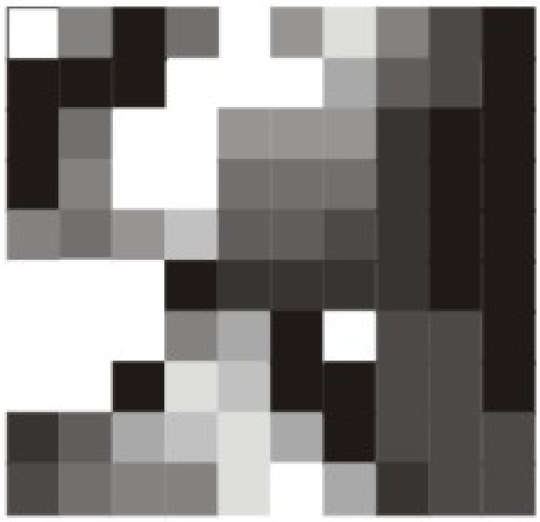


Figure 7.a Active remote sensing , using energy generated by a source.
b Passive remote sensing , which detects energy

The images recorded by a remote sensing sensor is a digital map of the scene that comprises of a regular grid array of squares, called pixels, with an unique value attached to each (Figure 8). The value of each pixel is proportional to some property, like average reflectance, recorded by the sensor for the equivalent area on the ground. The pixel values normally range from 0 to 255. For example, images recorded in the visible spectrum are usually a combination of three values for each pixel, one each for blue, green and red colours. For each colour, the pixel has a value ranging from 0 to 255. A pixel that records the image of a pure white area, will have the pixel values of all the three bands as 255. For a pure black region, the three individual bands would have values of 0. A blue looking area shall have the value 255 for the image that records the blue colour, and 0 for green and red.



10	9	20	25	26	34	35	11	20	25
76	34	11	15	41	42	77	14	30	24
47	25	13	11	10	55	41	39	42	65
42	19	15	19	22	52	71	56	34	35
34	20	16	60	27	85	35	17	25	16
25	30	29	23	18	49	38	78	19	29
30	42	20	13	57	23	73	74	60	20
19	60	45	28	40	11	79	32	48	45
64	36	55	58	38	13	80	28	64	55
36	70	31	62	15	15	70	58	76	66

FIGURE 8.a A digital image represented by pixels with different colours.
b Corresponding digital number for each pixel.

Similarly, sensors record pixel values in the infrared areas of the electromagnetic spectrum in passive systems and in the microwave areas in the active systems. The Indian Remote Sensing (IRS) satellites are, till now, equipped with active sensors that record images in four wave bands and others that record in a single wave band. The latest group of satellites available for earth imaging are the IRS-IC/ID. There are three sensors in these satellites, and each has its own characteristics, as given below.

- **LISS (Linear Imaging Self-Scanning Sensor)-III.** This medium resolution sensor that records data in four spectral bands: Two in visible range (0.52-0.59μm and 0.62-0.68μm), one in near infrared range (0.77-0.86μm), and one in short wave infrared (1.55-1.70μm) region. The spatial resolution, that is, the pixel size of the images are 23m for the first three bands and 69m in the last band.

- **PAN (Panchromatic, or single wave band).** This is a high resolution (5.8m pixel size) sensor operating in the 0.50-0.75μm range.

- **WFS (Wide Field Sensor).** This is a coarse resolution (188m) sensor operating in two bands: visible (0.62-0.68μm) and near infrared (0.77-0.86μm).

6.3.7 Spectral signatures

By deducing earth features from Multi Spectral Scanned (MSS) images the Water Resources Engineer may derive various important information of a wide region of the earth that may be useful for analysis. Hence, primarily, the earth features have to be identified from MSS images based on the Spectral Reflectance characteristics or signatures of various objects as discussed in Section 6.3.5.

An MSS data of a region comprises of two or more images of the same area that has been scanned by the remote sensing sensor. For example, the LISS-III sensor shall give four images of the area corresponding to the four spectral bands in which the data is collected. Each of these images comprise of data stored for each pixel, which is in the form of a Digital Number (DN) corresponding to the pixel's average reflectance property in the particular waveband. The DN varies from 0 to 255, and hence, each image may be printed or discussed in a gray-scale. However, all the four images for a region printed or displayed in gray-tone may not be useful individually. Hence, a combined image is produced, called the False Colour Composite (FCC) image, which combines the characteristics of the images of all the four bands.

