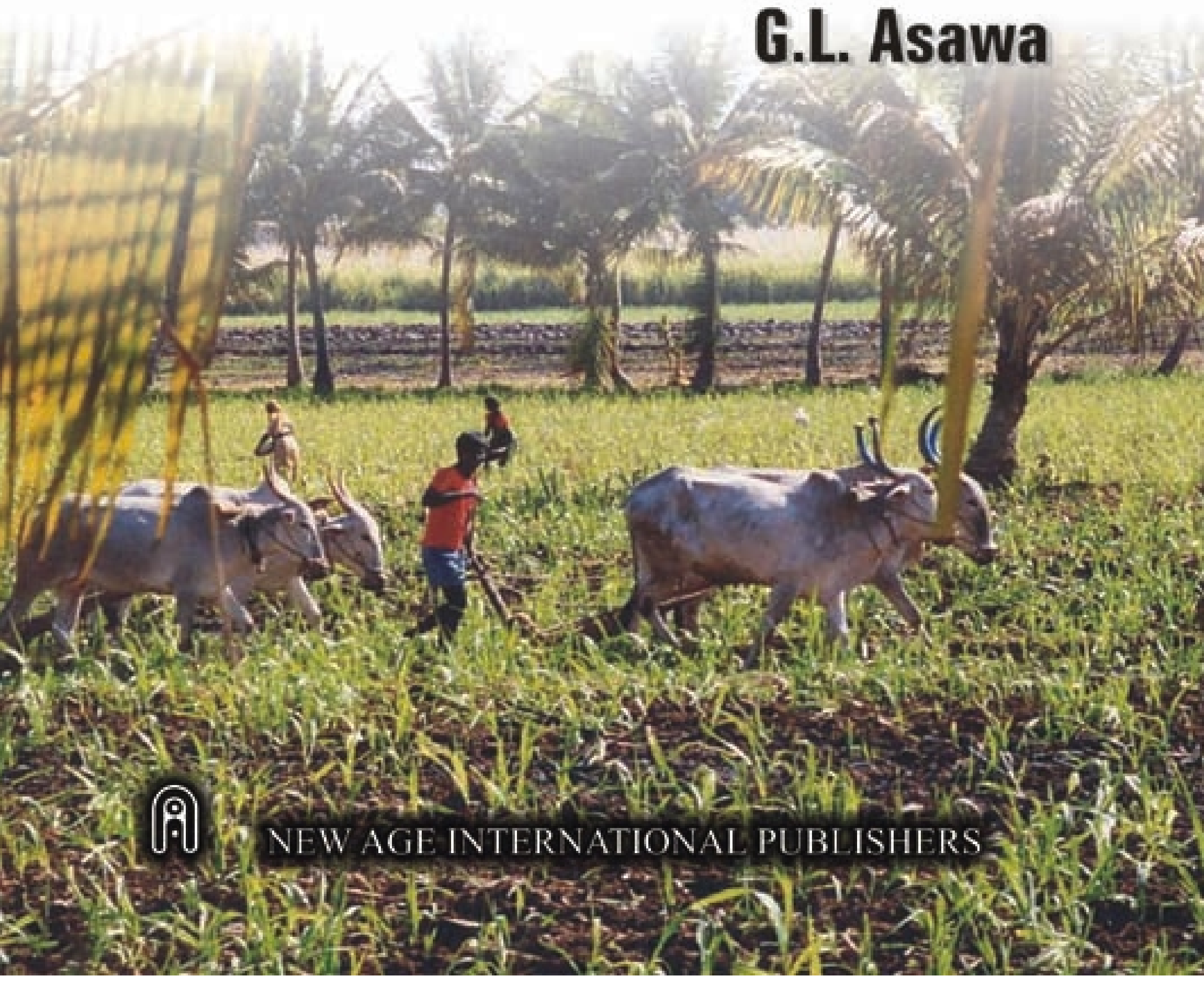
IRRIGATION AND WATER RESOURCES ENGINEERING

G.L. Asawa



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To *My Parents*

(Late) Shri Bhagwan Sahay Asawa

(Late) Smt. Kamla Devi Asawa

Author

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PREFACE

Since the publication of the book "Irrigation Engineering", the author has been receiving suggestions to include a chapter on 'Hydrology' as, at many institutions in the country, Hydrology is being taught as part of the course on either 'Irrigation and Water Resources Engineering' or 'Water Resources Engineering'. While the major contents of this book remain the same as that of the book "Irrigation Engineering", significant additions and revisions have been made in almost all chapters of the book. Besides an additional chapter on Hydrology, other significant additions are : (*i*) detailed environmental aspects for water resource projects, (*ii*) operation, maintenance, and evaluation of canal irrigation systems, (*iii*) note on interlinking of rivers in India, and (*iv*) design problems of hydraulic structures such as guide bunds, settling basin *etc*. Recent developments in Hydraulic Engineering related to Irrigation and Water Resources Engineering have been incorporated in the text. Accordingly, the book "Irrigation Engineering" has now been retitled as "Irrigation and Water Resources Engineering".

It is hoped that the book, in its present form, will be more useful to the undergraduate students of Civil Engineering (and also, to some extent, graduate students of Water Resources Engineering as well), and the practicing engineers dealing with water resources.

The author would like to sincerely appreciate the efforts of the learned authors as well as the publishers of the referenced literature. The author has been immensely benefited by his association with his colleagues at IIT Roorkee (formerly University of Roorkee) who have been associated with the teaching and research in the field of Water Resources Engineering. In particular, however, the author would like to express his sincere thanks to Dr. K.G. Ranga Raju (who reviewed the original manuscript of "Irrigation Engineering") and Dr. B.S. Mathur who reviewed the chapter on "Hydrology".

The forbearance of my wife Savi and sons Anshul and Manish during the period I was busy writing the manuscript of this book is heartily appreciated.

The staffs of the publishers of the book deserve all praise for their nice jobs of printing and publishing the book.

Suggestions from readers of the book are welcome.

G.L. ASAWA

April, 2005

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CONTENTS

1

INTRODUCTION

1.1. IRRIGATION

Three basic requirements of agricultural production are soil, seed, and water. In addition, fertilisers, insecticides, sunshine, suitable atmospheric temperature, and human labour are also needed. Of all these, water appears to be the most important requirement of agricultural production. The application of water to soil is essential for plant growth and it serves the following functions (1):

(*i*) It supplies moisture to the soil essential for the germination of seeds, and chemical and bacterial processes during plant growth.

(*ii*) It cools the soil and the surroundings thus making the environment more favourable for plant growth.

(*iii*) It washes out or dilutes salts in the soil.

(*iv*) It softens clods and thus helps in tillage operations.

(*v*) It enables application of fertilisers.

(*vi*) It reduces the adverse effects of frost on crops.

(*vii*) It ensures crop success against short-duration droughts.

In several parts of the world, the moisture available in the root-zone soil, either from rain or from underground waters, may not be sufficient for the requirements of the plant life. This deficiency may be either for the entire crop season or for only part of the crop season. For optimum plant growth, therefore, it becomes necessary to make up the deficiency by adding water to the root-zone soil. This artificial application of water to land for supplementing the naturally available moisture in the root-zone soil for the purpose of agricultural production is termed *irrigation*.

Irrigation water delivered into the soil is always more than the requirement of the crop for building plant tissues, evaporation, and transpiration. In some cases the soil may be naturally saturated with water or has more water than is required for healthy growth of the plant. This excess water is as harmful to the growth of the plant as lack of water during critical stages of the plant life. This excess water can be naturally disposed of only if the natural drainage facilities exist in or around the irrigated area. In the absence of natural drainage, the excess water has to be removed artificially. The artificial removal of the excess water is termed *drainage* which, in general, is complementary to irrigation.

To keep the optimum content of water in soil, irrigation supplies water to the land where water is deficient and drainage withdraws water from the land where water is in excess. The object of providing irrigation and drainage is to assist nature in maintaining moisture in the root-zone soil within the range required for maximum agricultural production. Usefulness and importance of irrigation can be appreciated by the fact that without irrigation, it would have been impossible for India to have become self-sufficient in food with such huge population to feed. Primary source of prosperity in Punjab is irrigation. Irrigation from the Nile is the source of food, life, and prosperity in Egypt. Similarly, without drainage, large parts of the Netherlands and the coastal regions of several countries would always be under water.

Irrigation schemes can be broadly grouped into two main categories: (*i*) surface water irrigation schemes, and (*ii*) ground water irrigation schemes. The former use diversion and storage methods and obtain their supplies from rivers. Ground water irrigation schemes use open wells, and deep and shallow tube wells to lift water from the water-bearing strata below the earth's surface. The choice of an irrigation scheme depends on several factors, such as surface topography, rainfall characteristics, type of source available, subsoil profile, etc. One should, however, always plan to use surface and ground waters together to derive maximum benefits. Such use is termed *conjunctive use* of surface and ground waters.

In India, the sites for diversion structures without storage potential from major river systems are now difficult to find. Therefore, further use of surface water has to be through storage methods only. India is not blessed with really good storage sites, particularly in the Himalayas, as can be noted from Table 1.1 which gives the storage for some major dams in the Himalayas as well as in other parts of the world.

Table 1.1 Storage of some major dams (2)

Dam	*Height (m)*	*Storage (million cubic metres)*
Bhakra (India)	226	11,320
Kishau (India, projected)	244	1,980
Tehri (India, projected)	260	3,550
Hoover (U.S.A.)	222	38,600
High Aswan (Egypt)	97	156,000

1.2. IMPACT OF IRRIGATION ON HUMAN ENVIRONMENT

The main impact of irrigation is in terms of the increased agricultural yield which, in turn, affects social, cultural, economic, political and other aspects of human environment (Table 1.2)

Table 1.2 Impact of irrigation on human environment (3)

Impact	*Positive*	*Negative*
Engineering	Improvement of the water regime of irrigated soils.	Danger of waterlogging and salination of soils, rise in ground water table.
	Improvement of the micro climate. Possibility provided for waste water use and disposal.	Changing properties of water in reservoirs. Deforestation of area which is to be irrigated and with it a change of the water regime in the area. Reservoir bank abrasion.
	Retention of water in reservoirs and possible multipurpose use thereof.	

(*Contd...*)

Impact	*Positive*	*Negative*
Health	Securing increased agricultural production and thus improving the nutrition of the population. Recreation facilities in irrigation canals and reservoirs.	Possible spread of diseases ensuing from certain types of surface irrigation. Danger of the pollution of water resources by return runoff from irrigation. Possible infection by waste water irrigation, new diseases caused by retention of water in large reservoir.
Social and Cultural	Culturing the area. Increasing the social and cultural level of the population. Tourist interest in the area of the newly-built reservoir.	Colonisation of the irrigated area. Displacement of population from retention area. Necessity of protecting cultural monuments in inundated areas.
Aesthetic	New man-made lakes in the area.	Project's architecture may not blend with the area.
Political	Increased self-sufficiency in food, thus lesser dependence on other countries.	

Dhawan (4) has compared average foodgrain yields from unirrigated and irrigated lands in respect of four states of India (Table 1.3). This comparison clearly shows the marked increase in the average yield on account of irrigation. Irrigation is responsible for about 55 per cent of food production in India (4). Increased agricultural production, besides adding to the national economy, reduces rural poverty and substitutes for imports and generates exports of food and non-food agricultural products. It generates additional employment in the main agricultural activity and also in related activities like input supply, processing, marketing, *etc.*

Besides the gains in agricultural production, there are significant permanent gains in the livelihood of the rural population. These can be grouped into the following four headings (5):

(*i*) Employment and income

(*ii*) Security against impoverishment

(*iii*) Migration

(*iv*) Quality of life

Table 1.3 Comparison of average foodgrain yields for unirrigated and irrigated lands (4)

State	*Period*	*Yield, Tonnes/Hectare*			
		Unirrigated	*Irrigated*		
			Tubewell	*Canal*	*Tank*
Punjab	1977–79	1.08	5.46	3.24	–
	1963–65	0.75	3.06	1.18	–
	1950–51	0.37	1.75	0.94	–
Haryana	1977–79	0.38	5.74	2.39	–

(*Contd...*)

State	*Period*		*Yield, Tonnes/Hectare*		
		Unirrigated		*Irrigated*	
			Tubewell	*Canal*	*Tank*
Andhra Pradesh	1977–79	0.42	5.69	3.43	1.96
	1957–59	0.47	3.11	2.27	1.35
Tamil Nadu	1977–79	0.49	6.53	2.60	2.33
	1964–66	0.61	4.00	2.14	2.08
	1956–58	0.66	3.78	1.69	1.86
Average		**0.58**	**4.35**	**2.21**	**1.92**

Reliable and adequate irrigation is known to raise the employment. According to some field studies, increases in working days per hectare with irrigation compared with rainfed conditions have been 61 per cent (on the Dantiwara canal project in Gujarat) to as high as 150 per cent in Ferozepur, Punjab (4). As a result, production as well as incomes are generally stable at higher levels.

Due to the assured employment and higher incomes spaced over the entire year, there is added security against impoverishment. Therefore, the need for having dependent relationships with moneylenders and employers as well as the dangers of having to dispose of assets like land to buy food or meet debts is much less.

Another beneficial aspect of irrigation is that it stops exodus and attracts people to the region. Therefore, hardships associated with split families are avoided and a more stable and settled family life results.

Irrigation influences the quality of life. One major effect is the increase in prosperity which must improve the nutrition intake and resistance of the people against disease. The prosperity, in its wake, does bring some evils such as dowry, drug habits, etc. But, these evils can be eradicated through education and social welfare programmes. In addition to the above-mentioned gains in agricultural production and livelihood of rural population, irrigation also provides protection against famine and increases the quality of agricultural yields. Other secondary benefits of irrigation projects, such as hydroelectric power generation, use of canals for inland navigation, domestic water supply, and improvement in communication systems also affect the human environment in a favourable manner.

There can, however, be adverse effects too. The adverse effects are mainly in the form of waterborne and water-related diseases and waterlogged saline lands.

1.3 WATER RESOURCES OF INDIA

India, with a geographical area of 329 Mha (million hectares), is blessed with large river basins which have been divided into 12 major (see Table 1.4) and 48 medium river basins comprising 252.8 Mha and 24.9 Mha of total catchment area, respectively (6). It possesses about 4 per cent of the total average annual runoff of the rivers of the world. The per capita water availability of natural runoff is, however, only 2200 cubic metre per year which is about one-third of the per capita water availability in USA and Japan (6). The per capita water availability in India would further decrease with ever-increasing population of the country.

Table 1.4 River basins of India (6)

Sl. No.	Name (Origin)	Length	Catchment Area	Water Resource Potential (in cubic km)		
		(km)	(sq. km)	Average Annual	Utilisable Surface Water	Utilised Surface Water (1989)
	MAJOR BASINS					
1.	Indus (Mansarovar)	1114	321289	73.31	46.00	40.00
2.	(*a*) Ganga (Gangotri)	2525	861452	525.02	250.00	–
	(*b*) Brahmaputra (Kailash)	916	236136	597.04	24.00	–
3.	Sabarmati (Aravalli)	371	21674	4.08	1.93	1.80
4.	Mahi (Dhar, MP)	583	34842	11.83	3.10	2.50
5.	Narmada (Amarkantak, MP)	1312	98796	41.27	34.50	8.00
6.	Tapi (Betul, MP)	724	65145	18.39	14.50	–
7.	Brahmani (Ranchi)	799	39033	36.23	18.30	–
8.	Mahanadi (Nazri, MP)	851	141589	66.88	49.99	17.00
9.	Godavari (Nasik)	1465	312812	118.98	76.30	38.00
10.	Krishna (Mahabaleshwar)	1401	258948	67.79	58.00	47.00
11.	Pennar (Kolar)	597	55213	6.86	6.74	5.00
12.	Cauvery (Coorg)	800	81155	21.36	19.00	18.00
	MEDIUM BASINS	–	248505	289.94	87.65	–
	Grand Total		**2776589**	**1878.98**	**690.01**	

The annual precipitation in the country is estimated at about 4000 cubic km (6). This amount includes snow precipitation as well. As per the assessment of Central Water Commission (CWC), the average annual runoff of various river basins in the country is about 2333 cubic km treating both surface and ground waters as one system. More than eighty (for Himalayan rivers) to ninety (for peninsular rivers) per cent of the annual runoff occurs during monsoon months. Because of this fact and other constraints, it is assessed that the total average annual potential of water available in India is about 1880 cubic km (see Table 1.4) out of which only about 1140 cubic km of water can be put to beneficial use by conventional methods of development of water resources.

The basinwise average annual potential, estimated utilisable surface water, and actual utilised surface water (1989) are shown in Table 1.4. Utilisable ground water is estimated at about 450 cubic km out of which about 385 cubic km is utilisable for irrigation alone (see Table 1.5). The primary uses of water include irrigation, hydro-electric power generation, inland water transport, and domestic and industrial uses including inland fish production. Table 1.6 indicates the amount of utilization of water in 1985 and the projected demands of water for various purposes in the year 2025.

Central Electricity Authority has estimated the hydro-electric potential of the entire country as 84 million KW at 60 per cent load factor from 845 economically feasible schemes in various river basins of the country. The developed hydro-electric potential in the country stands at about 12 million KW at 60 per cent load factor which is about 14.5 per cent of the assessed potential (6). About seven per cent of the assessed potential is under developmental stage; see Table 1.7.

Table 1.5 Ground water potential (annual) in India (6)

Sl. No.	*State*	*Utilisable for Irrigation (cubic km)*	*Net Draft (1989-90) (cubic km)*	*Stage of Development (%)*
1.	Andhra Pradesh	36.86	8.78	23.80
2.	Arunachal Pradesh	1.22	0.00	0.00
3.	Assam	18.42	0.80	4.33
4.	Bihar and Jharkhand	28.43	5.47	19.23
5.	Goa	0.45	0.03	7.71
6.	Gujarat	19.17	7.17	37.40
7.	Haryana	7.25	5.81	80.21
8.	Himachal Pradesh	0.29	0.07	23.55
9.	Jammu & Kashmir	3.74	0.05	1.24
10.	Karnataka	13.76	3.70	26.85
11.	Kerala	6.59	1.01	15.28
12.	M.P. and Chattishgarh	50.76	7.33	14.44
13.	Maharashtra	32.10	7.74	24.11
14.	Manipur	2.68	0.00	0.00
15.	Meghalaya	1.04	0.00	0.00
16.	Mizoram	NA	NA	NA
17.	Nagaland	0.62	0.00	0.00
18.	Orissa	19.79	1.41	7.13
19.	Punjab	16.05	15.76	99.21
20.	Rajasthan	10.80	5.82	53.89
21.	Sikkim	NA	NA	NA
22.	Tamil Nadu	22.43	13.56	60.44
23.	Tripura	2.14	0.10	4.54
24.	U.P. and Uttaranchal	71.25	26.76	37.49
25.	West Bengal	18.74	4.10	21.90
26.	Union Territories	0.53	0.40	74.25
	Grand Total	**385.10**	**115.81**	**30.07**

Table 1.6 Annual requirement of fresh water (6)

Unit: cubic km (0.1 Mha.m)

	Annual Water Requirement for	*1985*		*2025*	
		Surface water	*Ground water*	*Surface water*	*Ground water*
1.	Irrigation	320	150	510	260
2.	Other uses	40	30	190	90
*(*i*)	Domestic and Livestock	16.70		46.00	
*(*ii*)	Industries	10.00		120.00	
*(*iii*)	Thermal Power	2.70		4.00	
*(*iv*)	Miscellaneous	40.60		110.00	
	Subtotal	**360**	**180**	**700**	**350**
	Total	**540**		**1050**	

*Approximate

Table 1.7 Hydro-electric power potential (annual) in India (6)
(at 60% load factor)

Sl. No.	*State*	*Assessed Potential (MW)*	*Developed upto 1993 (MW)*	*Under Development (MW)*	*Stage of Develpment (%)*
1.	Andhra Pradesh	2909	1381.92	34.37	48.69
2.	Arunachal Pradesh	26756	6.17	116.50	0.46
3.	Assam	351	105.00	97.50	57.69
4.	Bihar and Jharkhand	538	99.17	231.78	61.51
5.	Goa	36	0.00	0.00	0.00
6.	Gujarat	409	136.67	112.67	60.96
7.	Haryana	64	51.67	5.00	88.54
8.	Himachal Pradesh	11647	1797.47	633.27	20.87
9.	Jammu & Kashmir	7487	308.33	358.17	8.90
10.	Karnataka	4347	1977.00	652.83	60.50
11.	Kerala	2301	972.33	359.13	57.86
12.	M.P. and Chattishgash	2774	546.00	1248.17	64.68
13.	Maharashtra	2460	1081.00	224.17	53.06
14.	Manipur	1176	73.17	2.00	6.39
15.	Meghalaya	1070	121.67	0.00	11.37
16.	Mizoram	1455	1.00	6.00	0.48

(*Contd...*)

Sl. No.	State	Assessed Potential (MW)	Developed upto 1993 (MW)	Under Development (MW)	Stage of Develpment (%)
17.	Nagaland	1040	0.00	81.88	7.87
18.	Orissa	1983	722.17	386.62	55.91
19.	Punjab	922	481.33	340.00	89.08
20.	Rajasthan	291	188.67	12.00	68.96
21.	Sikkim	1283	25.33	32.17	4.48
22.	Tamil Nadu	1206	942.17	51.83	82.42
23.	Tripura	9	8.50	0.00	94.44
24.	U.P. and Uttaranchal	9744	1127.00	959.83	21.42
25.	West Bengal	1786	21.67	75.33	5.43
	Grand Total	**84044**	**12175.38**	**6021.22**	**21.65**

Total estimated navigable length of inland waterways of the country is 14544 km (see Table 1.8) of which maximum navigable length (2441 km) lies in Uttar Pradesh and Uttaranchal followed by West Bengal with 2337 km. Table 1.9 shows navigable lengths of important river systems of the country.

Table 1.8 Statewise navigable length of inland waters in India (6)

Sl. No.	State	Navigable Waterways (km)		
		Rivers	Canals	Total
1.	Andhra Pradesh	309	1690	1999
2.	Assam	1983	–	1983
3.	Bihar and Jharkhand	937	325	1262
4.	Goa, Daman & Diu	317	25	342
5.	Gujarat	286	–	286
6.	Jammu & Kashmir	200	–	200
7.	Karnataka	284	160	444
8.	Kerala	840	708	1548
9.	Maharashtra	501	–	501
10.	Orissa	761	224	985
11.	Tamil Nadu	–	216	216
12.	U.P. and Uttaranchal	2268	173	2441
13.	West Bengal	1555	782	2337
	Total	**10241**	**4303**	**14544**

Table 1.9 Navigable length (in km) of important river systems in India (6)

Sl. No.	*River System*	*Navigable Length*	
		By Boats	*By Steamers*
1.	Ganga	3355	853
2.	Brahmaputra	1020	747
3.	Rivers of West Bengal	961	784
4.	Rivers of Orissa	438	42
5.	Godavari	3999	–
6.	Krishna	101	–
7.	Narmada	177	48
8.	Tapti	24	24
	Total	**10075**	**2498**

Inland water transport is the cheapest mode of transport for bulk cargo. In view of this, the following ten waterways have been identified for consideration to be declared as National Waterways (6):

(*i*) The Ganga-Bhagirathi-Hoogli
(*ii*) The Brahmaputra
(*iii*) The Mandavi Zuari river and the Cumbarjua canal in Goa.
(*iv*) The Mahanadi
(*v*) The Godavari
(*vi*) The Narmada
(*vii*) The Sunderbans area
(*viii*) The Krishna
(*ix*) The Tapti
(*x*) The West Coast canal

1.4 NEED OF IRRIGATION IN INDIA

The rainfall in India is very erratic in its spatial as well as temporal variations. The average annual rainfall for India has been estimated at 1,143 mm which varies from 11,489 mm around Cherrapunji in Assam (with the maximum one-day rainfall equal to 1040 mm) to 217 mm around Jaisalmer in Rajasthan. Besides, 75% to 90% of the annual rainfall occurs during 25 to 60 rainy days of the four monsoon months from June to September (2). In addition, there is also a large variation from year to year, the coefficient of variation being more than 20% for most parts of the country (2).