An FCC image which simulates a colour infrared image, the visual interpretability of features is better than that from image of each band taken separately. The typical colour signatures of some of the features on the surface of the earth in standard FCC is given in the following table:

Features on the earth surface	Colour signature
Healthy reflection	

Broad leaf type Needle leaf type	Red to magenta Reddish brown to purple
Stressed vegetation Pre-visual stage Visual stage	 Pink to blue Cyan
Water Clear Turbid due to sediments	 Dark blue/black Light blue
Soil Red soil/red bed outcrop Moist soil Sand-dunes	 Yellow Distinct dark tong Yellow/white
Land-use Uncultivated land City/town	 Blue/white Blue
Others Cloud/snow Shadow*	 White Black with a few visible details

∗Shadow is not very significant in MSS satellite images with the present day spatial resolution as the scales of features are too small to aid in recognition.

Digital interpretation

Visual image interpretation requires the person to have thorough knowledge of the features being identified and their spectral reflectance characteristics. The technique is subject to human limitation. Hence, another technique – the Digital method of image interpretation – is often used in identifying earth surface features from remotely sensed images. Infact, this comprises of a very important area, the details of which may be obtained in standard textbooks on Remote Sensing and Image Processing. Here only a brief account of the process is given below.

Primarily, this is possible due to the fact that an image actually comprises of a number of pixels, each being assigned a Digital Number (DN) according to the average reflectance of the corresponding ground area in the particular spectral band. Thus, an image is nothing but a matrix of DNs. Computer algorithms are available in Image Processing Software Packages that make use of these numbers to identify the feature of land corresponding to each pixel. The numerical operations carried out on these digital images are grouped as follows:

1. Pre-processing: Removal of flawed data, correction of image.
2. Image registration: Translation, solution or stretching of the image to match earth's true geometry.
3. Image enhancement: Improving images or image patches that suffer from low contrast between pixel DN values.
4. Image filtering: Methods to identify clearly the boundary between two district regions of separate reflectance characteristics.
5. Image transforms: Combination of one or more images of different spectral bands of the same area.

6. Image classification: Automatic categorization of pixels into various land-cover classes.

Though digital image interpretation has the capability to classify earth surface features with the help of a computer, it must be supplemented with ground truthing, that is, verification of the interpreted regions with actual information gathered from the ground by field survey.

6.3.8 Application areas of Remote Sensing in Water Resources Engineering

The interpretation of remotely sensed images may provide valuable information to the Water Resources Engineer, some of which are discussed below for various fields of applications

Sl. No.	Field of application	Useful interpreted information	Helpful in
1.	Irrigation Engineering	Crop area, Crop yield, Crop growth condition, Crop areas that are water stressed and are in need of water	Estimating the amount of irrigation water that is to be supplied to an irrigated area over different seasons
2.	Hydrology	Different types of soils, rocks, forest and vegetation of a water shed, soil moisture	Estimating runoff from a watershed, where the land-cover type and soil moisture would decide the amount that would infiltrate
3.	Reservoir sedimentation	Plan views of reservoir extent at different times of the year and over several years	Estimating the extent of sedimentation of a reservoir by comparing the extent of reservoir surface areas for different storage heights
4.	Flood monitoring	Flood inundated areas	Flood plain mapping and zoning
5.	Water Resources Project Planning	Identification of wasteland (from MSS images), mapping of infrastructure features (from PAN images) like existing roads, embankments, canals, etc. apart from plan view of a river	Recent information helpful in planning and designing of a water resources project based on the present conditions of the project area

6.3.9 Geographic Information System (GIS)

A GIS is a computer application program that stores Spatial and Non-Spatial information in a digital form. Spatial information for an area is what is traditionally represented in maps which for a region, may broadly be classified as given in the following table. The corresponding source of such data for our country is also indicated.

Sl. No.	Spatial features of a region	May be obtained from
1.	• Elevation contours • Drainage • Location of roads, towns, villages	Survey of India, in the form of Topo-Sheets
2.	Soil map	National Burean of Soil Survey and Land Use Planning
3.	Geological map	Geological Survey of India
4.	Latest information on land-use and land-cover, like • Vegetation, forest, crops, etc. • Towns, villages and other human habitation • Roads, Embankments, Canals • Rivers	Satellite imageries
5.	Maps of District, Block, Thana, Mouza, Taluk, etc.	State Land Record office
6.	Location of ground water wells and corresponding water tables as observed over time	Central or State Ground Water Boards

Non-Spatial data, also called Attributes, refer to information like demographic distribution of a town or a village, width or identification tag of a road (like NH-6), daily discharge of a river at a particular place, etc.

Thus, a GIS conveniently manages all variety of data of a given region in a single electronic file in a computer. This is helpful to any regional planner, including that of a Water Resources Project since all information is conveniently stored and accessed with the computer. Further, though the scales of various printed maps may be different, a GIS stores all of them in the same scale. Normally, different spatial features are stored in sub-files, called layers. Hence, one may use the GIS to open all the layers showing all thematic features. Else, one may display one or a few themes at a time by activating the respective layers. For example, the land-use layer may be displayed along with elevation contours, the other layers being kept off.

Important features of GIS software includes handling of spatial and attribute data, data input and editing, data analysis and output of data, which are discussed briefly in the following sections.

A GIS may be considered to comprise of the following components:

• A software package, the components of which include various tools to enter, manipulate, analyse and output data
• A computer system, consisting of the hardware and operating systems.

6.3.10 Handling of spatial and attribute data in GIS

There are two types of data storage structures in a GIS-Raster and vector. According to the Raster system, the space is assumed to be divided into a grid of cells, with a certain value attached to each cell according to the data that is represented by a grid of cells, would be done by marking the corresponding cells black (and assigning a value 1), with all other cells remaining vacant (that is, assigning a value of 0). In the vector system of data storage, the particular point would be stored by the coordinates of the location. This was an example of a point feature. Other types of geographic features include line, area, network of lines and surface, which have been shown in Figure 9.