Erratic behaviour of the south-west monsoon is the main cause of India's frequent droughts (Table 1.10) and floods. The recent proposal (Appendix–1) of the Government of India on interlinking of some major rivers of the country is aimed at (*i*) increasing the utilizable component of the country's water resources, and (*ii*) solving the problems of shortages and excesses of water in some parts of the country. Table 1.11 shows the values of the approximate probability of deficient rainfall (deficiency equal to or greater than 25 per cent of the normal) for different regions (8). Dependability of rainfall is thus rather low from the agriculture point of view and storage is essential to sustain crops during non-monsoon periods and also to provide water for irrigation during years of low rainfall. For a large part of any crop season, the evapotranspiration (*i.e.,* the water need of a crop) exceeds the available precipitation and irrigation is necessary to increase food and fibre production. About 45 per cent of agricultural production in India is still dependent on natural precipitation. The need and importance of irrigation in India can be appreciated from the mere fact that the country would need to produce 277 million tonnes (against the production of about 185 million tonnes for 1994-95) of food to meet the per capita requirement of 225 kg (*i.e.,* about one-fourth of a tonne) per year for an estimated population of 1,231 million in the year 2030 (8).

Table 1.10 Frequency of droughts in India (7)

Quarter Century	*1801–25*	*1826–50*	*1851–75*	*1876–1900*	*1901–25*	*1926–50*	*1951–75*	*1976–2000*
Drought Years	01,04,06 12,19,25	32,33 37	53,60,62 66,68,73	77,91 99	01,04,05 07,11,13 15,18,20 25	39,41	51,65,66 68,72,74	79,82 85,87
Frequency of drougts	**6**	**3**	**6**	**3**	**10**	**2**	**6**	**4**

Table 1.11 Periodicity of droughts in different regions (8)

Region	*Recurrence of the period of deficient rainfall*
Assam	Very rare, once in 15 years
West Bengal, Madhya Pradesh, Konkan, Coastal Andhra Pradesh, Maharashtra, Kerala, Bihar, Orissa	Once in 5 years
South interior Karnataka, eastern Uttar Pradesh, Vidarbha	Once in 4 years
Gujarat, eastern Rajasthan, western Uttar Pradesh, Tamil Nadu, Kashmir, Rayalaseema, Telengana	Once in 4 years
Western Rajasthan	Once in 2½ years

In addition, the export of agricultural products earns a major part of foreign exchange. Because of vastly different climate in different parts of the country, a variety of crops are produced in India. The country exports *basmati* rice, cotton, fruits (mango, apple, grapes, banana etc.), vegetables (potato, tomato etc.), flowers (rose etc.), and processed food products in order to earn precious foreign exchange. Still further, about seventy per cent of the country's population is employed in agricultural sector and their well-being, therefore, primarily depends on irrigation facilities in the country.

1.5 DEVELOPMENT OF IRRIGATION IN INDIA

Among Asian countries, India has the largest arable land which is close to 40 per cent of Asia's arable land (6). Only USA has more arable land than India. Irrigation has been practised throughout the world since the early days of civilization. In India too, water conservation for irrigation has received much attention since the beginning of civilization. The Grand Anicut across the river Cauvery was built in the second century. At the beginning of the 19th century, there were a large number of water tanks in peninsular India and several inundation canals in northern India. The Upper Ganga canal, the Upper Bari Doab canal and the Krishna and Godavari delta systems were constructed between 1836 and 1866. The famines of 1876–78, 1897–98 and 1899–1900 led to the setting up of the first Irrigation Commission in 1901 to ascertain the usefulness of irrigation as a means of protection against famine and to assess the extent of irrigation development required and the scope for further irrigation work. At this time (1901) the total gross irrigated area was only 13.3 Mha which increased to 22.6 Mha in 1950 as a result of a spurt in protective irrigation schemes (8).

The Bengal famine of 1943 underlined the urgency of increasing agricultural production to meet the needs of the growing population. After independence, the country began an era of planned development starting with the first five-year plan in 1951. The Planning Commission assigned a very high priority to irrigation development for increasing agricultural production. Giant projects like the Bhakra-Nangal, Hirakud, Damodar Valley, Nagarjunasagar, Rajasthan canal, etc. were taken up. This resulted in a great spurt in irrigation development activities and the irrigated area increased from 22.6 Mha in 1950–51 to 68 Mha in 1986–87. In June 1993, the irrigated area was 83.48 Mha *i.e.,* 2.39 Mha more than that in June 1992. The year-wise development of irrigation potential in India since 1950–51 and up to 1994–95 is shown plotted in Fig. 1.1. The present food grain production is slightly more than 200 million tonnes.

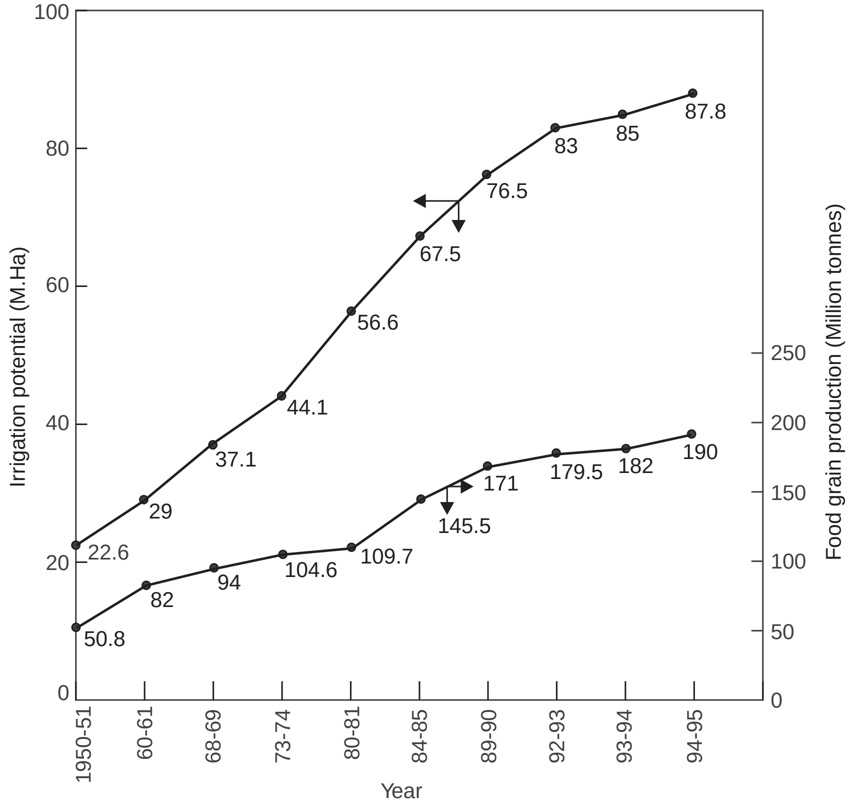

Fig. 1.1 Development of irrigation potential and production of food grains in India

The total ultimate irrigation potential is estimated (6) at 115.54 Mha (see Table 1.12 for statewise distribution) of which 58.47 Mha would be from major and medium irrigation schemes and the remaining from minor irrigation schemes (6).

1.6. MAJOR AND MEDIUM IRRIGATION SCHEMES OF INDIA

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 2,000 and 10,000 ha are classed as medium irrigation schemes (8). The important schemes of the first two plan periods include Bhakra-Nangal, Rajasthan canal, Gandhi Sagar dam, Gandak, Kosi, Nagarjunasagar, Hirakud, Tungabhadra, Malaprabha, and Ghatprabha projects. Later, the multipurpose Beas project, Ramganga dam and canals, Sri Ramsagar, Jayakwadi, Ukai, Kadana, Sardar Sarovar, Tawa, Teesta, etc. were taken up for utilising the monsoon waters.

The performance of major and medium irrigation schemes was examined by the National Irrigation Commission (1972), the National Commission on Agriculture (1976), and several other committees. It was found that the available irrigation potential was not fully utilised. The difference between the available and utilised irrigation potential exceeds 4.0 Mha. Waterlogging and salinity damaged large areas. Moreover, the return in terms of increased agricultural production was far below the expectations. For all these deficiencies, the following causes were identified (8).

(*i*) Need for modernisation of the pre-Plan and early-Plan systems to provide water at the outlet delivery points to farmers at the right time and in the right quantity.

(*ii*) Lack of adequate drainage resulting in waterlogging conditions due to excess water used in irrigating crops as well as due to soil characteristics.

(*iii*) The absence of a distribution system within the outlet and the non-introduction of rotational distribution of water to the farmers.

(*iv*) Inadequate attention to land consolidation, levelling and all other aspects which can promote a better on-farm management of water.

(*v*) Lack of anticipatory research on optimum water use, particularly in black soils with considerable moisture retention capacity.

(*vi*) Lack of suitable infrastructure and extension services.

(*vii*) Poor coordination between the concerned Government organisations in the command areas.

Irrigation projects constructed prior to 1965 were designed to meet the irrigation demand of traditional crops. With the use of high-yielding varieties of seeds since 1965, many of the earlier projects became inadequate to meet the exacting demands for water in respect of high-yielding varieties of crops.

Modernisation of the old irrigation systems listed in Table 1.13 has, therefore, become necessary (8). The weaknesses in the old structures, adequate capacity of the canals to cope with the latest cropping patterns, deficiencies in the control structure system, causes of heavy

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* Gross command area of an irrigation system is the total area which can be economically irrigated from the system without considering the limitations of the quantity of available water. Area of the cultivable land in the gross command of an irrigation system is called the culturable command area (C.C.A.). For more details, see Sec. 5.2.

Table 1.12 State-wise ultimate irrigation potential and potential created and utilised upto 1992–93 (6)
(Unit: Thousand Hectares)

Sl. No.	*State*	*Major and Medium Surface Water*	*Minor Irrigation*		*Total Irrigation Potential*	*Potential Created*		*Potential Utilised*	
			Surface Water	*Ground Water*		*Major and Medium*	*Minor*	*Major and Medium*	*Minor*
1.	Andhra Pradesh	5000	2300	2200	9500	3005.35	2887.94	2849.59	2670.42
2.	Arunachal Pradesh	*e*	150	*e*	150	–	68.09	–	59.04
3.	Assam	970	1000	700	2670	181.55	583.05	114.31	472.40
4.	Bihar and Jharkhand	6500	1900	4000	12400	2770.00	5078.90	2300.00	4488.24
5.	Goa	57	25	*e*	82	13.00	18.89	12.00	16.95
6.	Gujarat	3000	347	1500	4847	1275.18	1921.27	1057.76	1815.75
7.	Haryana	3000	50	1500	4550	2047.00	1534.77	1803.00	1491.92
8.	Himachal Pradesh	50	235	50	335	8.21	144.05	4.20	124.04
9.	Jammu & Kashmir	250	400	150	800	172.10	366.42	140.10	354.51
10.	Karnataka	2500	900	1200	4600	1421.04	1546.56	1208.99	1416.59
11.	Kerala	1000	800	300	2100	428.89	536.13	379.26	500.50
12.	M.P. and Chattishgarh	6000	2200	3000	11200	2032.00	2601.19	1434.50	2408.02
13.	Maharashtra	4100	1200	2000	7300	2076.39	2489.40	1036.00	2236.10
14.	Manipur	135	100	5	240	61.80	51.22	53.15	42.24
15.	Meghalaya	20	85	15	120	–	45.08	–	39.14
16.	Mizoram	*e*	70	*e*	70	–	11.04	–	9.33
17.	Nagaland	10	75	5	90	–	65.90	–	56.41
18.	Orissa	3600	1000	1500	6100	1426.56	1276.50	1349.05	1129.68
19.	Punjab	3000	50	3500	6550	2381.70	3320.00	2323.70	3266.38
20.	Rajasthan	2750	600	2000	5350	2043.36	2425.62	1899.62	2349.88
21.	Sikkim	20	50	2	72	–	22.84	–	17.62
22.	Tamil Nadu	1500	1200	1500	4200	1545.00	2118.97	1544.97	2121.37
23.	Tripura	100	100	15	215	2.00	91.88	2.00	84.05
24.	U.P and Uttaranchal	12500	1200	12000	25700	6860.00	19850.00	5828.00	18304.00
25.	West Bengal	2310	1300	2500	6110	1362.08	2832.37	1268.65	2345.90
26.	Union Territories	98	41	49	188	15.13	82.41	7.13	78.14
	Total	58470	17378	39691	115539	31128.34	51880.49	26615.98	47898.62

e: included under Union Territories.

losses in the irrigation channels, methods to augment canal supplies in tail reaches, and other aspects of modernisation are being looked into for some important projects.

Table 1.13 Old irrigation systems needing modernisation (8)

Sl. No.	*Name of Canal System*	*Year of Construction*	*Area irrigated (10^5 hectares)*
1.	Godavari Delta (Andhra Pradesh)	1890	5.58
2.	Krishna Delta (Andhra Pradesh)	1898	4.42
3.	Sone Canal (Bihar)	1879	3.47
4.	Tribeni Canal (Bihar)	1914	0.48
5.	Western Yamuna Canal (Haryana)	1892	4.81
6.	Ranbir Canal (Jammu and Kashmir)	1904	0.54
7.	Tandula Reservoir (Madhya Pradesh)	1921	0.67
8.	Mahanadi Canal (Madhya Pradesh)	1927	0.85
9.	Nira Left Bank Canal (Maharashtra)	1906	0.48
10.	Godavari Canal (Darna Dam and Nander Weir) (Maharashtra)	1916	0.32
11.	Nira Right Bank Canal (Maharashtra)	1938	0.35
12.	Krishna Raj Sagar Dam and Canals (Karnataka)	1930	0.51
13.	Mahanadi Canals (Orissa)	1895	1.12
14.	Upper Bari Doab Canal System (Punjab)	1879	3.35
15.	Sirhind Canal (Punjab)	1887	6.00
16.	Ganga Canal (Rajasthan)	1928	3.03
17.	Cauvery Delta System (Tamil Nadu)	1889	4.29
18.	Periyar System (Tamil Nadu)	1897	0.62
19.	Upper Ganga Canal (Uttar Pradesh)	1854	6.99
20.	Lower Ganga Canal (Uttar Pradesh)	1878	5.28
21.	Agra Canal (Uttar Pradesh)	1873	1.38
22.	Sarda Canal (Uttar Pradesh)	1926	6.12
23.	Eastern Yamuna Canal (Uttar Pradesh)	1854	1.91
24.	Midnapur Canal (West Bengal)	1889	0.50
25.	Damodar Canal (West Bengal)	1935	0.73

The drainage problem is acute in the states of Punjab and Haryana. It also prevails in the command areas of some of the irrigation projects of UP, West Bengal, Gujarat, Madhya Pradesh, and Maharashtra. Serious waterlogging and consequent salinity problems have arisen in the Chambal Project areas in Madhya Pradesh and Rajasthan, the Kosi and Gandak Project areas in Bihar, Tungabhadra in Karnataka, Nagarjunasagar in Andhra Pradesh, and the Kakrapar system in Gujarat. Inadequate drainage and consequent waterlogging prevails in the Purna, Pravara, and Neera projects in Maharashtra, Dantiwara in Gujarat, Barna and Tawa in Madhya Pradesh, Lower Ganga canal in UP, Malampuzha in Kerala, Periyar-Vaigai in Tamil Nadu, Tungabhadra in Andhra Pradesh and Karnataka, the Sone system in Bihar, Ranbir and Pratap canals in Jammu and Kashmir, Hirakud and Mahanadi Delta in Orissa, and Krishna Delta in Andhra Pradesh. According to the estimates of the National Commission on Agriculture (1976), a total area of about 6 Mha is waterlogged (8).

1.7. MINOR IRRIGATION

Minor irrigation schemes include all ground water and surface water irrigation (flow as well as lift) projects having culturable command area up to 2000 ha. Minor surface water flow irrigation projects include storage and diversion works and are the only means of irrigation in several drought-prone tracts such as undulating areas south of the Vindhyas and also hilly regions. Such projects offer considerable opportunity for rural employment and also help in recharging the meagre resources of ground water in the 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.

Ground water is widely distributed and provides an instant and assured source of irrigation to farmers. It improves the status of irrigation supply and helps in controlling waterlogging and salinisation in the command area of a canal. Ground water development is the major activity of the minor irrigation programme. It is mainly a cultivator's own programme implemented primarily through individual and cooperative efforts. Finance for such programmes are arranged through institutional sources. The first large-scale venture in scientific planning and development of ground water was initiated in India in 1934 when a project for the construction of about 1,500 tubewells in the Indo-Gangetic plains in the Meerut region of Uttar Pradesh was undertaken. 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. Besides electricity and diesel, biogas-operated pumps need to be popularised. The use of solar energy through photovoltaic systems will, probably, be the ultimate solution to the energy problem. Wind energy should also be tapped in desert, coastal and hilly regions.

1.8. COMMAND AREA DEVELOPMENT

The irrigation potential created by the construction of a large number of major and medium irrigation projects has more than doubled since independence. However, the available irrigation potential has always been under-utilized and the optimum benefits by way of increased production have not been fully realised. Several studies have been made to analyse the reasons for inefficient and continued under-utilisation of available irrigation potential and unsatisfactory increase in agricultural production in irrigated areas. The Second Irrigation Commission and the National Commission on Agriculture recommended an integrated command area development programme for optimising benefits from available irrigation potential. The objectives of the programme were as follows (9):

(*i*) Increasing the area of irrigated land by proper land development and water management.

(*ii*) Optimising yields by adopting the best cropping pattern consistent with the availability of water, soil, and other local conditions.

(*iii*) Bringing water to the farmer's field rather than only to the outlets and thus assuring equitable distribution of water and adequate supply to tailenders.

(*iv*) Avoiding wastage and misuse of water.

(*v*) Optimising the use of scarce land and water resources, including ground water where available, in conjunction with necessary inputs and infrastructure.

The command area development programme is a series of coordinated measures for optimising the benefits from the irrigated agriculture. Some of these measures are (8):

(*i*) Scientific crop planning suited to local soil and climatic conditions.

(*ii*) Consolidation of holdings and levelling/shaping of lands.

(*iii*) Provision of field channels to ensure equitable distribution of water to the farmer's field.

(*iv*) Ensuring the supply of other inputs (good quality seeds, fertilisers, etc.).

(*v*) Construction of rural roads, markets, storages, and other infrastructural facilities in the command areas of irrigation projects.

The Second Irrigation Commission (10) stressed the need for a programme of integrated command area development involving cooperative efforts among the State Departments of Irrigation, Agriculture, Animal Husbandry, Community Development, Finance and Public Works and other institutions like Agriculture Refinance Corporation, Land Development Banks, Commercial Banks, etc. The commission suggested the formation of a special administrative agency for the coordinated and expeditious development of command areas under major and medium irrigation projects. The functions of such agencies would be to assign tasks to various departments and institutional organisations to enforce coordination among them and to ensure the implementation of the agreed programme. As a result, in 1973, State Governments were requested to set up Command Area Development Authorities for 50 irrigation projects in the country.

In December 1974, the formation of a Central Sector Scheme for Command Area Development Programme in selected irrigated commands was approved by the Government of India. This programme included the following components (8) :

(*i*) Modernisation and efficient operation of the irrigation system.

(*ii*) Development of a main drainage system beyond the farmer's blocks of 40 ha.

(*iii*) Construction of field channels and field drains.

(*iv*) Land shaping/levelling and consolidation of holdings.

(*v*) Lining of field channels/watercourses.

(*vi*) Exploitation of ground water and installation of tubewells.

(*vii*) Adoption and enforcement of a suitable cropping pattern.

(*viii*) Enforcement of an appropriate rostering system on irrigation.

(*ix*) Preparation of a plan for the supply of key inputs like credit, seeds, fertilisers, pesticides, and implements.

(*x*) Making arrangements for timely and adequate supply of various inputs.

(*xi*) Strengthening of existing extension, training, and demonstration organisations.

The National Commission on Agriculture emphasised the need for development of land in the command area in an integrated manner comprising the following actions (8):

(*i*) Layout of plots and of common facilities like watercourses, field channels, drains and farm roads.

(*ii*) Consolidation of farmers' scattered plots into one or two operational holdings.

(*iii*) Construction of watercourses and field channels.

(*iv*) Construction of field drains where necessary and linking them with connecting drains.

(*v*) Provision of farm roads.

(*vi*) Land formation to suitable slopes.

(*vii*) Introduction of the '*warabandi*' system for rotational distribution of water.

For future irrigation projects, the National Commission on Agriculture suggested that the project report should be prepared in the following three parts (8):

Part I: All engineering works from source of supply to outlets, including drains.

Part II: All engineering works in the command area comprising land levelling and shaping, construction of watercourses, lined or unlined field channels, field drains, and farm roads.

Part II: All other items pertaining to agriculture, animal husbandry, forestry, communications, and cooperation.

Upto 1985, 102 projects covering an ultimate irrigation potential of 16.5 Mha has been included in Command Area Development Authorities (CADA) (11). In financial terms, the allocation in the sixth and the seventh five-year plans was Rs. 856 crores and Rs. 1,800 crores, respectively (5).

1.9. PLANNING OF IRRIGATION PROJECTS

Agricultural establishments capable of applying controlled amounts of water to lands to produce crops are termed *irrigation projects.* These projects mainly consist of engineering (or hydraulic) structures which collect, convey, and deliver water to areas on which crops are grown. Irrigation projects may range from a small farm unit to those serving extensive areas of millions of hectares. A small irrigation project may consist of a low diversion weir or an inexpensive pumping plant along with small ditches (channels) and some minor control structures. A large irrigation project includes a large storage reservoir, a huge dam, hundreds of kilometres of canals, branches and distributaries, control structures, and other works. Assuming all other factors (such as enlightened and experienced farmers, availability of good seeds, *etc*.) reasonably favourable, the following can be listed as conditions essential for the success of any irrigation project.

(*i*) Suitability of land (with respect to its soil, topography and drainage features) for continued agricultural production,

(*ii*) Favourable climatic conditions for proper growth and yield of the crops,

(*iii*) Adequate and economic supply of suitable quality of water, and

(*iv*) Good site conditions for the safe construction and uninterrupted operations of the engineering works.

During the last four decades, many large irrigation projects have been built as multipurpose projects. Such projects serve more than one purpose of irrigation or power generation. In India, such large projects (single-purpose or multipurpose) are constructed and administered by governmental agencies only. Most of the irrigation projects divert stream flow into a canal system which carries water to the cropland by gravity and, hence, are called gravity projects. In pumping projects, water is obtained by pumping but delivered through a gravity system.

A gravity type irrigation project mainly includes the following works:

(*i*) Storage (or intake) and diversion works,

(*ii*) Conveyance and distribution channels, (*iii*) Conveyance, control, and other hydraulic structures,

(*iv*) Farm distribution, and

(*v*) Drainage works.

1.9.1. Development of an Irrigation Project

A small irrigation project can be developed in a relatively short time. Farmers having land suitable for agriculture and a source of adequate water supply can plan their own irrigation system, secure necessary finance from banks or other agencies, and get the engineering works constructed without any delay. On the other hand, development of a large irrigation project is more complicated and time-consuming. Complexity and the time required for completion of a large project increase with the size of the project. This is due to the organisational, legal, financial administrative, environmental, and engineering problems all of which must be given detailed consideration prior to the construction of the irrigation works. The principal stages of a large irrigation project are: (*i*) the promotional stage, (*ii*) the planning stage, (*iii*) the construction stage, and (*iv*) the settlement stage. The planning stage itself consists of three substages: (*i*) preliminary planning including feasibility studies, (*ii*) detailed planning of water and land use, and (*iii*) the design of irrigation structures and canals. Engineering activities are needed during all stages (including operation and maintenance) of development of an irrigation project. However, the planning and construction stages require most intensive engineering activities. A large irrigation project may take 10–30 years for completion depending upon the size of the project.

1.9.2. Feasibility of an Irrigation Project

A proposed irrigation project is considered feasible only when the total estimated benefits of the project exceed its total estimated cost. However, from the farmer's viewpoint, an irrigation project is feasible only if his annual returns (after completion of the project) exceed his annual costs by sufficient amount. The feasibility of an irrigation project is determined on the basis of preliminary estimates of area of land suitable for irrigation, water requirements, available water supplies, productivity of irrigated land, and required engineering works.

1.9.3. Planning of an Irrigation Project

Once the project is considered feasible, the process of planning starts. Sufficient planning of all aspects (organisational, technical, agricultural, legal, environmental, and financial) is essential in all irrigation projects. The process of planning of an irrigation project can be divided into the following two stages:

(*i*) Preliminary planning, and

(*ii*) Detailed planning.