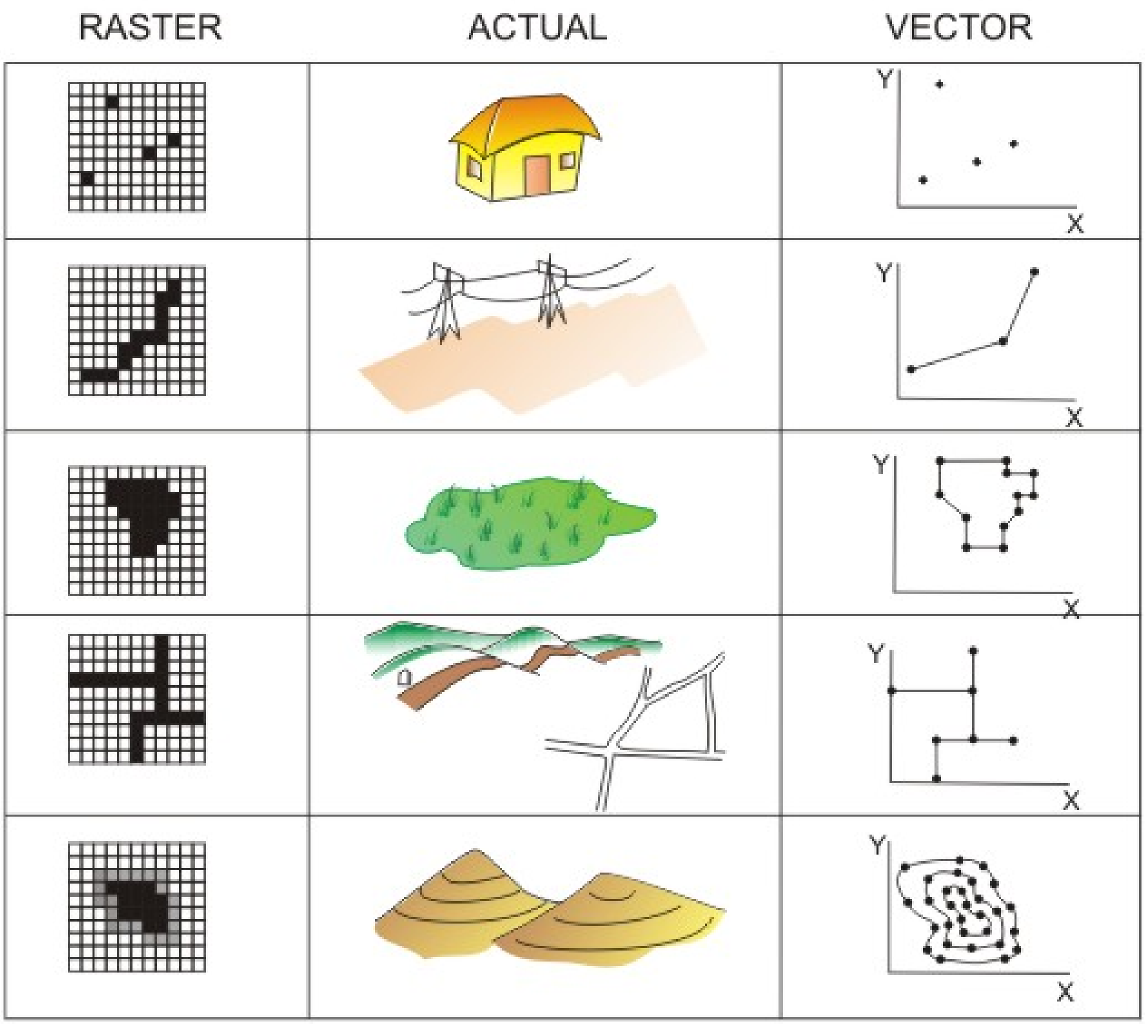


FIGURE 9. Raster and Vector representations of actual objects

It may be noticed that in the raster grid representation of an area, the size of the grids is a choice of the person using the GIS. For example, in representing the spatial information of a town, a grid size of 10 to 25 m may be sufficient but for a state, 100 to 250 m would be enough. Adopting a finer grid size would, naturally, provide a better representation of data. But that would require a higher computer storage space, which therefore has to be judged optimally.

For vector data representation, too, a better resolution of data may be achieved for line features by selecting more number of points closely. This applies also for representing the lines defining the boundary of an area. For surface representing more number of points defining the elevation contours would result in a more precise definition of the region.

Attribute data is non-spatial, that is, it is not something that varies continuously in space. This is actually the database that defines the spatial data. For example, the location of ground water wells is a spatial data, but the water level record or variation of water level with time is an attribute data of the particular well. Similarly, rivers may be represented as a network of lines, but the width and average depth at different points would be represented as attribute data.

6.3.11 Input and editing of data in GIS

The user of a GIS has to input data, whether Spatial (in Raster or Vector formats) or Non-Spatial (usually in the form of tables). Spatial data, as shown in section 6.3.9, may be obtained from different sources and in different formats. They may be input into a GIS in a variety of methods, depending on the format in which they are being supplied. For example, maps would usually be supplied in printed sheets but the satellite image of an area or the land-use map derived from it would be in electronic, that is digital, form. The latter may be directly transferred to a GIS but the former has to be Scanned and then Digitized. Scanning means producing an electronic file of the image, which would usually be a raster representation of the map. This may be done with the help of scanners that are available from sizes of A_4 (the smallest) to A_0 (the largest). The scanned images are not of much use to a GIS since it does not differentiate between the different objects indicated in the image. For this, manual help is required in the form of Digitization, where by a person uses a mouse over the scanned image to physically point to the various features and store them in GIS format. In order to do this, the vectorization tools of the GIS software would be used. Tabular attribute data may be directly transferred to a GIS and attached to the corresponding spatial data with certain tools provided by the GIS software. All the various data are stored in a GIS as layers, or themes.

After data input, the uses might have to edit some of the data to remove duplicacy, redundancy, etc. of some of the vector data or to remove specks or 'noise' in raster data. The errors in the vector data appear while undergoing the process of digitization and therefore, has to be corrected before an analysis with the data is made. For example, while digitizing the boundary of a reservoir from a map, the starting and ending points may not be located right over one another. The GIS would not recognize the outline as a closed boundary, and the reservoir plan cannot be defined as an area. Hence, an editing has to be done to correct this deficiency. Errors in raster data appear due to a variety of reasons, one being the presence of aberration during data capture. For example, LISS-III MSS imagery of an area used to classify land-use may be misrepresented by the presence over some places, unless these are removed, they may lead to false interpretation of land-use classification.