Preliminary plans, based on available information, are generally approximate but set the course for detailed planning. Based on preliminary planning, the detailed measurements are taken and the detailed plans are prepared. Obviously, detailed plans are more accurate. Alterations in the detailed plans may be necessary at all stages of the project. The preparation of plans of an irrigation project in an undeveloped region is a complicated task and needs the expertise of specialists in areas of engineering, agriculture, soil science, and geology. The following are the main factors which must be determined accurately during the planning stage of an irrigation project:

(*i*) Type of project and general plan of irrigation works, (*ii*) Location, extent and type of irrigable lands,

(*iii*) Irrigation requirements for profitable crop production,

(*iv*) Available water supplies for the project,

(*v*) Irrigable (culturable) areas which can be economically supplied with water,

(*vi*) Types and locations of necessary engineering works,

(*vii*) Needs for immediate and future drainage,

(*viii*) Feasibility of hydroelectric power development,

(*ix*) Cost of storage, irrigation, power, and drainage features,

(*x*) Evaluation of probable power, income, and indirect benefits,

(*xi*) Method of financing the project construction,

(*xii*) Desirable type of construction and development,

(*xiii*) Probable annual cost of water to the farmers,

(*xiv*) Cost of land preparations and farm distribution systems, and

(*xv*) Feasible crops, costs of crop production, and probable crop returns.

Most of these elements of project planning are interrelated to some extent. Hence, the studies of the factors listed above should be carried out concurrently so that necessary adjustments can be made promptly as planning progresses.

The preliminary planning of an irrigation project consists of collecting and analysing all available data for the current study, securing additional data needed for preparing preliminary plans for major project features by limited field surveys, and determining the feasibility of the proposed development by making the preliminary study of major features in sufficient detail. While investigations for the preliminary planning of irrigation projects should be conducted with minimum expenditure, the results of the preliminary study must be sufficiently accurate. For preliminary investigations, hydrological studies can be based on the records of stations in the vicinity of the proposed project site. Suitability of land for cultivation purposes can be examined at representative sample areas. Foundation conditions at major irrigation works can be determined from surface and a few subsurface explorations. For detailed planning, accurate data on all aspects of the proposed irrigation project are required to work out the detailed plans and designs of various engineering works and to determine their economic site locations. Physical data needed for detailed planning are collected by topographic and location surveys, land and soil investigations and geological explorations (surface as well as subsurface) at the sites of major engineering works. Results of such surveys are suitably tabulated or plotted for convenient use in design offices and for planning further field work, if necessary. Hydrological data are usually determined by extensive studies of all available records and collecting additional data, if possible. Photographic records of pre-construction (and also during construction) condition at locations of all engineering works and aerial surveys for dams and reservoir sites must be supplemented by accurate ground surveys. Geological explorations are also needed at the sites of dams, reservoirs, and major structures. Such data are useful in studies of water loss due to leakage and foundation designs. Sources of suitable amounts of building material (such as earth material, concrete aggregates, *etc*.) must be located and explored. In case of insufficient supplies at the site, additional sources must be located.

Having collected the required data for detailed planning, general plans for irrigation structures are prepared. Such plans are dependent on topography, locations of irrigable areas, available water sources, storage requirements and construction costs. There can be different types of possible feasible plans for a particular project. Advantages and disadvantages of all such possible alternatives must be looked into before arriving at the final plan for the project.

Possibilities of using irrigation structures (dams and canal falls) for the development of hydroelectric power should also be examined in project planning.

1.9.4. Environmental Check-List for Irrigation and Water Resource Projects

The term environment includes the earth resources of land, water, air, vegetation, and man-made structures. The relationship between organisms (*i.e.,* plant, animal and human) and their environment is termed ecology. All water resource projects, whether for irrigation or for hydro-electric power or for flood control or for water supply, are constructed for the well-being of human beings and have definite impact on the surrounding ecosystems and environment. It is, however, unfortunate that some of the environmentalists get unreasonably influenced by the subtle propaganda against the development of water resources in India by the people of the developed nations who would not like the people of India to be able to reach near the level of living style of the people of the developed countries. These people oppose development of water resources in India on environmental considerations without appreciating the needs of India and the fact that India has not utilised even 50 per cent of its utilisable potential. As a result, the per capita consumption of electric power and all other human needs is much lower than that in the developed countries. Region-wise, India is already a water-short country and faces acute water problems in almost the entire country. This will continue to be so till India controls the increase in its population and harnesses its entire monsoon and redistributes it spatially and temporally. The mooted proposal of interlinking of rivers in the country (Appendix–1) envisages inter-basin transfers of surplus water to meet the water needs of the water-short regions of the country. Such developmental works do cost a fortune in terms of money and environmental impacts. However, if the benefits (monetary as well as environmental) exceed the cost (both monetary and environmental), the work should be considered justifiable. The decision of water resources development should be based upon analysing the future scenario 'with' and 'without' the proposed development. Therefore, the developmental activities cannot be stopped on environmental considerations alone. It should, however, be appreciated that both developmental activities and an intact environment are equally important for sustained well-being of human beings.Therefore, the water resources projects must be developed such that they minimise environmental disturbances and maintain ecological balance while meeting the demands of man.

The complexity of environmental processes seldom permits accurate prediction of the full spectrum of changes in the environment brought about by any particular human activity. Many countries, including India, have now made it a statutory requirement for environmental impact assessment (EIA) of all new projects within specified category. Water resources projects are included in this category and are approved only after favourable report of EIA studies. The statutory EIA authorities usually concentrate on negative aspects of environmental changes. This results in conflict between the EIA authorities and project planners. Since EIA requires detailed information, it is usually undertaken at the final stage of the project planning when changes in the project to mitigate adverse effects on environment are difficult and costly.

The environmental check-list (Table 1.14) prepared by the Environmental Impacts Working Group of International Commission on Irrigation and Drainage provides a comprehensive guide to the areas of environmental concern which should be considered in the planning, design, operation, and management of irrigation, drainage, and flood control projects (12). This check-list provides a tool which will enable planners concerned with irrigation and drainage development to appreciate the environmental changes which such projects may bring about so that adverse effects can be identified and, if possible, avoided or, at least, controlled and positive effects enhanced. Details of the parameters of ICID check-list (Table 1.14) are as explained in the following paragraphs (12) :

Table 1.14 Results sheet for assessing the ICID check-list (12)

Project name/location: .... *Assessment: 1st/2nd/* ....

Assessor's name/position: .... *Date:* ....

For each environmental effects, place a cross (X) in one of the columns			*Positive impact very likely*	*Positive impact possible*	*No impact likely*	*Negative impact possible*	*Negative impact very likely*	*No judgment possible at present*	*Comments*
			A	*B*	*C*	*D*	*E*	*F*	
Hydrological change	1.1	Low flow regime							
	1.2	Flood regime							
	1.3	Operation of dams							
	1.4	Fall of water table							
	1.5	Rise of water table							
Organic and inorganic pollution	2.1	Solute dispersion							
	2.2	Toxic substances							
	2.3	Organic pollution							
	2.4	Anaerobic effects							
	2.5	Gas emissions							
Soil properties and salinity effects	3.1	Soil salinity							
	3.2	Soil properties							
	3.3	Saline groundwater							
	3.4	Saline drainage							
	3.5	Saline intrusion							
Erosion and sedimentation	4.1	Local erosion							
	4.2	Hinterland effect							
	4.3	River morphology							
	4.4	Channel regime							
	4.5	Sedimentation							
	4.6	Estuary erosion							

(Contd....)

Biological and ecological changes	5.1	Project lands							
	5.2	Water bodies							
	5.3	Surrounding area							
	5.4	Valleys and shores							
	5.5	Wetlands and plains							
	5.6	Rare species							
	5.7	Animal migration							
	5.8	Natural industry							
Socio-economic aspects	6.1	Population change							
	6.2	Income and amenity							
	6.3	Human migration							
	6.4	Resettlement							
	6.5	Women's role							
	6.6	Minority groups							
	6.7	Sites of value							
	6.8	Regional effects							
	6.9	User involvement							
	6.10	Recreation							
Human health	7.1	Water and sanitation							
	7.2	Habitation							
	7.3	Health services							
	7.4	Nutrition							
	7.5	Relocation effect							
	7.6	Disease ecology							
	7.7	Disease hosts							
	7.8	Disease control							
	7.9	Other hazards							
Ecological imbalances	8.1	Pests and weeds							
	8.2	Animal diseases							
	8.3	Aquatic weeds							
	8.4	Structural damage							
	8.5	Animal imbalances							
		Number of crosses						(Total = 53)	

1.9.4.1. Hydrological Change

1.1 *Low flow regime.* Is the low flow regime of the river substantially changed by the Project and its dams (by more than ± 20% in low flow periods)? If so, does this change benefit or impair aquatic ecosystems, existing or potential downstream abstractions, hydropower navigation or recreational uses?

1.2 *Flood regime.* Is the flood regime of the river (peak discharge and stage, speed of flood waves, flood super-position with joining rivers, duration or extent of floodplain inundations downstream) substantially changed by the Project as well as the result of changes in abstractions, retention storage, reservoir releases, flood protection works, new road/rail routes, river training or surface drainage works? If so, does this change benefit or impair aquatic and flood-affected ecosystems, lead to an increase or decrease in flood damage or change land use restriction outside the project?

1.3 *Operation of dams.* Can modifications to the operation of any storage or flood retention reservoir(s) compensate for any adverse impacts associated with changes in flow regime, whilst minimising the losses to the Project and other users? Possible modifications affecting water quality downstream, saline intrusion, the sediment regime of channels, the ecology of affected area, amenity values, disease transmission or aquatic weed growth should be considered. (A separate environmental assessment of large reservoir(s) may be required).

1.4 *Fall of water table.* Does the Project cause a fall of the water table (from groundwater abstractions, reduced infiltration due to river training, drainage or flood protection works)? If so, does this fall lead to increased potential for groundwater recharge (from seasonal rainfall) and improved conditions for land use; or lead to depletion of the groundwater system, affecting wells, springs, river flows and wetlands?

1.5 *Rise of water table.* Does the Project cause a rise of the water table (from increased infiltration or seepage from irrigation, seepage from reservoirs and canals or increased floodplain inundation)? If so, does this rise lead to improved yield of wells and springs and improved capillary rise into the root zone; or lead to waterlogging of agricultural or other land in the Project area or vicinity?

1.9.4.2. Organic and Inorganic Pollution

2.1 *Solute dispersion.* Are the Project and its dams leading to changes in the concentrations of organic or inorganic solutes in the surface water due to changes to the pattern of water abstraction and reuse in the basin or flow regulation? If so, do the changes benefit or impair biological communities or domestic, agricultural or industrial water users in the basin?

2.2 *Toxic Substances.* Are significant levels of toxic substances accumulating or being introduced, mobilised and transmitted due to the construction and operation of the Project and its dams, or are levels being reduced? Substances such as pesticides, herbicides, hydrogen sulphide, oil derivatives, boron, selenium and heavy metals in irrigation supplies or surface, drainage and ground waters should be considered.

2.3 *Organic pollution.* Are nutrients, organic compounds and pathogens being reduced or introduced and concentrated, due to the Project, its dams and its associated domestic settlements? If so, does the change result in a reduction or increase in environmental and water use problems in the Project area or downstream (in rivers, canals, reservoirs, end lakes, evaporation wet lands, depressions, deltas, estuary regions) or in the groundwater?

2.4 *Anaerobic effects.* Is the Project reducing or creating anaerobic conditions or eutrophication in any impoundments, natural lakes, pools or wetlands due to changed input or accumulation of fertilisers, other nutrients and organic matter or due to changed water quality resulting from dams, river abstractions and drainage flows?

2.5 *Gas Emissions.* Is the Project, either directly or through associated industrial processing, causing decreased or increased gas emissions which contribute to air pollution (O_3, SO_3, H_2S, NO_x, NH_4, etc.) or the greenhouse effect (CO_2, CH_4, NO_x, etc.)?

1.9.4.3. Soil Properties and Salinity Effects

3.1 *Soil salinity*. Is the Project leading to progressive accumulation of salts in the soils of the project area or the vicinity because of prevailing high salt content in, the soil, the groundwater, or the surface water; or can a progressive leaching effect be expected?

3.2 *Soil properties.* Is the project leading to changes in soil characteristics within the Project area or the vicinity due to such activities as irrigation, the application of fertilisers or other chemicals, cultivation practices or dewatering through drainage? Changes which can improve or impair soil structure, workability, permeability, fertility associated with nutrient changes, humus content, pH, acid sulphate or hard pan formation or available water capacity should be considered.

3.3 *Saline groundwater*. Are changes to the rates of seepage, percolation or leaching from the Project and its dams increasing or decreasing the concentrations of chlorides, nitrates or other salts in the groundwater?

3.4 *Saline drainage*. Are changes to the concentrations of chlorides, nitrates or other salts in the runoff or drainage water from the Project area in danger of affecting biological communities or existing or potential downstream users (particularly during low flow conditions)?

3.5 *Saline intrusion*. Are the Project and its dams leading to changes in saline water (sea water) intrusion into the estuary or into groundwater due to changes in low flow, groundwater use, dredging or river training? If so, are the changes likely to affect biological communities and water users in the Project vicinity and other areas?

1.9.4.4. Erosion and Sedimentation

4.1 *Local erosion.* Is increased or decreased soil loss or gully erosion being caused within or close to the Project area by changes in land gradient and vegetative cover, by irrigation and cultivation practice, from banks of canals, roads and dams, from areas of cut and fill or due to storm drainage provision?

4.2 *Hinterland effect.* Are the Project and its dams leading to changes in natural vegetation, land productivity and erosion through changes in population density, animal husbandry, dryland farming practices, forest cover, soil conservation measures, infrastructure development and economic activities in the upper catchment and in the region surrounding the Project?

4.3 *River morphology.* Is the regime of the river(s) changed by the Project and its dams through changes in the quantity or seasonal distribution of flows and flood peaks in the river(s), the abstraction of clear water, changes in sediment yield (caused by 4.1 and 4.2), the trapping of sediment in reservoirs or the flushing of sediment control structures? If so, do these changes benefit or impair aquatic ecosystems or existing or potential users downstream?

4.4 *Channel structures.* Is scouring, aggradation or bank erosion in the river(s) endangering the Project's river headworks, offtake structures, weirs or pump inlets, its canal network, drainage or flood protection works, the free flow of its drainage system or structures and developments downstream? Consider effects associated with change noted in 4.3 as well as those caused by other existing and planned upstream developments.

4.5 *Sedimentation.* Are the changes noted in 4.1-4.4 causing increased or decreased sediment deposition in irrigation or drainage canals, hydraulic structures, storage reservoirs or on cultivated land, either via the irrigation system or the river(s)? If so, do these changes benefit or impair soil fertility, Project operation, land cultivation or the capacity and operation of reservoirs?

4.6 *Estuary erosion.* Are the Project and its dams leading to changes in the hydrological or sediment regimes of the river which can affect delta formation or estuary and coastal erosion? If so, do these changes benefit or impair aquatic ecosystems (estuarine or marine), local habitation, navigation or other uses of the estuary?

1.9.4.5. Biological and Ecological Changes

Is the Project, its dams or its associated infrastructure causing substantial and permanent changes (positive or negative) within the habitats listed in 5.1-5.5?

- in the natural ecology (habitat, vegetation, terrestrial animals, birds, fish and other aquatic animals and plants),
- in areas of special scientific interest, or
- in biological diversity

Include the likely ecological benefit of any new or modified habitats created and of any protective or mitigatory measures adopted (such as nature reserves and compensatory forests).

5.1 *Project lands*. The lands within the project area.

5.2 *Water bodies*. Newly created, altered or natural channels, reservoirs, lakes and rivers.

5.3 *Surrounding area*. All terrestrial areas influenced by the Project works and its associated domestic settlements and hinterland effects.

5.4 *Valleys and shores*. River and canal banks, lake, reservoir and sea shores and the offshore marine environment.

5.5 *Wetlands and plains*. Floodplains or permanent wetlands including deltas and coastal swamps.

5.6 *Rare species*. Is the existence of any rare, endangered or protected species in the region enhanced or threatened by the changes noted in 5.1-5.5?

5.7 *Animal migration*. Does the Project, its dams or new road/rail routes affect the migration patterns of wild animals, birds or fish? Make allowance for the compensatory effect or any additional provision within the Project (canal crossings, fish passes, spawning locations, resting or watering places, shade, considerate operation).

5.8 *Natural industry*. Are commercial or subsistence activities depending on the natural terrestrial and aquatic environment benefited or adversely affected by the Project through ecological changes or changes in human access? Changes affecting such activities as fisheries, harvesting from natural vegetation, timber, game hunting or viewing and honey production should be considered.

1.9.4.6. Socio-Economic Impacts

6.1 *Population change*. Is the Project causing significant demographic changes in the Project area or vicinity which may affect social harmony? Changes to population size/density and demographic/ethnic composition should be considered.

6.2 *Income and amenity*. Is the Project introducing significant economic/political changes which can increase or decrease social harmony and individual well-being? Changes in the general levels of employment and income, in the provision of local infrastructure and amenities, in the relative distribution of income, property values and Project benefits (including access to irrigation water) and in the demand for labour and skills (particularly in relation to family/political hierarchy and different sexes and social groups) should be considered.

6.3 *Human migration*. Has adequate provision been made for any temporary or migratory population influx to avoid social deprivation, hardship or conflicts within these groups or between the permanent and temporary groups? Human migration arising both from the demand for skills/labour during construction and from the requirements for seasonal agricultural labour should be considered.

6.4 *Resettlement*. Has adequate provision been made for the resettlement, livelihood and integration of any people displaced by the Project and its dams or losing land, grazing or other means of income due to the Project? Also, has adequate provision been made for the subsistence farming needs of people settled on or associated with the Project?

6.5 *Women's role*. Does the Project change the status and role of women (positively or negatively) in relation to social standing, work load, access to income and heritage and material rights?

6.6 *Minority groups*. Are the Project and its dams causing changes to the lifestyle, livelihoods or habitation of any social groups (particularly minority groups) leading to major conflicts with, or changes to their traditional behaviour, social organisation or cultural and religious practices?

6.7 *Sites of value*. Is access improved or hampered to places of aesthetic and scenic beauty, sites of historical and religious significance or mineral and palaeontological resources? Also, are any such sites being destroyed by the Project?

6.8 *Regional effects*. Are the economic, infrastructural, social and demographic changes associated with the Project likely to enhance, restrict or lead to unbalanced regional development? Also, has adequate provision been made for new transport, marketing and processing needs associated with the Project?

6.9 *User involvement*. Has there been adequate user and public participation in project planning, implementation and operation to ensure Project success and reduce future conflicts? The potential for incorporating within the Project existing systems of land tenure, traditional irrigation, and existing organisational and sociological structures and for the provision of new or extended facilities for credit, marketing, agricultural extension and training should be considered.

6.10 *Recreation*. Are the Project and its dams creating new recreational possibilities (fishing, hunting, sailing, canoeing, swimming, scenic walks, *etc*.) and are existing facilities impaired, preserved or improved?

1.9.4.7. Human Health

Consider each of the items 7.1-7.9 in relation to the local population, the labour force during construction and their camp followers, the resettled and newly settled populations and migratory labour groups.

7.1 *Water and Sanitation*. Are the provisions for domestic water, sanitation and refuse disposal such that oral, faecal, water washed and other diseases and the pollution of domestic water can be controlled?

7.2 *Habitation.* Are the provisions for housing and forecast population densities such that diseases related to habitation or location of dwellings can be controlled?

7.3 *Health services*. Are general health provisions adequate (treatment, vaccination, health education, family planning and other health facilities)?

7.4 *Nutrition*. Is the Project leading to an increase or decrease in the general nutritional status of the population or to changes in other lifestyle or income related disease? If so, are any specific groups particularly exposed to such health risks?

7.5 *Relocation effect*. Are population movements introducing new infectious or water-related diseases to the Project area or causing stress-related health problems or bringing people with a low resistance to particular diseases into areas of high transmission?

7.6 *Disease ecology*. Are the extent and seasonal character of reservoirs, canals, drains, fast flowing water, paddy fields, flooded areas or swamps and the closeness or contact of the population with such water bodies leading to significant changes in the transmission of water related diseases?

7.7 *Disease hosts*. Are the populations of intermediate and other primary hosts of parasitic and water-related diseases (rodents, birds, monkeys, fish, domestic animals) and the interaction of the human population with these hosts, decreased or increased by the Project?

7.8 *Disease control*. Can the transmission of the diseases identified in 7.1, 7.2, 7.5, 7.6 and 7.7 be reduced by introducing into the Project environmental modifications or manipulations or by any other sustainable control methods? Possible environmental measures include both removal of breeding, resting and hiding places of vectors and reducing contamination by and contact with humans.

7.9 *Other hazards*. Is the risk to the population decreased or increased with respect to:

- pathogens or toxic chemicals present in irrigation water (particularly through wastewater reuse) or in the soils, which can accumulate in food crops or directly threaten the health of the population ;
- dwellings adequately located and designed to withstand any storm, earthquake or flood hazards;
- sudden surges in river flow caused by the operation of spillways or power turbines; and
- structures and water bodies designed to minimise accident and allow escape?

1.9.4.8. Ecological Imbalances

8.1 *Pests and weeds*. Are crop pests or weeds likely to increase or decrease (particularly those favoured by irrigation/drainage/flood control) affecting yields, cultivation and requirements for pesticides or herbicides?

8.2 *Animal diseases*. Are domestic animals in the Project or vicinity more or less exposed to hazards, diseases and parasites as a fault of the Project and its dams?

8.3 *Aquatic weeds*. Are reservoirs, rivers or irrigation and drainage canals likely to support aquatic vegetation or algae? If so, can these plants be harvested or controlled, or will they reduce the storage/conveyance capacity, interfere with the operation of hydraulic structures or lead to oxygen-oversaturated or anaerobic water bodies?

8.4 *Structural damage*. Is there a danger of significant damage being caused to dams, embankments, canal banks or other components of the irrigation/drainage/flood control works through the action of plants and animals (including rodents and termites) favoured by the Project?

8.5 *Animal imbalances*. Does the Project cause zoological imbalances (insects, rodents, birds and other wild animals) through habitat modification, additional food supply and shelter, extermination of predators, reduced competition or increased diseases?

The recommended approach for use of the environmental check-list (Table 1.14) is to prepare a detailed description for each of the check-list items on the basis of collected information required for the purpose. Based on these descriptions, the extent of the environmental effect is assessed and a cross (×) is entered in one of the columns A to E. The total number of crosses in each column of Table 1.14 gives an indication of the number of responses in each category. However, these numbers should not be given strict quantitative significance in assessing the overall balance of positive and negative changes from the project since certain changes will be far more important than others (12).

1.10. CROPS AND CROP SEASONS

Major crops and crop seasons of India have been briefly described in this article.

1.10.1. Crop Seasons

Activities relating to crops go on continuously throughout the year in India. In north India, there are two main crop seasons. These are '*Kharif*' (July to October) and '*Rabi*' (October to March). Crops grown between March and June are known as '*Zaid*'. In other parts of the country there are no such distinct seasons but some kind of classification of crop seasons exists every where. The Kharif season is characterised by a gradual fall in temperature, larger number of rainy days, low light intensity, a gradual shortening of the photoperiod, high relative humidity, and cyclonic weather. On the other hand, bright sunshine, near absence of cloudy days, and lower relative humidity are the characteristics of the Rabi season. The Kharif season starts earlier in the eastern part of the country because of the earlier arrival of the monsoon and continues until the withdrawal of the monsoon. On the other hand, the Rabi season starts earlier in the western part and continues until the sun attains equatorial position. Thus, Kharif is longer in the eastern part and Rabi is longer in the western part.

There are several cropping patterns which are followed in India depending upon the climatic, edaphic, socio-economic conditions of the region. With a geographic area of about 329 Mha, stretching between 8°N and 36°N latitude and between 68°E and 98°E longitude, and its altitude varying from the mean sea level to the highest mountain ranges of the world, India hosts a variety of flora and fauna in its soil with few parallels in the world. The country has an average annual rainfall of 1,143 mm which varies from 11,489 mm around Cherrapunji in Assam to 217 mm around Jaisalmer in Rajasthan. Just as rainfall and temperature vary over a wide range, there is considerable difference in the socio-economic conditions of peasants of different parts of the country. Due to the variation in soil-climatic conditions there exists considerable variation in crop genotypes. Considering the potential of foodgrain production in different parts of India, the country has been divided into the following five agricultural regions (13):

(*i*) The eastern part including larger part of the north-eastern and south-eastern India, and another strip along the western coast form the rice region of India.