Once spatial and attribute data have been enclosed and edited, it may be necessary to process the different data obtained through various sources in such a way, that all are geometrically compatible.

Some of the mathematical transformations used in this process are:

1. Translation and scaling
2. Creation of a common origin
3. Rotation

6.3.12 Analysis of data in GIS

Once the derived data has been input in a GIS, they are analysed to derive meaningful information. Infact, analysis is essential for any decision-making strategy that may be derived from the stored GIS data. For example, imagine a GIS data that provides locations of ground water wells of a region and their corresponding water levels measured every month. This is overlain with the village boundary data of the region. By plotting the ground water table surface for every month, it may be seen which villages consume more water and when. In this simple example, the analysis is between two different data sets but overlain in the same GIS.

More complicated analysis may be done by interacting more themes or data layers. For example, if it is required to obtain the names of the villages that suffer from excess groundwater depletion in summer and also whose population is more than 10,000 then the population attribute data has to be considered in the query. Another example of using three layers for data analysis includes that of finding the names of the villages that are within 1 km distance from a river and also located at an elevation of 50 m or less. In this case, the river feature has to be 'buffered' with a 1 km zone on either bank and the surface area below 50 m elevation has to be plotted from the digitized contour map. On top of this, the layer representing digitized boundaries of villages has to be overlain to get the desired output.

Though the above examples are only limited to analysis of recorded data, considerable scope lies in the use of GIS data along with mathematical modeling tools that mimic physical processes. For example, watershed runoff model may be conveniently integrated with GIS to provide answers like:

- Which areas of a watershed produce more runoff if a rainfall of a particular intensity is given?
- If the land-use map of the area is overlain on the above is it possible to find out the areas that are prone to excessive soil erosion?

Hence, a GIS database may become extremely useful, if coupled with a modelling software. Much work on similar lines has been done by Prof. Maidment of University of

Texas by integrating GIS with hydrologic process models. Interested users may visit the following site for more information in the following web-site:

http://www.ce.utexas.edu/prof/maidment/GISHYDRO/home.html

6.3.13 Data output in a GIS

The most common form of output from a GIS is a map. In many cases, a thematic map would be that illustrate the spatial variation or pattern in a particular variable. Apart from maps, a GIS output may be in the form of table, like that that showing the names of villages whose groundwater drawdown is more understandable may be output for the decision-makers. For example, the ground water table contour may be output as a three dimensional surface, which may provide a visual guide to the trend of the water table's dip.

6.3.14 Application areas of GIS in Water Resources Engineering

There are many areas in Water Resources Engineering where GIS may be successfully applied. Some examples have been given in this lesson in the previous sections, and some more are illustrated below.

Project planning for a storage structure

In this example, a dam is proposed to be constructed across a river, for which the following information may be desired:

- Watershed area contributing to the project site
- Reservoir surface area and volume, given the height of the dam
- villages that may be inundated under reservoir

For the above, the following themes may be stored in a GIS:

- Elevation contours of the watershed area, including the project site
- Satellite image derived land-use map of the watershed
- Village boundary map, showing location of habitation clusters

Using the above data, one may obtain desired in information as follows:

- Watershed area may be found by using the elevation contour data, and using a suitable GIS software that has a tool to delineate the watershed boundary. Once the boundary is identified, the area calculation tool may be used in the GIS software to calculate the watershed area.

- Reservoir surface area can similarly found using the area calculation tool. Volume calculation tool of the GIS software may be used to find out the storage volume, which is the space between a plane at the reservoir surface and the reservoir bottom.

- By overlying the reservoir extent over the village boundary map and the locations of habitation clusters one may identify the villages that are likely to be inundated once the reservoir comes up. The area of the cultivable village farms that would be submerged may also be similarly identified, as it would be required to pay compensation for the loss to the villagers.

- The amount of forest land that is going to be submerged may be identified by overlaying the reservoir area map over the land use map, for which compensatory afforestation has to be adopted.

Project planning for a diversion structure

Here, a barrage is proposed across a river to divert some of its water through a canal, for which the following information may be desired:

- Location site of the barrage
- Location and alignment of the off taking canal
- Command area that may be irrigated by the canal