(*ii*) The wheat region occupies most of northern, western, and central India.

(*iii*) The millet (*bajra*)–sorghum (*jawar*) region comprising Rajasthan, Madhya Pradesh, and the Deccan plateau.

(*iv*) The Himalayan region of Jammu and Kashmir, Himachal Pradesh, Uttar Pradesh, and some adjoining areas in which potatoes, cereal crops (mainly maize and rice), and fruits are grown.

(*v*) The plantation crops (*e.g*. tea, coffee, rubber, and spices) are grown in Assam, hills of south India and peninsular region of India which form the plantation region.

1.10.2. Ideal Weather for Kharif and Rabi Seasons

At the end of May or beginning of June, there should be some rainfall so that the fields can be ploughed. Towards the end of June, heavy rainfall is required for thorough wetting of the land. This must be followed by a period of clear sky for tillage and sowing operations. In the months of July and August, there should be periods of bright sunshine (not exceeding ten days) between two spells of rain. The weather in the month of September should be similar to that in July and August, but with less rainfall. A few showers at the end of September are needed to prepare the land for Rabi crops.

The first requirement for a good Rabi crop is that the soil temperature should fall rapidly to germination temperature. During November and early December, clear days and cool weather are beneficial. Towards the end of December, a light rainfall is useful. The winter rain must be broken by clear weather as continuous cloudy weather results in widespread plant diseases. The rest of the Rabi season should be dry and free from hailstorms.

1.10.3. Crops of Kharif Season

Kharif (or south-westerly monsoon) crops include rice, maize, *jawar, bajra,* groundnut, cotton and other crops.

1.10.3.1. Rice

Rice cultivation in India stretches from 8°N latitude to 34°N latitude. Rice is also grown in areas below the sea level (as in the Kuttanad region of Kerala) as well as at altitudes of about 2000 m (as in parts of Jammu and Kashmir). High rainfall or assured irrigation is essential for areas of rice cultivation. Rice crop requires about 30 cm of water per month during the growing period stretching from about 3 to 8 months. Rice is grown on about 40 Mha in the country. This area also includes about 7 Mha which is saline, alkaline or flood-prone. Twenty-five per cent of the rice growing area has assured irrigation and about 55 per cent of the rice growing area is ill-drained or waterlogged. The rest of the rice-growing area is rainfed uplands where the rainfall is marginal to moderate and its distribution is erratic.

Rice cultivation in India is either upland cultivation or lowland cultivation. The upland system of cultivation is confined to such areas which do not have assured irrigation facilities. In this system, fields are ploughed in summer, farmyard manure is uniformly distributed 2–3 weeks before sowing, and the rain water is impounded in the field until the crop is about 45–60 days old.

In the lowland system of rice cultivation, the land is ploughed when 5–10 cm of water is standing in the field. Seeds may be sown after sprouting. Alternatively, seedling which are 25–30 days old are transplanted. The nursery area required to provide seedlings for transplanting on one hectare is roughtly one-twentieth of a hectare. The water requirement of lowland rice cultivation is much higher than that of other cereal crops with similar duration.

1.10.3.2. Maize

Maize is one of the main cereals of the world and ranks first in the average yield. Its world average yield of 27.8 quintals/hectare (q/ha) is followed by the average yields of rice (22.5 q/

ha), wheat (16.3 q/ha) and millets (6.6 q/ha). In terms of area of maize cultivation, India ranks fifth (after USA, Brazil, China and Mexico) in the world. However, India stands eleventh in the world in terms of maize production. Within India, maize production ranks only next to rice, wheat, jawar, and bajra in terms of area as well as production. Most of the maize cultivation (around 75 per cent) is in the states of Uttar Pradesh (1.4 Mha), Bihar (0.96 Mha), Madhya Pradesh (0.58 Mha), Rajasthan (0.78 Mha) and Punjab (0.52 Mha).

Maize requires deep and well-drained fertile soils, but can be grown on any type of soil ranging from heavy clays to light sands provided that the pH does not deviate from the range 7.5 to 8.5. Maize plants, particularly in the seedling stage, are highly susceptible to salinity and waterlogging, and hence, proper drainage of the land is essential for the successful cultivation of maize. Over 85 per cent of the crop area in India is rainfed during the monsoon.

Maize is essentially a warm weather crop grown in different regions of the world ranging from tropical to temperate ones. It cannot withstand frost at any stage of its growth. In India, its cultivation extends from the hot arid plains of Rajasthan and Gujarat to the wet regions of Assam and West Bengal.

Maize is a short-duration (80–95 days) crop and, hence, can conveniently fit into a wide range of crop rotations. It is usually grown as a pure crop, but sometimes legumes (*e.g.*, *moong, arhar* or beans), and quick-growing vegetables (*e.g.,* pumkins, gourds) are grown as mixed crops with it.

The sowing of maize starts 7–10 days before the usual date of the onset of monsoon. One irrigation at the initial stage is useful for the establishment of seedlings and the crop yield is increased by about 15–20 per cent. The maize crop is harvested when the grains are nearly dry and do not contain more than 20 per cent moisture. Maize is grown for grains as well as fodder.

1.10.3.3. Sorghum (Jawar)

Sorghum (popularly known as *jawar*) is the main food and fodder crop of dryland agriculture. It is grown over an area of about 18 Mha with the average yield of about 600 kg/ha. *Jawar* cultivation is concentrated mainly in the peninsular and central India. Andhra Pradesh, Gujarat, Karnataka, Madhya Pradesh, Maharashtra, Rajasthan, Tamil Nadu, and Uttar Pradesh are the major *jawar*-growing states. *Jawar* is mainly grown where rainfall distribution ranges from 10–20 cm per month for at least 3 to 4 months of the south-westerly monsoon.

Sorghum is grown during both Kharif (July–October) and Rabi (October–February) seasons. The Rabi cultivation of *jawar* constitutes about 37 per cent of the total *jawar*-growing area. Sorghum cultivation still remains predominantly traditional in most parts of the country. Mixed cropping of *jawar* and *arhar* (*tur*) is very common. Harvesting and threshing are still carried out manually or with bullock power. The national average yields are still low and around 500 kg/ha. However, the high-yielding hybrid varieties can yield 2000–3000 kg/ha under average growing conditions.

1.10.3.4. Spiked Millet (Bajra)

Bajra is a drought-resistant crop which is generally preferred in low rainfall areas and lighter soils. It is grown in Rajasthan, Maharashtra, Gujarat, and Uttar Pradesh. Over 66 per cent of this crop is grown in areas receiving 10–20 cm per month of rainfall, extending over 1 to 4 months of the south-westerly monsoons. It should be noted that *jawar* and *bajra* are grown mostly under identical environmental conditions and both have a wide range of adaptability to drought, temperature, and soil.

1.10.3.5. Groundnut

Groundnut is grown over an area of about 7 Mha concentrated in the states of Gujarat (24 percent), Andhra Pradesh (20 per cent), Karnataka (12 per cent), Maharashtra (12 per cent), and Tamil Nadu (13 per cent). Madhya Pradesh, Orissa, Punjab, Rajasthan, and Uttar Pradesh together have about 20 per cent of the total groundnut producing area in the country. Groundnut is generally grown as a rainfed Kharif crop. Groundnut is sown during May and June in the subtropics. In the tropics, however, it is sown during either January and February or June and July. Under rainfed conditions the average yield is 1200–1400 kg per hectare.

1.10.3.6. Cotton

Cotton occupies about 7.5 Mha in India. Maharashtra (36 per cent), Gujarat (21 per cent), Karnataka (13 per cent), and Madhya Pradesh (9 per cent) are the leading states which together grow cotton over an area of about 6 Mha. Other cotton growing states are Punjab (5 per cent), Andhra Pradesh (4 per cent), Tami Nadu (4 per cent), Haryana (3 per cent), and Rajasthan (3 per cent). Most of the cotton-growing areas in the country are in the high to medium rainfall zones.

Cotton requires a well-drained soil. It is grown as a rainfed crop in the black cotton and medium black soils and as an irrigated crop in alluvial soils. The sowing season varies from region to region and starts early (April-May) in north India.

1.10.4. Crops of Rabi Season

Main crops of Rabi (Post-monsoon) season are wheat, barley and gram.

1.10.4.1. Wheat

In terms of production, wheat occupies the first place among the food crops in the world. In India, it is the second most important food crop, next only to rice. The Indo-Gangetic plains form the most important wheat area. The cool winters and hot summers are conducive to a good crop of wheat. Well-drained loams and clayey loams are considered good soils for the cultivation of wheat. However, good crops of wheat can be raised in sandy loams and black soils also.

Wheat crop requires a well-pulverized but compact seedbed for good and uniform germination. Under irrigated conditions, the first fortnight of November is considered the optimum time for sowing the medium to long-duration wheats (*e.g*. the ‘Kalyanasona’ variety). For short-duration wheats (*e.g*. the ‘Sonalika’ variety) the second fortnight of November is the optimum time of sowing. In eastern India, wheat is sown in the third week of December due to the late harvesting of paddy. In north-western India also, wheat sowings get delayed due to the late harvesitng of paddy, sugarcane or potato.

For wheat sown under irrigated conditions, four to six irrigations are required. The first irrigation should be given at the stage of initiation of the crown root, *i.e.,* about 20–25 days after sowing. Two or three extra irrigations may be required in case of very light or sandy soils.

The crop is harvested when the grains harden and the straw becomes dry and brittle. The harvesting time varies in different regions. In the peninsular region, harvesting starts in the latter half of February and is over in the first week of March. In the central zone, the peak season for harvesting is in the month of March. In the north-western zone, the peak harvesting period is the latter half of April. In the eastern zone, harvesting is over by mid–April. However, in the hills, the wheat crop is harvested in the months of May and June.

Punjab, Haryana, Delhi, Uttar Pradesh, Madhya Pradesh, Rajasthan, Gujarat, Bihar, and West Bengal together grow wheat over an area exceeding 70 per cent of the total area of wheat crop for the country. These states also produce 76 percent of the total what production of India and have extensive irrigation systems covering from 85 per cent of the area in Punjab to 51 per cent in Bihar.

1.10.4.2. Barley

Barley (*Jau*) is an important rabi crop ranking next only to wheat. The total area under this crop is about 3.0 Mha, producing nearly 3 million tonnes of grain. Main barley growing states are Rajasthan, Uttar Pradesh, and Bihar which together grow barley over an area which is about 80 per cent of total barley growing area.

This crop can be grown successfully on all soils which are suitable for wheat cultivation. Barley crop needs less water and is tolerant to salinity. Recent experiments indicate that this crop can be grown on coastal saline soils of Sunderbans in West Bengal and on saline soils in areas of north Karnataka irrigated by canals.

The normal sowing season for barley extends from middle of October to the middle of November, but it can be sown as late as the first week of January. Barley is grown either on conserved moisture or under restricted irrigation. Generally, it needs two to three irrigations. On highly alkaline or saline soils, frequent light irrigations are given.

Harvesting period for barley is between mid-March to mid-April. Harvesting starts in the month of February in Maharashtra, Gujarat, and Karnataka. In the foothills of the Himalayas, harvesting time varies from the end of April to the end of May. The average grain yield of the 'dry' crop is about 700–1000 kg/ha whereas that of the irrigated crop is about twice as much.

1.10.4.3. Gram

Gram (*Chana*) is the most important pulse which accounts for more than a third of the pulse growing area and about 40 per cent of the production of pulses in India. The average annual area and production of gram are about 7–8 Mha and about 4–5 million tonnes of grain respectively. Haryana, Himachal Pradesh, Rajasthan, and Uttar Pradesh together grow gram over an area exceeding 6 Mha.

In North India, gram is grown on light alluvial soils which are less suitable for wheat. In south India, gram is cultivated on clay loams and black cotton soils. 'Kabuli gram', however, requires soil better than light alluvial soils. Gram is generally grown as a dry crop in the Rabi season.

The preparation of land for gram is similar to that for wheat. The seeds are sown in rows from the middle of October to the beginning of November. The crop matures in about 150 days in Punjab and Uttar Pradesh and in 120 days in south India.

1.10.5. Other Major Crops

1.10.5.1. Sugarcane

Sugarcane is the main source of sugar and is an important cash crop. It occupies about 1.8 per cent of the total cultivated area in the country. In the past, the area under sugarcane has been fluctuating between 2 and 2.7 Mha. Uttar Pradesh alone accounts for about 47 per cent of annual production in terms of raw sugar. However, the production per hectare is the highest in Karnataka followed by Maharashtra and Andhra Pradesh.

Medium heavy soils are best suited for sugarcane. It can also be grown on lighter and heavy soils provided that there is sufficient irrigation available in the former and drainage is good in the latter type of soils. In north India, it is cultivated largely on the loams and clay loams of the Gangetic and other alluviums. In peninsular India, it is grown on brown or reddish loams, laterites, and black cotton soils.

Sugarcane grows over a prolonged period. In north India, planting of sugracane coincides with the beginning of warm weather and is completed well before the onset of summer. Usually, January and February are the best months for planting of suagrcane in Bihar, February in Uttar Pradesh, and the first fortnight of March in Punjab and Haryana.

In the case of sugarcane, the maintenance of optimum soil moisture during all stages of growth is one of the essential requisites for obtaining higher yields. The crop should, therefore, be grown in areas of well-distributed rainfall with assured and adequate irrigation. The total irrigation requirement of the crop for optimum yield varies between 200 and 300 cm. Sugarcane ripens around December and its sugar content continues to rise till about the end of March by which time it is harvested in north India.

1.10.5.2. Tea

Tea is an important beverage and its consumption in the world is more than that of any other beverage. India and Sri Lanka are the important tea growing countries. In India, tea is grown in Assam, West Bengal, Kerala, Karnataka, and Tamil Nadu. Tea is grown over an area of about 358,000 hectares and about 470 million kilograms of the product is obtained annually. The tea crop is the most important plantation crop of India.

The tea plant, in its natural state, grows into a small or medium-sized tree. In commercial plantations, it is pruned and trained to form a multi-branched low bush. Appropriate schedule of fertiliser applications is very useful to produce vigorous vegetative growth of the tea crop. The tea plants are generally raised in nurseries. About one to one-and-a-half year old nursery seedlings are used for field plantation. Timely irrigation is essential for the production of good quality leaves.

1.10.5.3. Potato

Amongst vegetables, potato is grown over the largest area (for any single vegetable) in the world. In the plains of north India, potato is sown from the middle of September to the beginning of January. Two successive crops can be raised on the same land. Potato needs frequent irrigation depending upon the soil and climatic conditions. Generally, six irrigations are sufficient.

Salient details of some of the main crops of India are given in Table 1.15. Table 1.16 gives details about irrigated area under principal crops in different states (14).

1.10.6. Multiple Cropping

To meet the food requirements of ever-growing population of India, the available cultivable land (about 143 Mha) should be intensively cropped. This can be achieved by multiple cropping which increases agricultural production per unit area of cultivated land in a year with the available resource in a given environment. There are two forms of multiple cropping:

Table 1.15 Salient details of some crops of north India (Plains)

Crop	*Sowing time*	*Harvesting time*	*Seed requirement (kg/ha)*	*Average yield under normal condtions (q/ha)*	*Average water depth (mm)*
Rice	June–July	October–November	40–50	20–40	1500–2000
Maize	June–July Jan.–Feb.	September–October	40–50	15–30	150–200
Sorghum (*Jawar*)	June–July	October–November	20–30	15–30	150–200
Spiked Millet (*Bajra*)	July	October–November	5–10	15–30	150–200
Groundnut	June–July	November–December	100–120	20–25	200–250
Cotton	April–May	November–January	15–20	2–5 (with seeds)	500–700
Wheat	November–December	April–May	100–120	20–40	300–400
Barley	October–November	March–April	80–100	20–40	250–300
Gram	October–November	March–April	30–40	15–30	250–300
Sugarcane	October–November and February–March	October–April	3000–4000	8000–10000	1500–2000
Potatoes	September–December	November–February	1500–2500	25000–30000	400–500

(*i*) intercropping, and (*ii*) sequential cropping. When two or more crops are grown simultaneously on the same field, it is termed intercropping. Crop intensification is in both time and space dimensions. There is, obviously, strong intercrop competition in this form of multiple cropping. On the other hand, when two or more crops are grown in sequence on the same field in a year, it is termed sequential cropping. The succeeding crop is planted after the preceding crop has been harvested. Crop intensification is only in time dimension and there is no intercrop competition in sequential cropping.

Table 1.16 State-wise irrigated area under selected major crops (2)

Sl. No.	*State*	*Rice*	*Wheat*	*Pulses*	*Total Food Grains*	*Other Major Crops*	*All Crops*	*Total Irrigated Area to Total Crop Area (%)*
1.	Andhra Pradesh	3830	8	19	4037	830	5369	40.7
2.	Arunachal Pradesh	32	–	–	32	–	32	13.0
3.	Assam	532	–	6	538	–	572	15.1
4.	Bihar and Jharkhand	1910	1681	21	3912	60	4192	40.0
5.	Goa	11	–	5	16	2	31	20.5
6.	Gujarat	283	523	78	1144	1107	2904	28.0
7.	Haryana	655	1805	196	2861	931	4237	71.6
8.	Himachal Pradesh	49	65	1	144	2	167	17.0
9.	Jammu & Kashmir	250	58	6	341	46	436	40.9
10.	Karnataka	716	181	65	1340	668	2598	22.1
11.	Kerala	224	–	–	224	2	383	12.7
12.	M.P. & Chattish Garh	1019	2014	586	3655	439	4431	18.6
13.	Maharashtra	467	388	129	1483	540	2487	11.4
14.	Manipur	75	–	–	75	–	75	41.7
15.	Meghalaya	47	–	–	47	–	47	19.3
16.	Mizoram	8	–	–	8	–	8	10.8
17.	Nagaland	59	–	–	59	–	60	28.6
18.	Orissa	1566	32	115	1748	137	2314	24.1
19.	Punjab	1998	3144	73	5352	860	7055	94.0
20.	Rajasthan	27	1626	341	2291	1759	4652	24.0
21.	Sikkim	16	–	–	16	–	16	10.5
22.	Tamil Nadu	1686	–	58	1886	585	2894	43.6
23.	Tripura	33	–	1	34	1	41	9.2
24.	U.P & Uttaranchal	2440	7669	744	11457	2063	14771	58.0
25.	West Bengal	1251	221	7	1480	127	1911	22.1
	Union Territories	29	32	1	70	9	91	
	Total (1990–91)	19213	19347	2452	44250	10168	61774	33.1
	Total (1950–51)	9844	3402	1939	18317	1648	2256	

Note : (1) Other major crops include groundnut, cotton, sugarcane, tobacco, rapeseed and mustard.
(2) – below 500 hectares.

Choice of a suitable cropping pattern for an area is dependent mainly on the soil characteristics and climatic conditions of the area. From the considerations of management of canal supplies, it is important to arrive at a cropping pattern which could be sustainable by the available water and also maximise economic benefits for the people of that area. For this purpose, the systems approach is very useful. Parameters, such as self sufficiency for the area in staple food and fodder, use of a diversified pattern to reduce risks of failure, problems related to storage and marketing particularly for perishable crops, reasonably uniform demand of water all through the year, and the preferences of the local farmers are always incorporated in the analysis.

1.10.7. Hybrid Cropping

Hybrid is an offspring of parents belonging to different characteristic groups of the same genetic group. Plant and animal breeders have developed special techniques for producing hybrids artificially in laboratories, zoos, and farms.

Hybrids generally tend to be sterile. Even if they can produce, the first generation offspring may resemble their parents but next generation may not. The second generation usually shows different combination of the characteristics of the original crossbred parents.

Growing of a crop with hybrid seeds is called hybrid cropping. The hybrid seeds may unite the desirable traits of both parents. For example, a gardener may crossbreed an ornamental large flower with a sweet-smelling variety to produce a hybrid variety of large aromatic flowers.

The hybrid seeds have what is called hybrid vigour *i.e.,* they generally tend to be large, faster-growing and healthier than their parent. This fact has been exploited commercially in the cultivation of corn (maize), potatoes, cotton, and several varieties of flowers. However, the hybrid seeds are very costly and, therefore, cannot be adopted on a mass scale in the country. Hybrid seeds, however, appear more promising for glasshouse cultivation of plants.

EXERCISES

1.1. What is irrigation? What has been its impact on human environment?

1.2. Justifying the need of irrigation in India, describe its development in the country.

1.3. What are the requirements for the success of an irrigation project? How is an irrigation project planned?

1.4. What are the objectives of command area development? How are these achieved through command area development programmes?

1.5. What are the main crop seasons of India? Describe the ideal weather conditions for these seasons.

1.6. What are the factors that must be considered for deciding an ideal cropping pattern in a given area ?

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2
HYDROLOGY

2.1. HYDROLOGY

The word *hydrology* means science of water which deals with the spatial and temporal characteristics of the earth's water in all its aspects such as occurrence, circulation, distribution, physical and chemical properties, and impact on environment and living things. Engineering hydrology deals with all these aspects which are pertinent to planning, design, and operation of hydraulic engineering projects for the control and use of the available water. Hydrology finds its application in the design and operation of water resources projects to estimate the magnitudes of flood flows at different times of a year to decide reservoir capacity, spillway discharge, dimensions of hydraulic structures etc. Basic concepts of hydrology have been dealt with in this chapter.

2.2. HYDROLOGIC CYCLE

The total water of earth, excluding deep ground water, is in constant circulation from the earth (including oceans) to atmosphere and back to the earth and oceans. This cycle of water amongst earth, oceans, and atmospheric systems is known as *hydrologic cycle*.

Figure 2.1 is an enormously simplified sketch of the hydrologic cycle for which sun is the source of energy. The hydrologic cycle (Fig. 2.1) can be visualized to begin with the evaporation (due to solar heat) of water from the oceans, streams and lakes of the earth into the earth's atmosphere. The water vapour, under suitable conditions, get condensed to form clouds moving with wind all over the earth's surface and which, in turn, may result in precipitation (in the form of rain water, snow, hail, sleet *etc*.) over the oceans as well as the land surface of the earth. Part of the precipitation, even while falling, may evaporate back into the atmosphere. Another part of the precipitation may be intercepted by vegetation on the ground or other surfaces. The intercepted precipitation may either evaporate into the atmosphere or fall back on the earth's surface. The greater part of the precipitation falling on the earth's surface is retained in the upper soil from where it may return to the atmosphere through evaporation and transpiration by plants and/or find its way, over and through the soil surface as runoff, to stream (or river) channels and the runoff thus becoming stream flow. Yet another part of the precipitation may penetrate into the ground to become part of the ground water. The water of stream channels, under the influence of gravity, moves towards lower levels to ultimately meet the oceans. Water from ocean may also find its way into the adjoining aquifers. Part of the stream water also gets evaporated back into the atmosphere from the surface of the stream. The ground water too moves towards the lower levels to ultimately reach the oceans. The ground water, at times, is a source of stream flow.

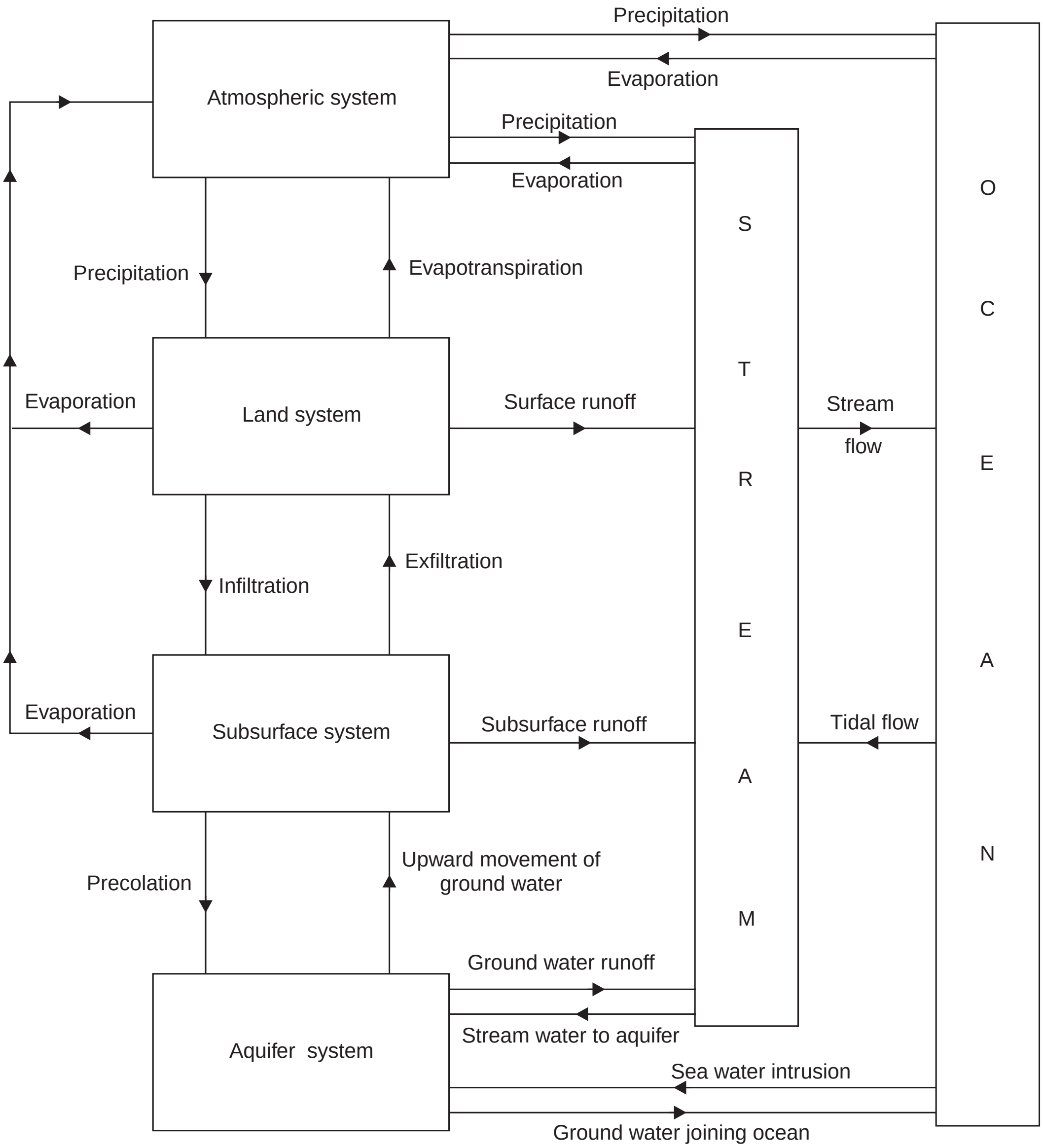

Fig.2.1 Hydrologic cycle

The description of the hydrologic cycle should not lead one to conclude that there is a continuous mechanism through which water moves steadily at a constant rate. The movement of water through the cycle is evidently variable, both in time and space although the total water resources of the earth remains invariant since the formation of the earth system. Further, the hydrologic cycle is a very complex phenomenon that has been taking place since the earth formed. It should also be noted that the hydrologic cycle is a continuous recirculating cycle with neither a beginning nor an end.

Hydrologic system is defined as a structure or volume in space surrounded by a boundary that receives water and other inputs, operates on them internally, and produces them as outputs

(1). The global hydrologic cycle can be termed a hydrologic system containing three subsystems : the atmospheric water system, the surface water system, and the subsurface water system. Another example of the hydrologic system is storm-rainfall-runoff process on a watershed. *Watershed* (or *drainage basin or catchment*) is a topographic area that drains rain water falling on it into a surface stream and discharges surface stream flow through one particular location identified as watershed outlet or watershed mouth. The term 'watershed' used for the catchment area should be distinguished from the watershed used in the context of canal alignment, chapter-5.

2.3. PRECIPITATION

The atmospheric air always contains moisture. Evaporation from the oceans is the major source (about 90%) of the atmospheric moisture for precipitation. Continental evaporation contributes only about 10% of the atmospheric moisture for precipitation. The atmosphere contains the moisture even on days of bright sun-shine. However, for the occurrence of precipitation, some mechanism is required to cool the atmospheric air sufficiently to bring it to (or near) saturation. This mechanism is provided by either convective systems (due to unequal radiative heating or cooling of the earth's surface and atmosphere) or by orographic barriers (such as mountains due to which air gets lifted up and consequently undergoes cooling, condensation, and precipitation) and results into, respectively, convective and orographic precipitations. Alternatively, the air lifted into the atmosphere may converge into a low-pressure area (or cyclone) causing cyclonic precipitation. Artificially induced precipitation requires delivery of dry ice or silver iodide or some other cloud seeding agent into the clouds by aircrafts or balloons.

The common forms of precipitation are *drizzle* or *mist* (water droplets of diameters less than 0.5 mm), *rain* (water drops of size between 0.5 mm and 6.0 mm), *snow* (ice crystals combining to form flakes with average specific gravity of about 0.1), *sleet* (rain water drops, falling through air at or below freezing temperatures, turned to frozen rain drops), and *hail* (precipitation in the form of ice balls of diameter more than about 8 mm). Most of the precipitation, generally, is in the form of rains. Therefore, the terms precipitation and rainfall are considered synonymous. Rainfall, *i.e.,* liquid precipitation, is considered light when the rate of rainfall is upto 2.5 mm/hr, moderate when the rate of rainfall is between 2.5 mm/hr and about 7.5 mm/hr, and heavy when the rate of rainfall is higher than about 7.5 mm/hr.

2.3.1. Characteristics of Precipitation in India

India receives more than 75% of its annual precipitation during the monsoon season (June to September). The monsoon (*i.e.,* south-west monsoon) originates in the Indian ocean and appears in the southern part of Kerala by the end of May or the beginning of June. The monsoon winds, then, advance and cover the entire country by mid-July.The monsoon season is, however, not a period of continuous rainfall. The temporal and spatial variability of the magnitude of rainfall results into regions of droughts and floods. Relatively speaking, Assam and the north-eastern reigon are the heavy rainfall regions (with average annual rainfall ranging from 2000-4000 mm) and U.P., Haryana, Punjab, Rajasthan, and Gujarat constitute low rainfall regions (with average annual rainfall less than about 1000 mm). Western Ghats receive about 2000-3000 mm of annual rainfall. Around mid-December, the western disturbances cause moderate to heavy rain and snowfall (about 250 mm) in the Himalayas and Jammu and Kashmir and other northern regions of the country. Low pressure areas formed in the Bay of Bengal during this period cause some rainfall in the south-eastern parts of the country.

The temporal variation of annual rainfall at a given place is expressed in terms of the coefficient of variation, C_v defined as

$$C_v = \frac{100 \times \text{standard deviation}}{\text{mean}} = \frac{100\sigma_{m-1}}{\overline{P}} \tag{2.1}$$

The coefficient of variation of the annual rainfall for different places may vary between 15 (for regions of high rainfall) and 70 (for regions of scanty rainfall) with an average value of about 30.

2.3.2. Measurement of Precipitation

One of the most crucial and least known components of the global hydrologic cycle is the precipitation that is the basic data required to estimate any hydrologic quantity (such as runoff, flood discharge *etc.*). Therefore, measurement of precipitation is an important component of all hydrologic studies. Weather and water-balance studies too require information on precipitation.

2.3.2.1. Precipitation Gauges

Precipitation (of all kinds) is measured in terms of depth of water (in millimeters) that would accumulate on a level surface if the precipitation remained where it fell. A variety of instruments have been developed for measuring precipitation (or precipitation rate) and are known as precipitation gauges or, simply, rain gauges which are classified as either recording or non-recording rain gauges.

Non-recording rain gauges only collect rain water which, when measured suitably, gives the total amount of rainfall at the rain gauge station during the measuring interval. The Indian Meteorological Department has adopted Symon's rain gauge (Fig. 2.2). A glass bottle and funnel with brass rim are put in a metallic cylinder such that the top of the cylinder is 305 mm above the ground level. Rain water falls into the glass bottle through the funnel. The water collected in the bottle is measured with the help of a standard measuring glass jar which is supplied with the rain gauge. The jar measures rainfall in millimeters. At each station, rainfall observations are taken twice daily at 8.30 a.m. and 5.30 p.m.

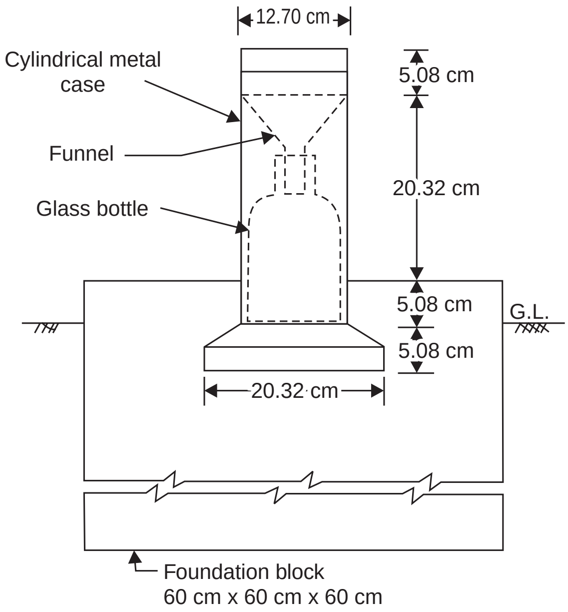

Fig. 2.2 Symon's rain gauge

Recording rain gauges automatically record the intensity of rainfall and the time of its occurrence in the form of a trace (or graph) marked on a graph paper wrapped round a revolving drum. Following three types are the most widely used recording rain gauges :

(*i*) Tipping bucket rain gauge,

(*ii*) Weighing bucket rain gauge, and

(*iii*) Siphon rain gauge.

(*i*) ***Tipping bucket rain gauge :*** A 300 mm diameter funnel collects rain water and conducts it to one of the two small buckets (Fig. 2.3) which are so designed that when 0.25 mm of rainfall is collected in a bucket, it tilts and empties its water into a bigger storage tank and, simultaneously, moves the other bucket below the funnel. When any of the two buckets tilts, it actuates an electric circuit causing a pen to make a mark on a revolving drum. The recording equipment can be remotely located in a building away from the rain gauge. At a scheduled time, the rain water collected in the storage tank can be measured to yield total rainfall in the measuring duration. The rainfall intensity (and also the total rainfall) can be estimated by studying the record sheet on which each mark indicates 0.25 mm of rain in the duration elapsed between the two adjacent marks.

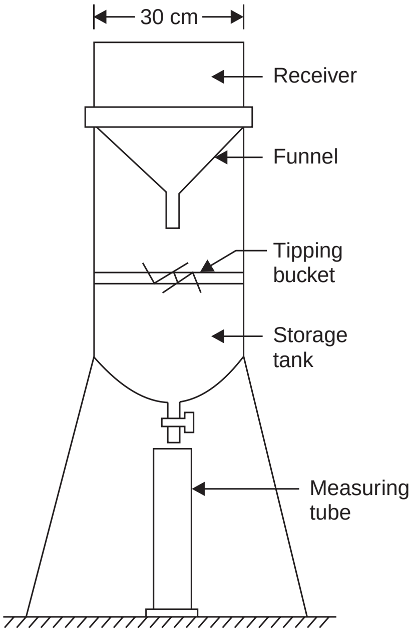

Fig. 2.3 Tipping bucket rain gauge

(*ii*) ***Weighing bucket rain gauge :*** This gauge (Fig. 2.4) has a system by which the rain that falls into a bucket set on a platform is weighed by a weighing device suitably attached to the platform. The increasing weight of rain water in the bucket moves the platform. This movement is suitably transmitted to a pen which makes a trace of accumulated amount of rainfall on a suitably graduated chart wrapped round a clockdriven revolving drum. The rainfall record of this gauge is in the form of a mass curve of rainfall (Fig. 2.5). The slope of this curve at any given time gives the intensity of rainfall at that time.

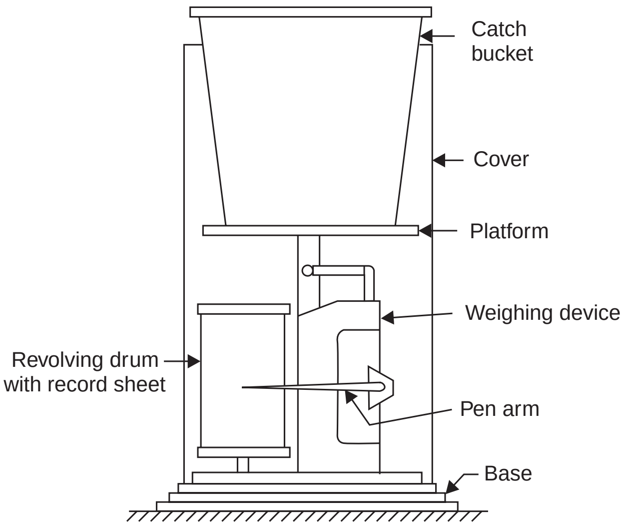

Fig. 2.4 Weighing bucket rain gauge

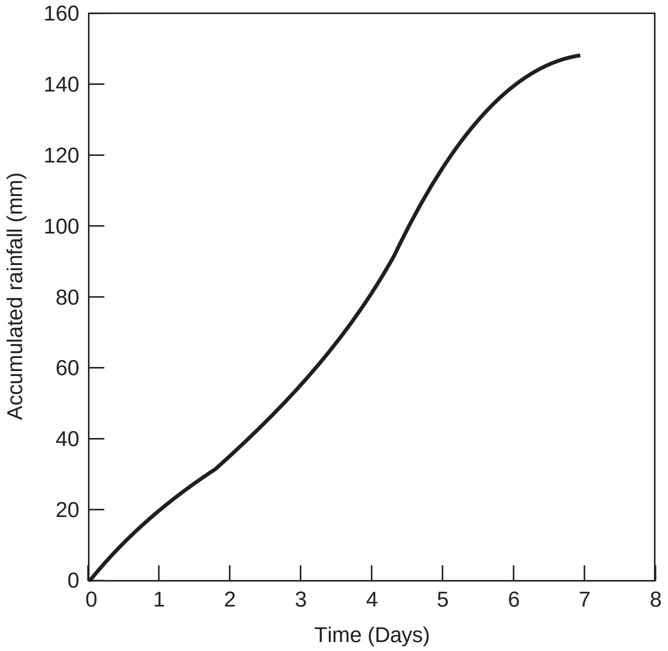

Fig. 2.5 Rainfall record of bucket rain gauge (mass curve of rainfall)

(*iii*) ***Siphon rain gauge :*** This gauge (Fig. 2.6) is also called float type rain gauge as this gauge has a chamber which contains a light and hollow float. The vertical movement of float on account of rise in the water level in the chamber (due to rain water falling in it) is transmitted by a suitable mechanism to move a pen on a clock-driven revolving chart. The record of rainfall is in the form of a mass curve of rainfall and, hence, the slope of the curve gives the intensity of rainfall.

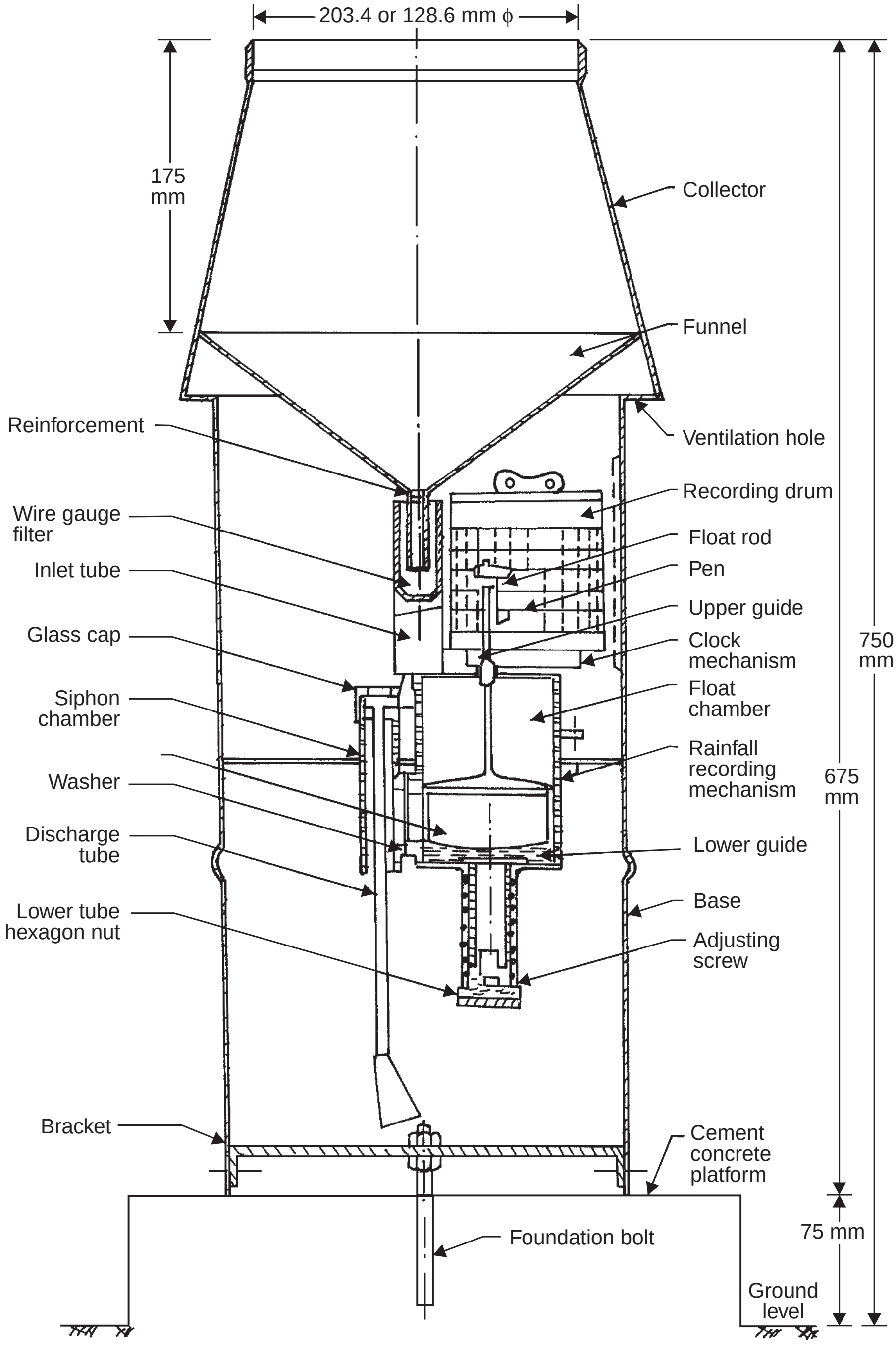

Fig. 2.6 Siphon rain gauge

Bureau of Indian Standards has laid down the following guidelines for selecting the site for rain gauges (IS : 4897-1968):

1. The rain gauge shall be placed on a level ground, not upon a slope or a terrace and never upon a wall or roof.

2. On no account the rain gauge shall be placed on a slope such that the ground falls away steeply in the direction of the prevailing wind.

3. The distance of the rain gauge from any object shall not be less than twice the height of the object above the rim of the gauge.

4. Great care shall be taken at mountain and coast stations so that the gauges are not unduly exposed to the sweep of the wind. A belt of trees or a wall on the side of the prevailing wind at a distance exceeding twice its height shall form an efficient shelter.

5. In hills where it is difficult to find a level space, the site for the rain gauge shall be chosen where it is best shielded from high winds and where the wind does not cause eddies.

6. The location of the gauge should not be changed without taking suitable precautions. Description of the site and surroundings should be made a matter of record.

2.3.2.2. Radar Measurement of Precipitation

In regions of difficult and inaccessible terrains, precipitation can be measured (within about 10% accuracy of the rain gauge measurements) with the help of a radar (radio detecting and ranging). A radar transmits a pulse of electromagnetic waves as a beam in a direction depending upon the position of the movable antenna. The wave travelling at a speed of light is partially reflected by cloud or precipitation particles and returns to the radar where it is received by the same antenna. The display of the magnitude of the energy of the returned wave on the radarscope (*i.e.,* radar screen) is called an *echo* and its brightness is termed *echo intensity*. The duration between the transmission of the pulse and appearance of the echo on the radarscope is a measure of the distance (*i.e.,* range) of the target from the radar. Direction of the target with respect to the radar is decided by the orientation of the antenna at the time the target signal is received. The echo is seen in polar coordinates. If there is no target (*i.e.,* cloud or precipitation particles), the screen is dimly illuminated. A small target would appear as a bright point whereas an extended target (such as a rain shower) would appear as a bright patch. The radarscope being divided as per the coordinate system, the position of the target can be estimated. By having a proper calibration between the echo intensity and rainfall (or its intensity), one can estimate the rainfall (or rainfall intensity). The Indian Meteorological Department has a well-established radar network for the purpose of detecting thunderstorms besides a few cyclone-warning radars along the eastern coast of the country.

The wavelength of the electromagnetic waves transmitted by the meterological radars is in the range of 3 to 10 cm; the usual operating range being 5 cm (for light rains) to 10 cm (for heavy rains). The relationship among the characteristics of the waves and the rainfall intensity is represented by

$$P_r = CZ/r^2$$

where, P_r is the average echo power, r is the distance from radar to target and C is a suitable constant. The radar echo factor Z is related to the intensity of rainfall I (in mm/hr) as

$$Z = aI^b$$

in which, a and b are numerical constants that are determined by calibrating the radar. One may, thus, obtain

$$I = [r^2 P_r/(aC)]^{1/b}$$

Present day developments in radar measurements of precipitation include on-line processing of the radar data and Doppler type radars for measuring the velocity and distribution of raindrops.

2.3.2.3. Satellite Measurement of Preciptation

It is a common experience that gauge network for measuring precipitation in a large and inaccessible area (such as in desert and hilly regions) is generally inadequate, and non-existent in oceans. The satellite observation is the only effective way for continuous monitoring of precipitation events over a large or inaccessible area. Use of the meterological satellites for weather and water balance studies is, therefore, continuously increasing.

In satellite measurements, the precipitation is estimated by correlating the satellite-derived data and observed rainfall data. These relationships can be developed for a part of electromagnetic spectrum using cloud life history or cloud indexing approach. The first approach uses data from geo-stationary satellites that produce data at every half an hour interval. The second approach, based on cloud classification, does not require a series of consecutive observations of the same cloud system (2).

Microwave remote sensing techniques that can directly monitor the rainfall characteristics have great potential in rainfall measurement.

2.3.3. Average Depth of Precipitation Over an Area

The information on the average depth of precipitation (or rainfall) over a specified area on either the storm basis or seasonal basis or annual basis is often required in several types of hydrologic problems. The depth of rainfall measured by a rain gauge is valid for that rain gauge station and in its immediate vicinity. Over a large area like watershed (or catchment) of a stream, there will be several such stations and the average depth of rainfall over the entire area can be estimated by one of the following methods:

2.3.3.1. Arithmetic Mean Method

This is the simplest method in which average depth of rainfall is obtained by obtaining the sum of the depths of rainfall (say P_1, P_2, P_3, P_4 .... P_n) measured at stations 1, 2, 3, ... n and dividing the sum by the total number of stations *i.e.* n. Thus,

$$\overline{P} = \frac{P_1 + P_2 + P_3 + \ldots\ldots P_n}{n} = \frac{1}{n}\sum_{i=1}^{n} P_i \tag{2.2}$$

This method is suitable if the rain gauge stations are uniformly distributed over the entire area and the rainfall variation in the area is not large.

2.3.3.2. Theissen Polygon Method

The Theissen polygon method takes into account the non-uniform distribution of the gauges by assigning a weightage factor for each rain gauge. In this method, the enitre area is divided into number of triangular areas by joining adjacent rain gauge stations with straight lines, as shown in Fig. 2.7 (*a* and *b*). If a bisector is drawn on each of the lines joining adjacent rain gauge stations, there will be number of polygons and each polygon, within itself, will have only one rain gauge station. Assuming that rainfall P_i recorded at any station i is representative rainfall of the area A_i of the polygon i within which rain gauge station is located, the weighted average depth of rainfall $\overline{P}$ for the given area is given as

$$\overline{P} = \frac{1}{A}\sum_{i=1}^{n} P_i A_i \tag{2.3}$$

where, $$A = \sum_{i=1}^{n} A_i = A_1 + A_2 + A_3 + \ldots\ldots A_n$$

Here, $\frac{A_i}{A}$ is termed the weightage factor for ith rain gauge.

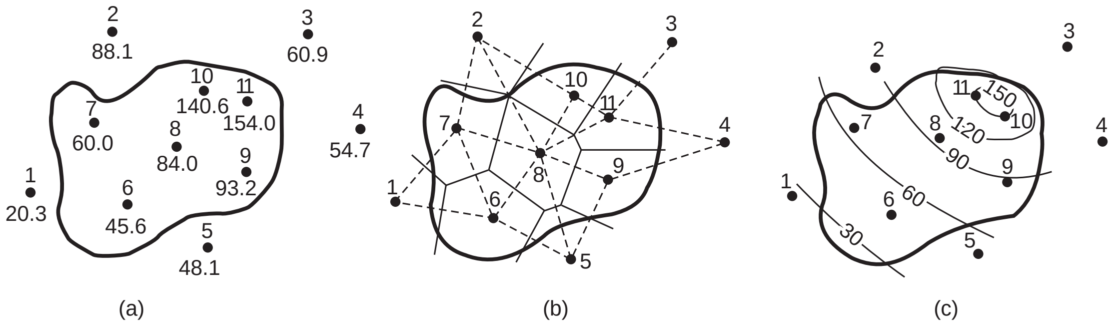

Fig. 2.7 Areal averaging of precipitation (*a*) rain gauge network, (*b*) Theissen polygons (*c*) isohyets (Example 2.1)

This method is, obviously, better than the arithmetic mean method since it assigns some weightage to all rain gauge stations on area basis. Also, the rain gauge stations outside the catchment can also be used effectively. Once the weightage factors for all the rain gauge stations are computed, the calculation of the average rainfall depth $\overline{P}$ is relatively easy for a given network of stations.

While drawing Theissen polygons, one should first join all the outermost raingauge stations. Thereafter, the remaining stations should be connected suitably to form quadrilaterals. The shorter diagonals of all these quadrilaterals are, then, drawn. The sides of all these triangles are, then bisected and, thus, Theissen polygons for all raingauge stations are obtained.

2.3.3.3. Isohyetal Method

An isohyet is a contour of equal rainfall. Knowing the depths of rainfall at each rain gauge station of an area and assuming linear variation of rainfall between any two adjacent stations, one can draw a smooth curve passing through all points indicating the same value of rainfall, Fig. 2.7 (*c*). The area between two adjacent isohyets is measured with the help of a planimeter. The average depth of rainfall $\overline{P}$ for the entire area A is given as

$$\overline{P} = \frac{1}{A} \Sigma \text{ [Area between two adjacent isohyets]} \times \text{[mean of the two adjacent isohyet values]} \qquad (2.4)$$

Since this method considers actual spatial variation of rainfall, it is considered as the best method for computing average depth of rainfall.

Example 2.1 The average depth of annual precipitation as obtained at the rain gauge stations for a specified area are as shown in Fig. 2.7 (*a*). The values are in cms. Determine the average depth of annual precipitation using (*i*) the arithmetic mean method, (*ii*) Theissen polygon method, and (*iii*) isohyetal method.

Solution: (*i*) Arithmetic mean method :

Using Eq. (2.2), the average depth of annual precipitation,

$$\overline{P} = \frac{1}{11} [20.3 + 88.1 + 60.9 + 54.7 + 48.1 + 45.6 + 60.0 + 84.0 + 93.2 + 140.6 + 154.0]$$

$$= \frac{1}{11} (849.5) = 77.23 \text{ cm.}$$

(*ii*) Theissen polygons for the given problem have been shown in Fig. 2.7 (*b*). The computations for the average depth of annual precipitation are shown in the following Table :

Rain Gauge Station	*Rainfall* P_i *(cm)*	*Area of Polygon,* A_i *(km²)*	*Weightage factor (%)* $\frac{A_i}{\Sigma A_i} \times 100$	$P_i \frac{A_i}{\Sigma A_i}$
1	20.3	22	1.13	0.23
2	88.1	0	0	0
3	60.9	0	0	0
4	54.7	0	0	0
5	48.1	62	3.19	1.53
6	45.6	373	19.19	8.75
7	60.0	338	17.39	10.43
8	84.0	373	19.19	16.12
9	93.2	286	14.71	13.71
10	140.6	236	12.14	17.07
11	154.0	254	13.07	20.13
Total		1944	100.01	87.97

$$\therefore \quad \text{Average annual precipitation} = \Sigma P_i \frac{A_i}{\Sigma A_i} = 87.97 \text{ cm}$$

$$\approx 88 \text{ cm}$$

(*iii*) Isohyetal method : Isohyets are shown in Fig. 2.7(*c*). The computations for the average depth of annual precipitation are shown in the following Table :

Isohyets (cm)	*Net area* A_i *(km)²*	*Average Precipitation* P_i *(cm)*	P_iA_i
< 30	96	25	2400
30–60	600	45	27000
60–90	610	75	45750
90–120	360	105	37800
120–150	238	135	32130
> 150	40	160	6400
Total	1944		151480

$$\therefore \quad \text{Average annual precipitation for the basin} = \frac{151480}{1944} = 77.92 \text{ cm.}$$

2.3.4. Precipitation Gauge Network

The spatial variability of the precipitation, nature of the terrain and the intended uses of the precipitation data govern the density (*i.e.*, the catchment area per rain gauge) of the precipitation gauge (or rain gauge) network. Obviously, the density should be as large as possible depending upon the economic and other considerations such as topography, accessibility *etc*. The World Meteorological Organisation (WMO) recommends the following ideal densities (acceptable values given in brackets) of the precipitation gauge network (3):

1. For flat regions of temperate, mediterranean, and tropical zones, 600 to 900 sq. km (900–3000 sq. km) per station.
2. For mountainous regions of temperate, mediterranean, and tropical zones, 100 to 250 sq. km (250 to 1000 sq. km) per station.
3. For small mountainous islands with irregular precipitation, 25 sq. km per station.
4. For arid and polar zones, 1500 to 10,000 sq. km per station.

At least ten per cent of rain gauge stations should be equipped with self-recording gauges to know the intensities of rainfall. The Bureau of Indian Standards (4) recommends the following densities for the precipitation gauge network:

1. In plains: 520 sq. km per station;
2. In regions of average elevation of 1000 m: 260 to 390 sq. km per station; and
3. In predominantly hilly areas with heavy rainfall: 130 sq. km per station.

For an existing network of raingauge stations, one may need to know the adequacy of the raingauge stations and, therefore, the optimal number of raingauge stations N required for a desired accuracy (or maximum error in per cent, ε) in the estimation of the mean rainfall. The optimal number of raingauge stations N is given as

$$N = \left(\frac{C_v}{\varepsilon}\right)^2 \tag{2.5}$$

Here, C_v = the coefficient of variation of the rainfall values at the existing m stations (in per cent) and is calculated as (Eq. 2.1))

$$C_v = \frac{100 \times \sigma_{m-1}}{\overline{P}}$$

in which, σ_{m-1} = standard deviation = $\sqrt{\dfrac{\sum_i^m (P_i - \overline{P})^2}{m-1}}$ (2.6)

P_i = precipitation measured at i^{th} station

and $\overline{P}$ = mean precipitation = $\dfrac{1}{m}\left(\sum_i^m P_i\right)$

For calculating N, ε is usually taken as 10%. Obviously, the number N would increase with the decrease in allowable error, ε.

Example 2.2 A catchment has eight rain gauge stations. The annual rainfall recorded by these gauges in a given year are as listed in column 2 of the following Table.

Table for computation of $(P_i - \overline{P})^2$ (Example 2.2)

Rain Gauge	*Annual Rainfall (cm)*	$P_i - \overline{P}$	$(P_i - \overline{P})^2$
A	80.8	– 32.1	1030.41
B	87.6	– 25.3	640.09
C	102.0	– 10.9	118.81
D	160.8	47.9	2294.41
E	120.4	7.5	56.25
F	110.8	– 2.1	4.41
G	142.3	29.4	864.36
H	98.5	– 14.4	207.36
Total	903.2	0.0	5216.10

What should be the minimum number of the raingauges in the catchment for estimating the mean rainfall with an error of less than 7% ?

Solution: Mean rainfall, $\overline{P} = \frac{1}{m}\sum_{i=1}^{m} P_i = \frac{1}{8}\sum_{i=1}^{8} P_i = \frac{903.2}{8} = 112.9$ cm

Values of $P_i - \overline{P}$ and $(P_i - \overline{P})^2$ are as in cols. (3) and (4) of the Table.

$$\sigma_{m-1} = \sqrt{\frac{\Sigma(P_i - \overline{P})^2}{m-1}} = \sqrt{\frac{1}{7}(5216.10)} = 27.298$$

$$C_v = \frac{100 \times \sigma_{m-1}}{\overline{P}} = \frac{100 \times 27.298}{112.9} = 24.18$$

Therefore, the number of raingauges for errors of less than 7% in estimation of the mean annual rainfall

$$= \left(\frac{C_v}{\varepsilon}\right)^2 = \left(\frac{24.18}{7}\right)^2$$

$$= 11.93, \text{ say } 12.$$

Hence, four additional raingauges are required in the catchment.

2.3.5. Interpretation of Precipitation Data

Precipitation data must be checked for the continuity and consistency before they are analysed for any significant purpose. This is essential when it is suspected that the gauge site (or its surroundings) might have changed appreciably during the period for which the average is being computed.

2.3.5.1. Estimation of Missing Data

The continuity of a record of precipitation data may have been broken with missing data due to several reasons such as damage (or fault) in a rain gauge during a certain period. The missing data is estimated using the rainfall data of the neighbouring rain gauge stations. The missing annual precipitation P_x at a station x is related to the annual precipitation values, P_1, P_2, P_3 ...... P_m and normal annual precipitation, N_1, N_2, N_3 ...... N_m at the neighbouring M stations 1, 2, 3, ... M respectively. The normal precipitation (for a particular duration) is the mean value of rainfall on a particular day or in a month or year over a specified 30-year period.

The 30-year normals are computed every decade. The term normal annual precipitation at any station is, therefore, the mean of annual precipitations at that station based on 30-year record.

The missing annual precipitation P_x is simply given as

$$P_x = \frac{1}{M} (P_1 + P_2 + ... P_m) \tag{2.7}$$

if the normal annual precipitations at various stations are within about 10% of the normal annual precipitation at station x *i.e.,* N_x. Otherwise, one uses the normal ratio method which gives

$$P_x = \frac{N_x}{M} \left[\frac{P_1}{N_2} + \frac{P_2}{N_2} + ... \frac{P_M}{N_M} \right] \tag{2.8}$$

This methods works well when the precipitation regimes of the neighbouring stations and the station x are similar (or almost the same).

Multiple linear regression (amongst precipitation data of M stations and the station x, excluding the unknown missing data of station x and the concurrent (or corresponding) data of the neighbouring M stations) will yield an equation of the form

$$P_x = a + b_1 P_1 + b_2 P_2 + ... b_m P_m \tag{2.9}$$

in which, $a \approx 0$

and $b_i \approx \dfrac{N_x}{MN_i}$

The regression method allows for some weighting of the stations and adjusts, to some extent, for departures from the assumption of the normal ratio method.

2.3.5.2. Test for Cosistency of Precipitation Data

Changes in relevant conditions of a rain gauge (such as gauge location, exposure, instrumentation, or observation techniques and surroundings) may cause a relative change in the precipitation catchment of the rain gauge. The consistency of the precipitation data of such rain gauges needs to be examined. Double-mass analysis (5), also termed double-mass curve technique, compares the accumulated annual or seasonal precipitation at a given station with the concurrent accumulated values of mean precipitation for a group of the surrounding stations (*i.e.,* base stations). Since the past response is to be related to the present conditions, the data (accumulated precipitation of the station x, *i.e.,* ΣP_x and the accumulated values of the average of the group of the base stations, *i.e.,* ΣP_{av}) are usually assembled in reverse chronological order. Values of ΣP_x are plotted against ΣP_{av} for the concurrent time periods, Fig. 2.8. A definite break in the slope of the resulting plot points to the inconsistency of the data indicating a change in the precipitation regime of the station x. The precipitation values at station x at and beyond the period of change is corrected using the relation,

$$P_{cx} = P_x \frac{S_c}{S_a} \tag{2.10}$$

where, P_{cx} = corrected value of precipitation at station x at any time t

P_x = original recorded value of precipitation at station x at time t.

S_c = corrected slope of the double-mass curve

S_a = original slope of the curve.

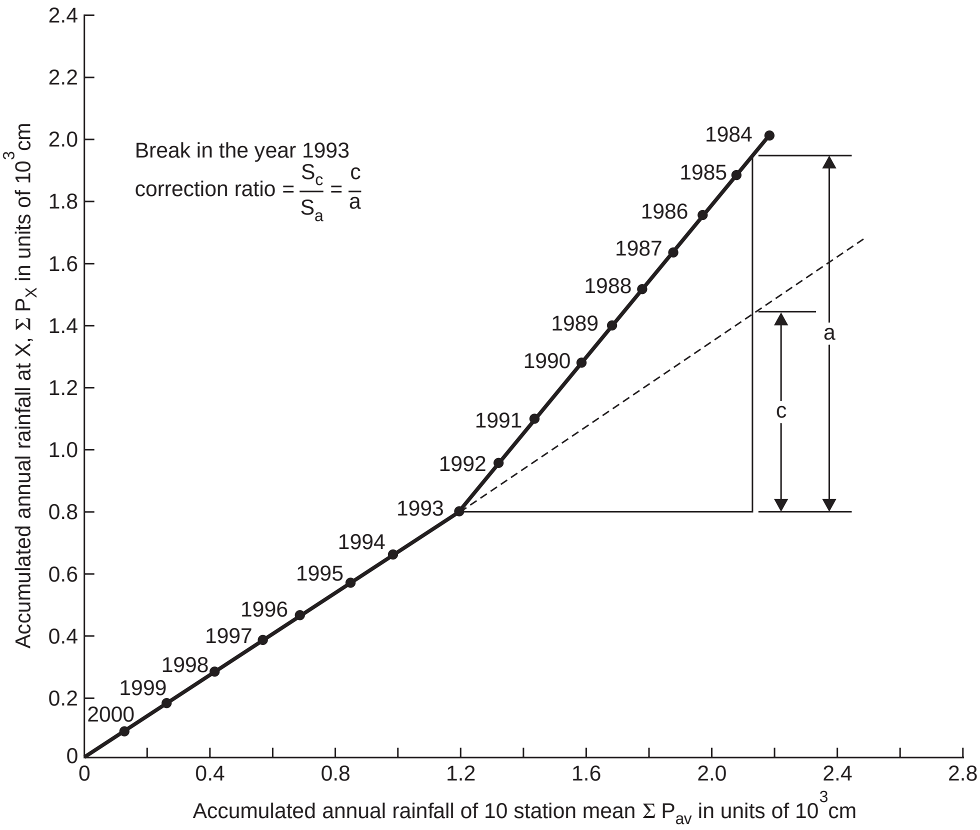

Fig. 2.8 Double-mass curve

Thus, the older records of station x have been corrected so as to be consistent with the new precipitation regime of the station x.

2.3.6. Presentation of Precipitation Data

Precipitation (or rainfall) data are presented as either a mass curve of rainfall (accumulated precipitation v/s time plotted in chronological order, Fig. 2.5) or a hyetograph (rainfall intensity v/s time). Mass curves of rainfall provide the information on the duration and magnitude of a storm. Intensities of rainfall at a given time can be estimated by measuring the slope of the curve at the specified time. The hyetograph derived from the mass curve, is usually represented as a chart. The area of a hyetograph represents the total precipitation received during the period.

2.3.7. Depth—Area—Duration (DAD) Analysis

Depth-area-duration (DAD) curves, Fig. 2.9, are plots of accumulated average precipitation versus area for different durations of a storm period. Depth—area—duration analysis of a storm is performed to estimate the maximum amounts of precipitation for different durations and over different areas. A storm of certain duration over a specified basin area seldom results in uniform rainfall depth over the entire specified area. The difference between the maximum rainfall depth over an area P_0 and its average rainfall depth $\overline{P}$ for a given storm, *i.e.*, $P_0 - \overline{P}$ increases with increase in the basin area and decreases with increase in the storm duration. The depth-area-duration curve is obtained as explained in the following example :

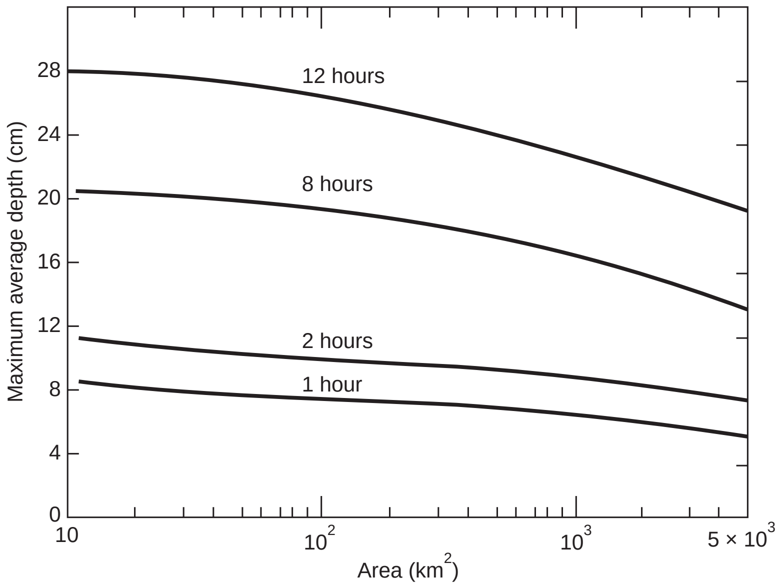

Fig. 2.9 DAD curves

Example 2.3 *The rainfall data of 8 rain gauge stations located in and around the basin, shown in Fig. 2.10, are as given in the following table :*

Cumulative rainfall in mm (Example 2.3)

Time in hours	*Gauge a*	*Gauge b*	*Gauge c*	*Gauge d*	*Gauge e*	*Gauge f*	*Gauge g*	*Gauge h*
2	*8*	*6*	*5*	*4*	*4*	*3*	*2*	*0*
4	*14*	*11*	*10*	*8*	*10*	*8*	*7*	*3*
6	*23*	*20*	*17*	*15*	*17*	*14*	*11*	*8*
8	*35*	*29*	*26*	*22*	*25*	*18*	*25*	*18*
10	*48*	*42*	*38*	*35*	*35*	*28*	*33*	*24*

The basin has an area of 5850 km^2. Obtain the depth-area-duration curves for 2, 4, and 6-hour durations.

Solution : Based on the rain gauge data at the end of the storm, isohyets and Thiessen polygons are drawn on the basin map (Fig. 2.10) as explained in Art. 2.3.3 and in Example 2.1. The isohyets (for 25, 35 and 45 mm) divide the entire basin into three zones, say zone *I*, zone *II*, and zone *III*. The polygon of any rain gauge station may lie in different zones of the basin. Each zone, at any time, will have a representative value of cumulative rainfall which would depend upon the rainfall depths of the influencing rain gauge stations at the same time and the areas of the corresponding polygons falling partly or fully into the zone. The zone I is made of part of the polygon of the rain gauge station *a* while the zone *II* is made up of part polygons of rain gauge stations *a, b, d*, and *g*, and full polygons of the rain gauge stations *c* and *e*. Similarly, zone *III* is made up of part polygons (of the raingauge stations *b, d* and *g*) and full polygons (of the rain gauge stations *f* and *h*). These details are given in the following table :

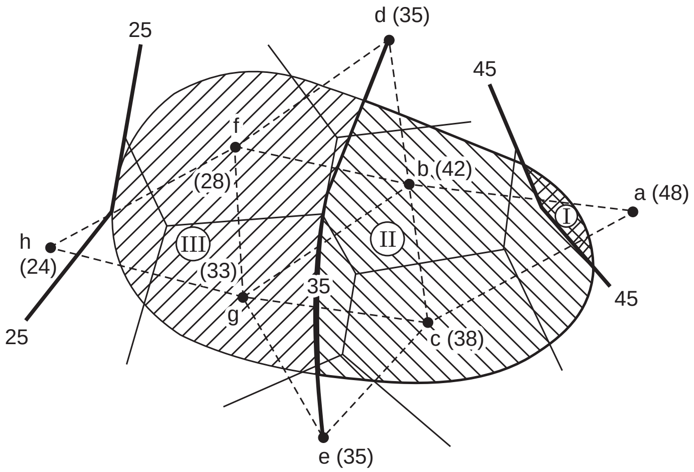

Fig. 2.10 Isohyets and Theissen polygons for Example 2.3

Area of Theissen polygons of different gauges in different zones (km²) (Example 2.3)

Zone	*Gauge a*	*Gauge b*	*Gauge c*	*Gauge d*	*Gauge e*	*Gauge f*	*Gauge g*	*Gauge h*	*Total*
I	100	0	0	0	0	0	0	0	100
II	350	1000	1200	100	50	0	200	0	2900
III	0	20	0	80	0	1400	1100	250	2850

The average cumulative depth of rainfall in any zone at any given time (since the beginning of the storm) is computed as

$$P_j = \frac{\sum_{i=1}^{m_i} (A_{ij})(P_i)}{\sum_{i=1}^{m_i} A_{ij}} \tag{2.11}$$

where, P_j = average cumulative rainfall depth at a given time for zone j,

A_{ij} = Part (or full) area of polygons of rain gauge station i whose polygon is falling partly (or fully) in the zone j,

P_i = cumulative rainfall depth at the same time, and

m_i = number of rain gauge stations influencing the average cumulative rainfall depth in zone j.

The values of cumulative rainfall depths for all the zones and at different times are computed and tabulated as below :

Cumulative average rainfalls in different zones in mm (Example 2.3)

Time	*Zone I*	*Zone II*	*Zone III*
2	8	5.45	2.4
4	14	10.55	7.2
6	23	18.28	12.39
8	35	27.9	20.89
10	48	40.1	29.87

Thereafter, cumulative average rainfalls for the progressively accumulated areas are worked out taking into account appropriate weights in proportion to the areas of the zones. For example, the cumulative average rainfall at a given time over all the three zones would be

$$P_{I+II+III} = \frac{A_I P_I + A_{II} P_{II} + A_{III} P_{III}}{A_I + A_{II} + A_{III}} \tag{2.12}$$

where, A_I, A_{II}, and A_{III} are the area of zones *I, II,* and *III,* respectively, and P_I, P_{II}, and P_{III} are the average cumulative depths of rainfall for zones *I, II* and *III,* respectively, and at the same specified time. The computed values are shown in the following Table:

Cumulative average rainfalls for progressively accumulated areas in mm (Example 2.3)

Time	*Zone I* *100 km*^{2}	*Zone (I + II)* *3000 km*^{2}	*Zone (I + II + III)* *5850 km*^{2}
2	8	5.54	4.01
4	14	10.67	8.98
6	23	18.44	15.49
8	35	28.14	24.61
10	48	40.36	35.25

Now the maximum average depths of rainfall for the desired durations of 2 hrs, 4 hrs and 6 hrs can be worked out for three areas of 100 km^2, 3000 km^2 and 5850 km^2 and tabulated as below and plotted as shown in Fig. 2.11.

Maximum depths of rainfall for accumulated areas in mm (Example 2.3)

2	13	12.22	10.64
4	25	21.92	19.76
6	34	29.69	26.27

It should be noted that the DAD curves need not be straight line as seen in Fig. 2.11 and the area axis may be logarithmic.

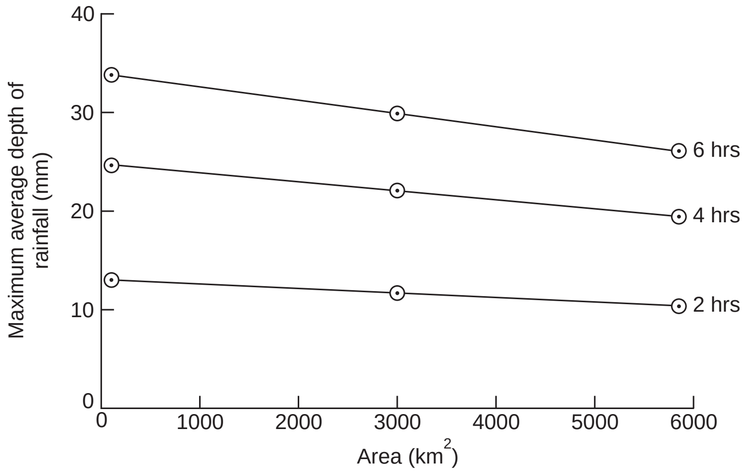

Fig. 2.11 DAD curves for Example 2.3

2.3.8. Mean Annual Rainfall

Mean annual rainfall for a given basin/catchment/area is computed as the arithmetic average of total yearly rainfall for several consecutive years. Mean annual rainfall obtained from rainfall records of about 30–40 years is expected to be true long-term mean annual rainfall with an error of about 2% and is acceptable for all types of engineering problems.

2.3.9. Probable Maximum Precipitation (PMP)

Probable maximum precipitation (PMP) is that magnitude of precipitation which is not likely to be exceeded for a particular basin at any given time of a year in a given duration. Thus, PMP would yield a flood which would have virtually no risk of being exceeded in that duration. Obviously, such a precipitation would occur under the most adverse combination of hydrological and meteorological conditions in the basin/area. Estimation of PMP is useful for obtaining the design flood for the purpose of designing hydraulic structures such as spillways failure of which would result in catastrophic damage to life and property in the surrounding region.

PMP can be estimated (6) using either meteorological methods (7) or statistical studies of rainfall data. One can derive a model (for predicting PMP) based on parameters (such as wind velocity and humidity *etc*.) of the observed severe storms over the basin and then obtain the PMP for maximum values of those parameters. Alternatively, one can also estimate the PMP by adopting a severe storm over a neighbouring catchment basin and transposing it to the catchment/basin under consideration. PMP estimates for North-Indian plains vary from about 37 to 100 cm for one-day rainfall.

2.4. ABSTRACTIONS FROM PRECIPITATION

Prior to rain water reaching the watershed outlet as surface runoff or stream flow, it has to satisfy certain demands of the watershed such as interception, depression storage, evaporation and evapotranspiration, and infiltration.

A part of precipitation may be caught by vegetation on the ground and subsequently get evaporated. This part of precipitation is termed intercepted precipitation or interception loss (which, incidentally, is the gain for the atmospheric water) which does not include through-fall (the intercepted water that drips off the plant leaves to join the surface runoff) and stemflow (the intercepted water that runs along the leaves, branches and stem of the plants to reach the ground surface). Interception loss primarily depends on storm characteristics, and type and density of vegetation.

Part of precipitation which fill up all the depressions on the ground before joining the surface runoff is called depression storage. It depends primarily on (*i*) soil characteristics, (*ii*) magnitude of depressions on the ground and (*iii*) antecedent precipitation which would decide soil moisture level.

Evaporation is the physical phenomenon by which a liquid is tranformed to a gas. The rate of evaporation of precipitation depends on (*i*) the vapour pressure of water, (*ii*) prevailing temperature, (*iii*) wind speed, and (*iv*) atmospheric pressure. Transpiration is a phenomenon due to which water received by the plant through its root system leaves the plant and reaches the atmosphere in the form of water vapour. Evaporation and transpiration are usually considered together as evapotranspiration (or consumptive use), Art. 3.7.

Infiltration is the passage of water from the soil surface into the soil and is different than percolation which is the gravity flow of water within the soil. Infiltration cannot continue unless percolation removes infiltered water from the surface soil. The maximum rate at which a soil can absorb water at a given time is known as infiltration capacity, f_c, expressed in cm/hour. The actual (prevailing) rate of infiltration f at any time is expressed as

$$f = f_c \qquad \text{if } i \geq f_c$$

and $$f = i, \qquad \text{if } i < f_c$$

where, i is the intensity of rainfall. Infiltration capacity depends on several factors such as soil characteristics including its moisture content, and vegetation or organic matter. Porosity is the most important characteristics of soil that affects infiltration. Forest soil, rich in organic matter, will have relatively higher infiltration capacity, largely because of the corresponding increase in porosity. Also, infiltration capacity for a given soil decreases with time from the beginning of rainfall primarily because of increasing degree of saturation of soil. Therefore, it is obvious that the infiltration capacity of a soil would vary over a wide range of values depending upon several factors. Typical values of f_c for sand and clay would be about 12 mm/h and 1.5 mm/h, respectively. A good grass cover may increase these values by as much as 10 times.

Difficulties in theoretical estimation of infiltration capacity due to its complexity have led to the use of infiltration indices. The simplest of these indices is the ϕ-index defined as the rate of rainfall above which the rainfall volume equals the runoff volume. This means that other initial losses (such as due to interception, evaporation and depression storage) are also considered as infiltration. The ϕ-index can be obtained from the rainfall hyetograph, Fig. 2.12. On this hyetograph is drawn a horizontal line such that the shaded area above this line equals the measured runoff.

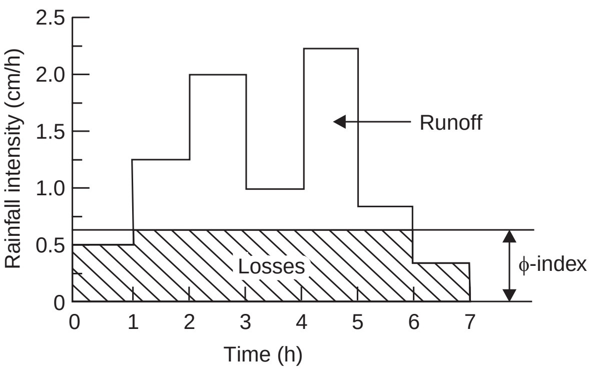

Fig. 2.12 ϕ-index

2.5. RUNOFF

Precipitation (or rainfall), after satisfying the requirements of evapotranspiration, interception, infiltration into the ground, and detention storage, drains off or flows off from a catchment basin as an overland flow (or surface runoff which includes precipitation falling on the stream system too) into a stream channel. Some part of the infiltrating water moves laterally through the upper layers of the soil and returns to the ground surface as interflow or subsurface runoff at some place away from the point of infiltration into the soil. Part of the infiltrating water percolates deep into the ground and joins the ground water storage. When water table intersects the stream channels of the catchment basin, some ground water may reach the surface or join the stream as ground water runoff, also called base flow or dry-weather flow. Thus, the runoff from a catchment includes surface runoff, subsurface runoff and base flow. The surface runoff starts soon after the precipitation and is the first to join the stream flow. Subsurface runoff is slower and joins the stream later. Depending upon the time taken by the subsurface runoff between the infiltration and joining the stream channel, it may be termed as prompt subsurface runoff or delayed subsurface runoff. The groundwater runoff is the slowest in joining the stream channel but, is responsible in maintaining low flows in the stream during dry season. Based on the time interval between the precipitation and runoff, the runoff is categorized as direct runoff (that enters the stream immediately after precipitation *i.e.,* surface runoff and subsurface runoff) and base flow (*i.e.,* ground water runoff). Runoff, thus is the response of a catchment to the precipitation reflecting the combined effects of the nature of precipitation, other climatic characteristics of the region, and the physiographic characteristics of the catchment basin.

Type, intensity, duration and areal distribution of precipitation over the catchment are the chief characeteristics of the precipitation that affect the stream flow. Precipitation in the form of rainfall is quicker to appear as stream flow than when it is in the form of snow. For the surface runoff to start, the intensity of rainfall (or precipitation) must exceed the infiltration capacity of the soil which decreases with the increase in the duration of rainfall. It is, therefore, obvious that a longer duration rainfall may produce higher runoff even if the intensity of rainfall is less but, of course, exceeding the infiltration capacity of the soil. Heavy rainfalls in the downstream region of the catchment will cause rapid rise in the stream levels and early peaking of the discharge. A rare occurrence of uniformly distributed rainfall may result in increased infiltration and, therefore, increased subsurface runoff and base flow resulting in slow rise in levels and delayed peaking of the discharge. Likewise, antecedent higher soil moisture conditions at the time of precipitation would hasten the rise in the stream levels.

Other climatic characteristics influencing the runoff are temperature, wind velocity, and relative humidity. These characteristics affect the evapotranspiration and thus influence the availability of the precipitation for runoff. Physiographic characteristics affecting the runoff have been discussed in Art. 2.7.

2.5.1. Runoff—Rainfall Relations

Yield of a river is the total quantity of water that flows in a stream during a given period. One can estimate the yield of a river by either correlating stream flow and rainfall or using some empirical equation or some kind of simulation.

By plotting measured stream flow *i.e.,* Runoff R versus corresponding rainfall P and drawing a best-fit line (or curve), one can establish an approximate relationship between rainfall and runoff. Alternatively, one can establish rainfall-runoff relationship using regression analysis.

Some empirical relations have been developed using the observed rainfall and runoff for streams of a given region. Such relations, therefore, have limitations to their applicability only to specific regions.

Based on his studies of small catchments (less than about 150 km^2) of U.P., Barlow expressed the runoff R as (8)

$$R = K_b\, P \tag{2.13}$$

in which, K_b is the runoff coefficient, Table 2.1, that depends on the catchment and monsoon characteristics.

Table 2.1 Barlow's runoff coefficient K_b in per cent (Developed for use in UP)

Class	*Description of catchment*	*Values of K_b (per cent)*		
		Season 1	*Season 2*	*Season 3*
A	Flat, cultivated and absorbent soils	7	10	15
B	Flat, partly cultivated, stiff soils	12	15	18
C	Average catchment	16	20	32
D	Hills and plains with little cultivation	28	35	60
E	Very hilly, steep and hardly any cultivation	36	45	81

Season 1 : Light rain, no heavy downpour
Season 2 : Average or varying rainfall, no continuous downpour
Season 3 : Continuous downpour

Strange analysed the rainfall and runoff data for the border region of Maharashtra and Karnataka and expressed runoff in terms of rainfall as (8)

$$R = K_s P \tag{2.14}$$

in which the runoff coefficient K_s depends on catchment characteristics only as given in Table 2.2.

Table 2.2 Strange's runoff coefficient K_s in per cent

(For use in border areas of Maharashtra and Karnataka)

Total monsoon rainfall (cm)	*Runoff coefficient K_s (per cent)*		
	Good Catchment	*Average Catchment*	*Bad Catchment*
25	4.3	3.2	2.1
50	15.0	11.3	7.5
75	26.3	19.7	13.1
100	37.5	28.0	18.7
125	47.6	35.7	23.8
150	58.9	44.1	29.4

For Western Ghats, Inglis and De Souza gave (8)

$$R = 0.85\, P - 30.5 \tag{2.15}$$

Similarly, for Deccan plateau, they gave

$$R = \frac{1}{254}\, P\,(P - 17.8) \tag{2.16}$$

In both these relations, R and P are in cm.

Khosla analysed monthly data of rainfall P_m (cm), runoff R_m (cm) and temperature T_m (°C) for various catchments of India and USA to obtain (9)

$$R_m = P_m - L_m \tag{2.17}$$

with $R_m \geq 0$ and $L_m = 0.48\, T_m$ for $T_m > 4.5$°C.

Here, L_m represents monthly losses in cm.

For $T_m \leq 4.5$°C, the monthly loss may be assumed as follows:

T_m°C	4.5	– 1	– 6.5
L_m (cm)	2.17	1.78	1.52

Annual runoff = $\sum_{i=1}^{12} R_{mi}$

This formula is found to give fairly good estimates of runoff.

Using hydrologic water-budget equation, runoff R can be expressed as

$$R = P - E_{et} - \Delta S \qquad (2.18)$$

where, E_{et} is actual evapotranspiration and ΔS is the change in soil moisture storage. Both these parameters depend upon the catchment characteristics and the regional climatic conditions. With the help of available data and computers (to handle large mass of data), one can develop a mathematical relationship (*i.e.,* model) incorporating interdependence of parameters involved. This watershed model is, then, calibrated *i.e.,* the numerical values of various coefficients of the model are determined using part of the data available. The remaining available data are used for validation of the model. Once the model is validated satisfactorily, it becomes a handy tool to predict the runoff for a given rainfall.

Another technique that can be used for computer simulation of watershed is artificial neural network (ANN) which is being increasingly employed for predicting such quantities which cannot be expressed in the form of mathematical expressions due to inadequate understanding of the influence of all the factors that affect the quantities.

2.6. STREAM FLOW

2.6.1. Flow Characteristics of a Stream

The flow characteristics of a stream depend upon (*i*) the intensity and duration of rainfall besides spatial and temporal distribution of the rainfall, (*ii*) shape, soil, vegetation, slope, and drainage network of the catchment basin, and (*iii*) climatic factors influencing evapotranspiration. Based on the characteristics of yearly hydrograph (graphical plot of discharge versus time in chronological order), one can classify streams into the following three types:

(*i*) Perennial streams which have some flow, Fig. 2.13, at all times of a year due to considerable amount of base flow into the stream during dry periods of the year. The stream bed is, obviously, lower than the ground water table in the adjoining aquifer (*i.e.,* water bearing strata which is capable of storing and yielding large quantity of water).

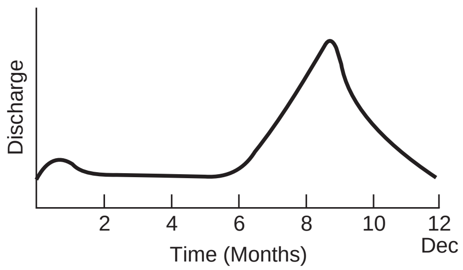

Fig. 2.13 Temporal variation of discharge in perennial streams

(*ii*) Intermittent streams have limited contribution from the ground water and that too during the wet season only when the ground water table is above the stream bed and, therefore, there is base flow contributing to the stream flow, Fig. 2.14. Excepting for some occasional storm that can produce short duration flow, such streams remain dry for most of the dry season periods of a year.

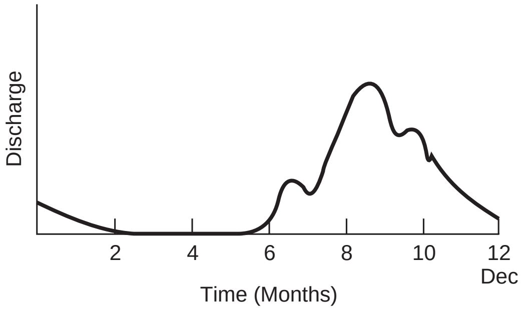

Fig. 2.14 Temporal variation of discharge in intermittent streams

(*iii*) Ephemeral streams do not have any contribution from the base flow. The annual hydrograph, Fig. 2.15, of such a stream shows series of short duration hydrographs indicating flash flows in response to the storm and the stream turning dry soon after the end of the storm. Such streams, generally found in arid zones, do not have well-defined channels.

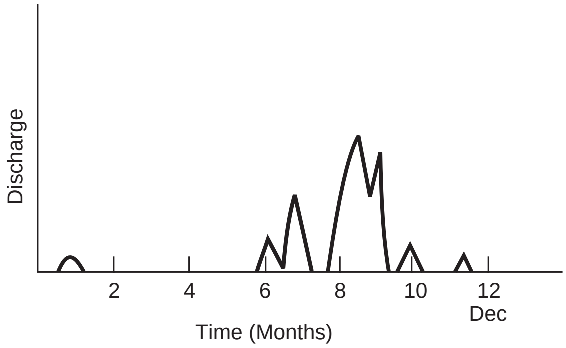

Fig. 2.15 Temporal variation of discharge in ephemeral streams

Streams are also classified as effluent (streams receiving water from ground water storage) and influent (streams contributing water to the ground water storage) streams. Effluent streams are usually perennial while the influent streams generally remain dry during long periods of dry spell.

2.6.2. Graphical Representation of Stream Flow

The stream flow data are usually recorded in tabular form. For analyzing these data, one has to prepare graphical plots of the stream flow data such as hydrograph, flow-duration curve, flow-mass curve or simply mass curve etc. Hydrograph is a graphical plot between discharge (on y-axis) and the corresponding time (days or months or even hours). Hydrograph analysis is dealt with separately in Art. 2.7. Description of the mass curve and flow-duration curve has been included in this article.

2.6.2.1. Flow-Mass Curves

Flow-mass curve or runoff-mass curve or inflow mass curve or simply mass curve is cumulative flow volume V versus time curve. The mass curve ordinate V (m^3 or ha.m or cumec-day) at any time t (in days or weeks or months) is given as

$$V = \int_{t_0}^{T} Q \,.\, dt \tag{2.19}$$

where, t_0 is the time at the beginning of the curve. Obviously, mass curve, Fig. 2.16, is an integral (*i.e.,* summation) curve of a given hydrograph, Art. 2.7. Also, slope of the mass curve at any point on the plot *i.e.,* dV/dt equals the rate of stream flow (*i.e.,* stream discharge) at that time. Mass curve is always a rising curve or horizontal (when there is no inflow or runoff added into the stream) and is a useful means by which one can calculate storage capacity of a reservoir to meet specified demand as well as safe yield of a reservoir of given capacity.

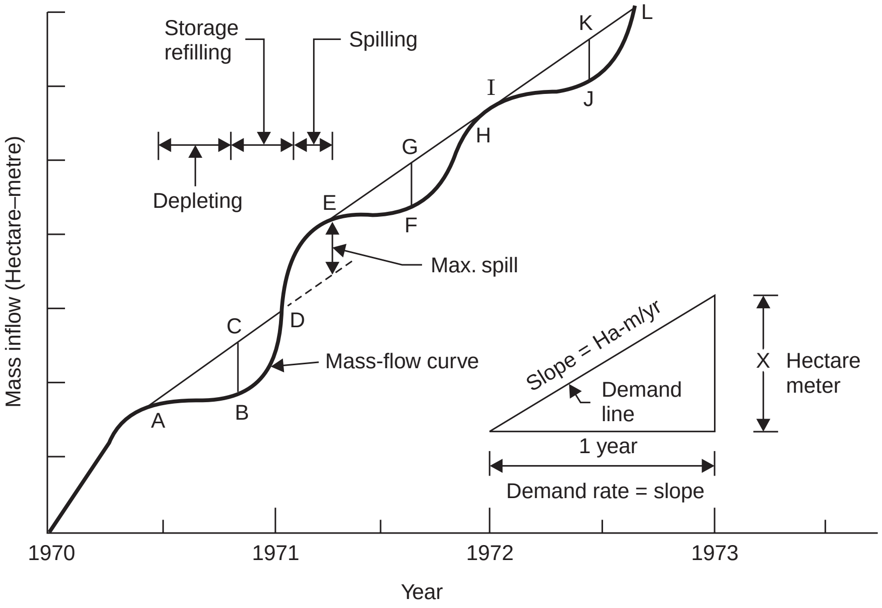

Fig. 2.16 Reservoir capacity from mass-flow curve

Slope of the cumulative demand curve (usually a line since the demand rate is generally constant) is the demand rate which is known. The reservoir is assumed to be full at the beginning of a dry period (*i.e.,* when the withdrawal or demand rate exceeds the rate of inflow into the reservoir) such as A in Fig. 2.16. Draw line AD (*i.e.,* demand line) such that it is tangential to the mass curve at A and has a slope of the demand rate. Obviously, between A and B (where there is maximum difference between the demand line and the mass curve) the demand is larger than the inflow (supply) rate and the reservoir storage would deplete. Between B and D, however, the supply rate is higher than the demand rate and the reservoir would get refilled. The maximum difference in the ordinates of the demand line and mass curve between A and D

(*i.e., BC*) represents the volume of water required as storage in the reservoir to meet the demand from the time the reservoir was full *i.e., A* in Fig. 2.16. If the mass curve is for a large time period, there may be more than one such duration of dry periods. One can, similarly, obtain the storages required for those durations (*EH* and *IL* in Fig. 2.16). The largest of these storages (*BC*, *FG* and *JK* in Fig. 2.16) is the required storage capacity of the reservoir to be provided on the stream in order to meet the demand.

For determining the safe yield of (or maintainable demand by) a reservoir of given capacity one needs to draw tangents from the apex points (*A, E* and *I* of Fig. 2.16) such that the maximum difference between the tangent and the mass curve equals the given capacity of the reservoir. The slopes of these tangents equals to the safe yield for the relevant dry period. The smallest slope of these slopes is, obviously, the firm dependable yield of the reservoir.

It should be noted that a reservoir gets refilled only if the demand line intersects the mass curve. Non-intersection of the demand line with the mass curve indicates inflow which is insufficient to meet the given demand. Also, the vertical difference between points *D* and *E* represents the spilled volume of water over the spillway.

The losses from reservoir (such as due to evaporation and seepage into the ground or leakage) in a known duration can either be included in the demand rates or deducted from inflow rates.

In practice, demand rates for irrigation, power generation or water supply vary with time. For such situations, mass curve of demand is superposed over the flow-mass curve with proper matching of time. If the reservoir is full at the first intersection of the two curves, the maximum intercept between the two curves represents the required storage capacity of the reservoir to meet the variable demand.

Example 2.4 The following Table gives the mean monthly flows of a stream during a leap year. Determine the minimum storage required to satisfy a demand (inclusive of losses) rate of 50 m^3/s.

Data and computation of mass curve (Example 2.4)

Month	*Mean monthly flow (m^3/s)*	*Days in month*	*Monthly flow volume (cumec-day)*	*Accumulated volume (cumec-day)*
January	60	31	1860	1860
February	50	29	1450	3310
March	40	31	1240	4550
April	28	30	840	5390
May	12	31	372	5762
June	20	30	600	6362
July	50	31	1550	7912
August	90	31	2790	10702
September	100	30	3000	13702
October	80	31	2480	16182
November	75	30	2250	18432
December	70	31	2170	20602

Solution: Actual number of days in a month (Col. 3 of the Table) are used for calculating monthly flow volume (Col. 4 of the Table). Mass curve of the accumulated flow versus time is shown plotted in Fig. 2.17. For the mass curve and demand rate, all months are assumed to be of equal duration *i.e.,* 30.5 days. A demand line (with a slope of line *PR*) is drawn tangential to the mass flow curve at *A*. Another line parallel to this line is drawn so that it is tangential to the mass-flow curve at *B*. The vertical difference *BC* (= 2850 cumec-day) is the required storage for satisfying the demand rate of 50 m³/s.

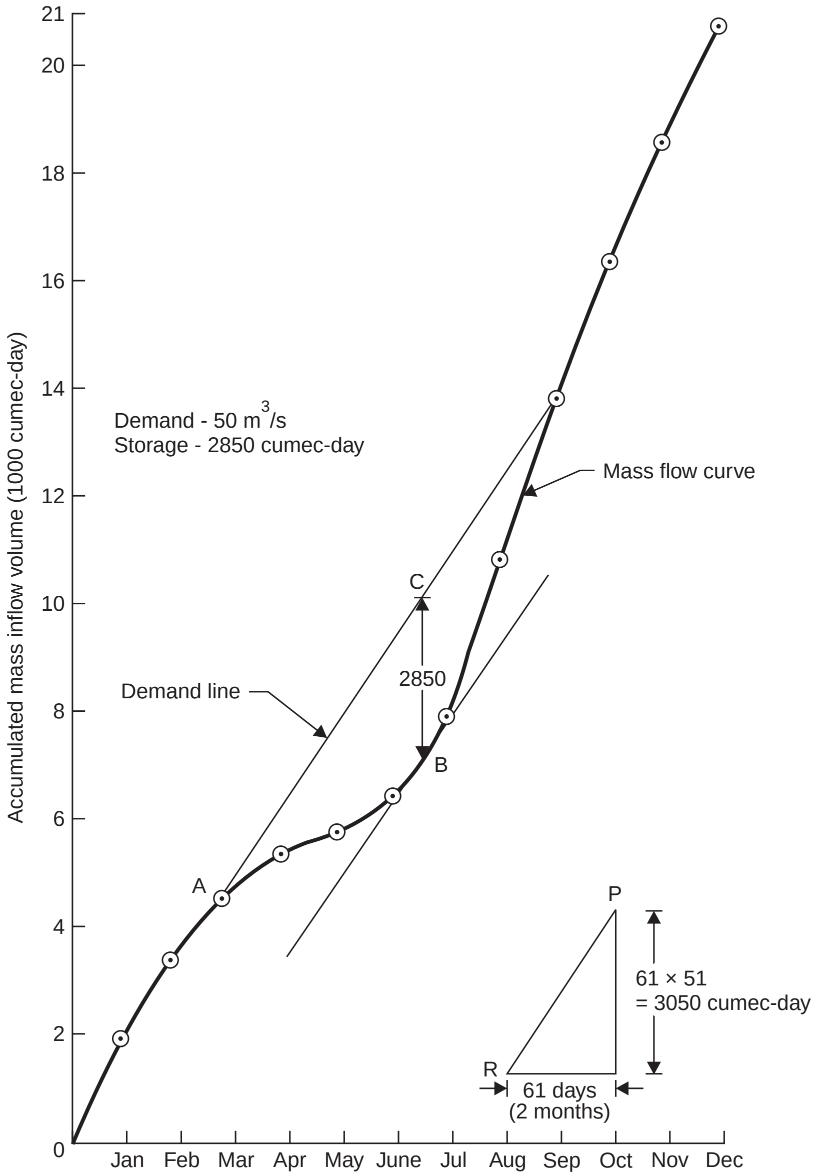

Fig. 2.17 Mass-flow curve and demand line for Example 2.4

2.6.2.2. Flow-Duration Curve

Flow-duration curve (or discharge-frequency curve) of a stream is a graphical plot of stream discharge against the corresponding per cent of time the stream discharge was equalled or exceeded. The flow-duration curve, therefore, describes the variability of the stream flow and is useful for

(*i*) determining dependable flow which information is required for planning of water resources and hydropower projects,

(*ii*) designing a drainage system, and

(*iii*) flood control studies.

For preparing a flow-duration curve, the stream flow data (individual values or range of values) are arranged in a descending order of stream discharges. If the number of such discharges is very large, one can use range of values as class intervals. Percentage probability P_p of any flow (or class value) magnitude Q being equalled or exceeded is given as

$$P_p = \frac{m}{N+1} \times 100(\%) \tag{2.20}$$

in which m is the order number of the discharge (or class value) and N is the number of data points in the list. The discharge Q is plotted against P_p to yield flow-duration curve, as shown in Figs. 2.18 and 2.19. The ordinate Q at any percentage probability P_p represents the flow magnitude in an average year that can be expected to be equalled or exceeded P_p perc ent of time and is termed as P_p% dependable discharge (or flow). The discharge Q in the flow-duration curve could be either daily average or monthly (usually preferred) average.

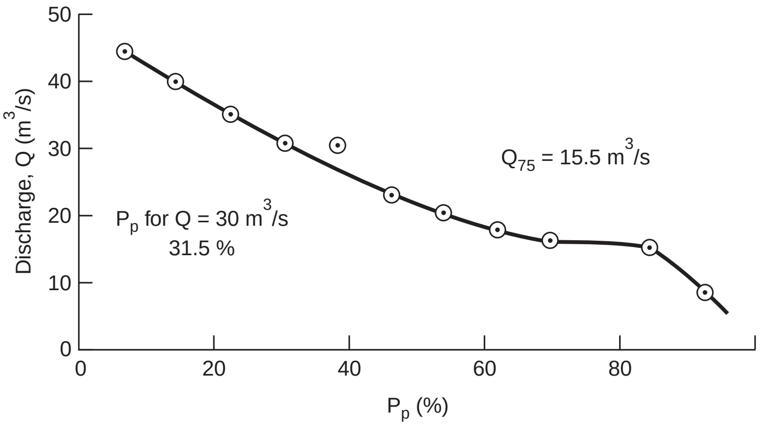

Fig. 2.18 Flow-duration curve for Example 2.5

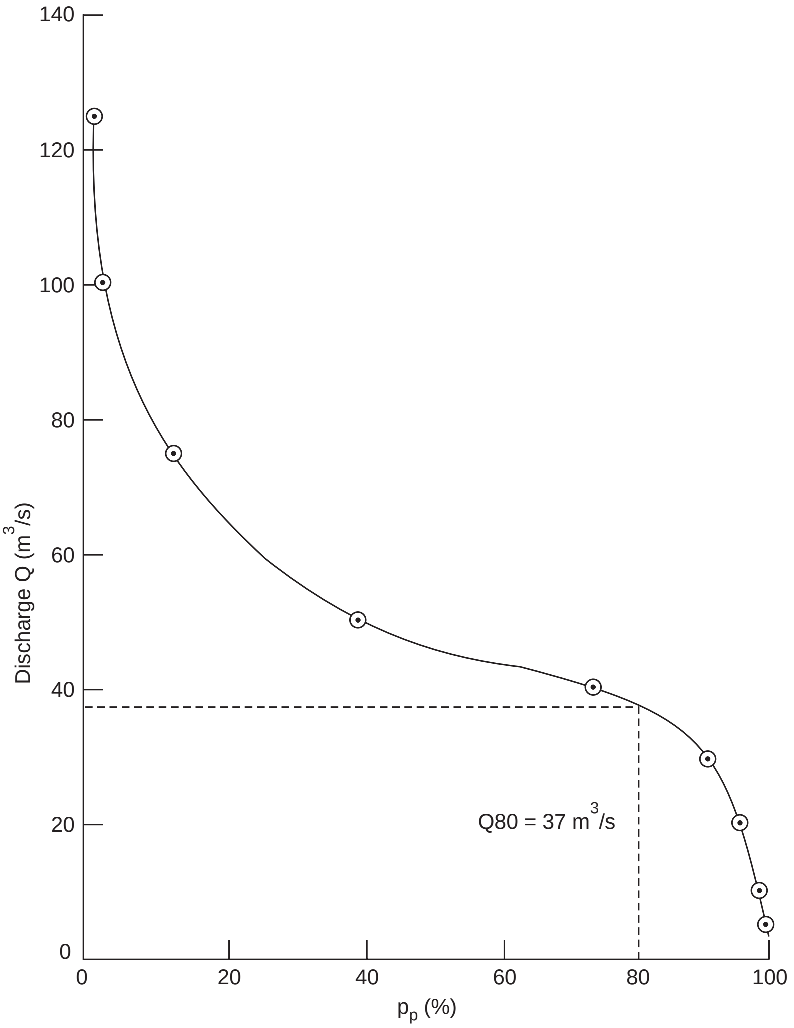

Fig. 2.19 Flow-duration curve for Example 2.6

Example 2.5 The observed mean monthly flows of a stream for a water year (June 01 to May 31) are as given in the first two columns of the following Table. Plot the flow-duration curve and estimate the flow that can be expected 75% of the time in a year (*i.e.*, 75% dependable flow, Q_{75}) and also the dependability (*i.e., P_p*) of the flow of magnitude 30 m^3/s.

Table : Data and computations for flow-duration curve (Example 2.5)

Month	*Observed flow* Q *(m³/s)*	*Flow (Q) arranged in descending order, (m³/s)*	*Rank m*	$P_p = \frac{m}{(N+1)} \times 100$ *(%)*
June	15	44	1	7.7
July	16	40	2	15.4
August	44	35	3	23.1
September	40	31	4	30.8
October	35	30	5	38.5
November	31	23	6	46.2
December	30	21	7	53.8
January	21	18	8	61.5
February	23	16	9	69.2
March	18	15	11	84.6
April	15	15	11	84.6
May	8	8	N = 12	92.3

Solution: The flow-duration curve (Q (col. 3) v/s P_p (col. 5)) is as shown in Fig. 2.18. From the curve, one can obtain

$$Q_{75} = 15.5 \text{ m}^3/\text{s}$$

And the dependability (*i.e.*, P_p) of the flow of magnitude 30 m³/s = 31.5%.

Example 2.6 Column 1 of the following Table gives the class interval of daily mean discharges (m³/s) of a stream flow data. Columns 2, 3, 4, and 5 give the number of days for which the flow in the stream belonged to that class in four consecutive years. Estimate 80% dependable flows for the stream.

Data and computations for flow-duration curve (Example 2.6)

Daily mean discharge (m³/s)	*Number of days the flow in the stream belonged to the class interval*				*Total of cols. 2, 3, 4, and 5*	*Cumul-ative total, m*	$P_p = \frac{m}{(N+1)} \times 100$ *(%)*
	1995	1996	1997	1998			
1	2	3	4	5	6	7	8
150-125	0	1	4	2	7	7	0.48
124.9-100	2	5	8	4	19	26	1.78
99.9-75	20	52	40	48	160	186	12.72
74.9-50	95	90	100	98	383	569	38.92
49.9-40	140	125	117	124	506	1075	73.53
39.9-30	71	75	65	50	261	1336	91.38
29.9-20	15	10	20	21	66	1402	95.90
19.9-10	15	8	10	18	51	1453	99.38
9.9-5	7	0	1	0	8	1461	99.93
Total	365	366	365	365	N = 1461		

Solution: Column 6 of the Table shows the total number of days in a period of 4 years for which the discharge in that class (Col. 1) was flowing in the stream. Column 7 gives the cumulative total of the values of column 6. The probability of flow in the class interval being equalled or exceeded is obtained from Eq. (2.20) and tabulated in column 8. The smallest value of discharge in a class (Col. 1) is plotted against P_p (col. 8) as shown in Fig. 2.19. From this figure, one can obtain the desired value of 80% dependable flow as 37 m^3/s.

2.7. HYDROGRAPHS

Consider a concentrated storm producing a short-duration and reasonably uniform rainfall of duration t_r over a watershed. Part of this rainfall is retained on the land surface as detention storage. Yet another part of the rainfall infiltrates into the soil. The remaining part of the rainfall is termed rainfall excess (or effective rainfall) that is neither retained on the land surface nor infiltrated into the soil. This effective rainfall reaches the watershed outlet after flowing over the watershed surface. The flow over the watershed surface builds up some storage both in the overland and channel flow phases. This storage gradually depletes when the rainfall has ceased. There is, thus, a time lag between the occurrence of rainfall over a watershed and the time when the rainfall excess reaches the gauging station at the watershed outlet in the form of direct runoff. The runoff measured at the gauging station would typically vary with time as shown by the curve AMCE in the graph (known as hydrograph) of Fig. 2.20. The hydrograph is, therefore, the response of a given catchment (or watershed) to a rainfall input and can be regarded as an integral expression of the physiographic and climatic characteristics of the region that decide the rainfall-runoff relationship. It comprises all three phases of runoff, viz., surface runoff, interflow and base flow. Therefore, two different storms over the same watershed would, invariably, produce hydrographs of different shapes (*i.e.,* peak rate of discharge, time base *etc.*) Likewise, identical storms over different watersheds would also produce different hydrographs.

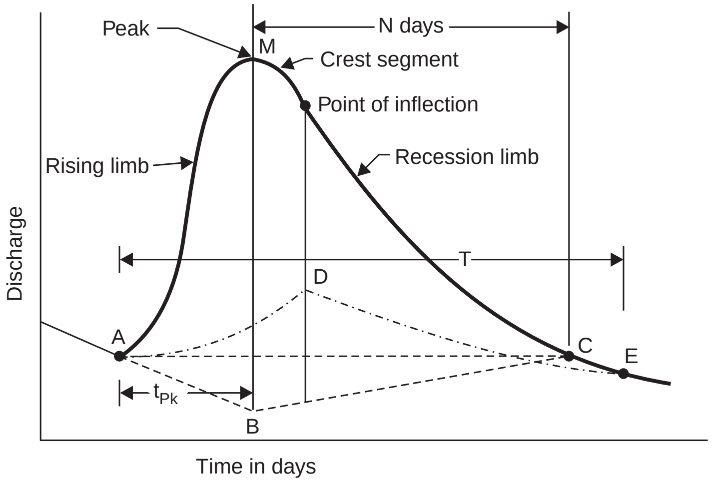

Fig. 2.20 Base flow separation

The inter-relationship among rainfall, watershed and climatic characteristics is, generally, very complex and so is the shape of the resulting hydrograph (having kinks, multiple peaks *etc.*) much different from the simple single-peaked hydrograph of Fig. 2.20.

A single-peaked hydrograph, Fig. 2.20, consists of (*i*) a rising limb, (*ii*) the crest segment, and (*iii*) the recession or falling limb. The rising limb (or concentration curve) of a hydrograph represents continuous increase in discharge (or runoff) at the watershed outlet. During the initial periods of the storm, the increase in runoff is rather gradual as the falling precipitation has to meet the initial losses in the form of high infiltration, depression storage and gradual building up of storage in channels and over the watershed surface. As the storm continues, losses decrease with time and more and more rainfall excess from distant parts of the watershed reaches the watershed outlet. The runoff, then, increases rapidly with time. When the runoff from all parts of the watershed reaches the watershed outlet simultaneously, the runoff attains the peak (*i.e.,* maximum) value. This peak flow is represented by the crest segment of the hydrograph. The recession limb of the hydrograph starts at the point of inflection (*i.e.,* the end of the crest segment) and continues till the commencement of the natural ground water flow.

The recession limb represents the withdrawal of water from the storage (in the channels and over the watershed surface) that was built-up during the initial periods of the storm. The point of inflection *i.e.,* the starting point of recession limb represents the condition of maximum storage in the channels and over the watershed surface. After the cession of the rainfall, this storage starts depleting. Therefore, the shape of the recession limb depends only on the watershed characteristics and is independent of the storm characteristics. The shape of the hydrograph is mainly influenced by the physiographic characteristics of the watershed (described briefly in the following paragraph) and the climatic characteristics of the region (dealt with in Art. 2.5).

Physiographic characteristics of a catchment basin influencing the runoff and, therefore, the shape of the hydrograph include area, shape, elevation, slope, and orientation of the basin besides the type of soil, land use, and drainage network. All other conditions remaining the same, a larger basin area results in smaller peak flow with a larger time base of the hydrograph (*i.e.,* relation depicting variability of stream discharge with time in chronological order) and better sustainable minimum flow in the stream due to the possibility of delayed subsurface runoff and base flow.

Different shapes (elongated or broad) of a catchment basin can be represented by the form factor defined as the ratio of average width to the axial length of the basin. In an elongated basin (form factor < 1), the precipitation falling at the farthest upstream end of the basin will take longer time to reach the downstream outlet end of the basin. This would, therefore, result in larger time base of the hydrograph with lesser peak flow. Catchment basin with higher slope would, obviously, hasten rise in the stream levels and peaking of the stream flow. Orientation of basins with respect to the sun would decide the magnitude of evapotranspiration and, thus, influence the runoff. Temperature, precipitation and other climatic characteristics of a region are influenced also by the mean elevation which, therefore, affects the runoff indirectly.

Soil characteristics affect the infiltration capacity and, hence, the runoff. A soil with high porosity increases the infiltration and, therefore, reduces the peak flow in the stream. Similarly, a forest area has larger capacity to retain water in its densely vegetated surface and, hence, reduces the peak flow (*i.e.,* flooding) in the stream.

Over a period of time a network of natural rivulets (*i.e.,* smaller stream channels) develop in a drainage basin. These channels act as tributaries to the main stream of the drainage basin. A well-developed network of these smaller channels (*i.e.,* drains) collect the precipitation and transport it quickly to the outlet end of the basin without giving much opportunity to the precipitation to infiltrate into the ground. Therefore, peak flows in the stream would be higher. Minimum flows, however, are likely to be lower due to lesser infiltration.

2.7.1. Hydrograph Analysis

Hydrograph represents the temporal variation of total runoff at a gauging point in a stream. The total runoff includes both direct and ground water runoff. In hydrologic studies, one needs to establish a suitable relationship between surface flow hydrograph and the effective rainfall. Surface flow (or direct runoff) hydrograph is obtained by subtracting base flow (*i.e.,* ground water runoff) from the total storm (or runoff) hydrograph. Division of a total runoff hydrograph into direct and ground water runoffs for subsequent analysis to analyse hydrologic problems is termed hydrograph separation or hydrograph analysis. There is no ready basis for differentiating between direct and ground water runoffs which have been defined rather arbitrarily. Therefore, the method of hydrograph separation too is arbitrary only.

For the purpose of unit hydrograph (Art. 2.7.2) theory, the hydrograph separation should be such that the time base of the direct runoff remains almost the same for different storms of the catchment basin. This can be attained by terminating the direct runoff at a fixed time after the time of occurrence of the peak of the hydrograph. The time interval N (in days) from the instant of occurrence of the peak to the time marking the end of the direct runoff (point C in Fig. 2.20) is empirically expressed as (9)

$$N = bA^{0.2} \tag{2.21}$$

Here, A is the drainage area in km^2 and b is a coefficient ranging from 0.8 to 0.85. The point A in Fig. 2.20 marks the beginning of the direct runoff and is identified easily as the point at the beginning of the rising limb where there is sharp increase in the runoff rate. Line AC provides the simplest method of base flow separation. The ordinates of hydrograph with respect to line AC, therefore, give the magnitudes of the direct runoff at the relevant time.

The most widely used method for hydrograph separation cosists of extending the recession (or base flow) curve existing before the commencement of the direct runoff (due to the storm under consideration) till it intersects the ordinate passing through peak of the hydrograph (at point B). Line segments AB and BC demarcate the separation between the surface runoff and base flow. The method is based on the reasoning that as the stream level rises there is flow from the stream into the banks of the stream. Therefore, the base flow (into the stream) should continuously decrease until the stream level starts falling and bank storage begins to return into the stream. It is, however, assumed that the decrease in base flow (*i.e., AB*) conforms to the usual recession existing prior to the storm.

In yet another method of base flow separation, the base flow recession curve (after the depletion of flood water as at E) is extended backward till it intersects the ordinate through the point of inflection on the recession limb at D. Points A and D are joined arbitrarily by a smooth curve. This method is preferred when the ground water contribution is expected to be significant and likely to reach the stream quickly.

After hydrograph separation, one can obtain surface (or direct) runoff hydrograph (DRH).

2.7.2. Unit Hydrograph

A unit hydrograph (or unit-Graph) is the direct runoff hydrograph resulting from one centimeter (or one millimeter or one inch) of excess rainfall generated uniformly over a catchment area at a constant rate for an effective duration (1). The unit hydrograph for a catchment basin is the direct runoff hydrograph produced by a unit (usually 1 cm) rainfall excess from a storm of

D-hour duration and, therefore, is the lumped repsonse of the basin to the storm. The unit hydrograph is a simple linear model that is most widely used for obtaining the surface runoff hydrograph resulting from any amount of excess rainfall. The physical characteristics of a catchment basin (shape, size, slope *etc*.) remain invariant to a large extent. Therefore, one may expect considerable similarity in the hydrographs of different storms of similar rainfall characteristics. This forms the basis of the unit hydrograph first proposed by Sherman (10). The unit hydrograph is a typical hydrograph for a catchment basin and is so called because the runoff volume under the hydrograph is adjusted to 1 cm (or 1 mm or 1 inch) equivalent depth over the basin. It should, however, be noted that the variable characteristics of storms (such as rainfall duration, time-intensity pattern, areal distribution, magnitude of rainfall) do cause variations in the shape of the resulting hydrographs. Therefore, it would be incorrect to imply that only one typical hydrograph would suffice for any catchment basin. The following basic assumptions are inherent in the unit hydrograph theory (1) :

1. The excess rainfall has a constant intensity ($1/D$ cm/hr) within effective storm duration of D hours.

2. The excess rainfall (giving rise to 1 cm depth of runoff) is uniformly distributed throughout the entire catchment basin.

3. The base time of direct runoff hydrograph (*i.e.,* the duration of the direct runoff) resulting from an excess rainfall of given duration is constant.

4. The ordinates of all direct runoff hydrographs of a common base time are directly proportional to the total amount of direct runoff represented by each hydrograph. This means that a rainfall excess of r cm due to a storm of duration D hours in a catchment basin will produce a direct runoff hydrograph whose ordinates would be r times the corresponding ordinates of a D-hour unit hydrograph of the basin.

5. For a given catchment basin, the hydrograph, resulting from a given excess rainfall, reflects the unchanging characteristics of the catchment basin.

2.7.3. Derivation of Unit Hydrograph

The unit hydrograph is best derived from the hydrograph of an isolated storm (*i.e.,* occurring individually) of reasonably uniform intensity during its duration (of desired magnitude) of occurrence covering the entire catchment basin, and resulting into a relatively large runoff volume. The hydrograph of such a storm of duration, say, D hour is separated (Art. 2.7.1) and the direct runoff hydrograph (*DRH*) is obtained. From this *DRH*, volume of direct runoff (area under the hydrograph) is determined in terms of the depth (*i.e.,* runoff depth) over the catchment basin. The ordinates of the *DRH* are divided by the runoff depth to obtain the ordinates of the D-hour unit hydrograph (for unit runoff depth). A unit hydrograph derived from a single storm may not be truly representative unit hydrograph for the given catchment basin. Therefore, for a given catchment basin, a number of unit hydrographs of a specified duration (say D hours) are derived using as many isolated storm hydrographs (which will not be identical due to variable storm characteristics) caused by storms of almost the same duration (0.9 to 1.1 D hours) and also satisfying the above-stated requirements as much as possible or feasible. These are, then, plotted and an average unit hydrograph, Fig. 2.21, is obtained. The ordinates of the averaged unit hydrograph should not be an arithmetic average of superposed ordinates because if peaks of different unit hydrographs do not occur at the same time, the average peak so determined will be lower than any individual peak (Fig. 2.21). The proper procedure is to compute average peak flow, average of times of occurrences of individual peak flows, and the average of base lengths. Thereafter, the average unit hydrograph is sketched so as to conform to the shape of other unit hydrographs, passing through the average peak at the averaged time to peak and having base length equal to the averaged base length. The volume of direct runoff for this unit hydrograph should, obviously, be unit depth and any departure from unit depth is corrected by suitable adjustments (including adjusting the peak) to the unit hydrograph, Fig. 2.21. It is usual to draw average effective rainfall hydrograph of unit depth alongside the unit hydrograph to indicate the duration of rainfall producing the unit hydrograph.

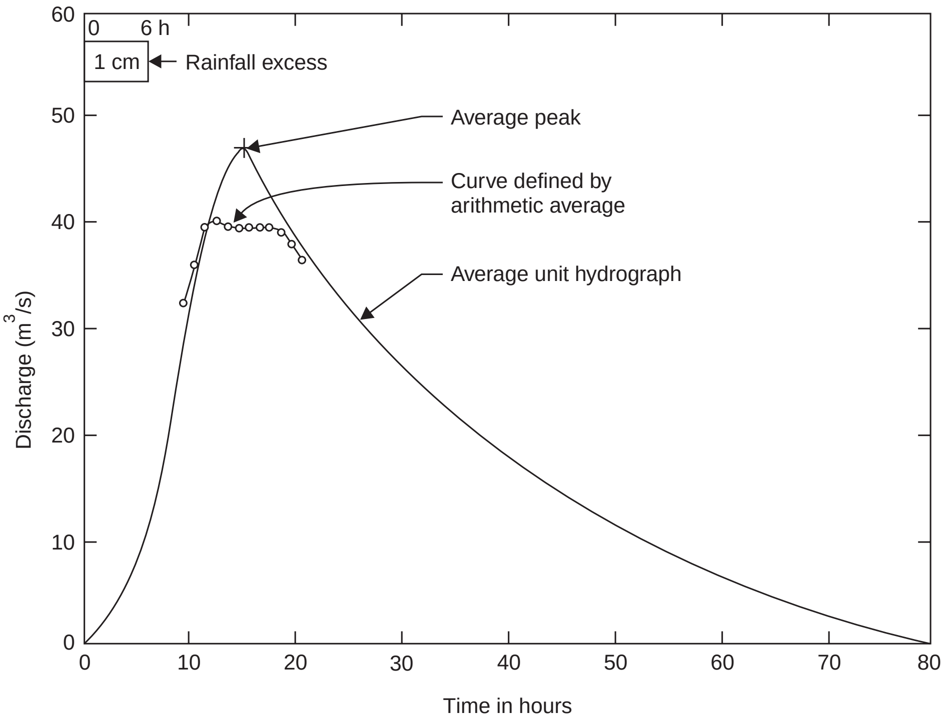

Fig. 2.21 Average unit hydrograph

It is rare to find a suitable storm of desired characteristics (as outlined earlier) for the derivation of a unit hydrograph. More often than not, one is required to derive unit hydrographs using the data of two or more closely-spaced storms (*i.e.,* complex storms) resulting into a hydrograph having two or more peaks. In such cases, it is necessary to separate the runoff caused by individual bursts of rainfall and, then, separate direct runoff from base flow. If individual bursts of rain in the complex storm result in well-defined peaks, it is possible to separate the hydrographs (*i.e.,* separating the direct and base runoff). Averaging of the unit hydrographs would minimize the errors in the separation process. Consider the complex hydrograph shown in Fig. 2.22. Separation of runoff caused by the two bursts of rainfall is accomplished by extending (in a way similar to the recession curve *TF*) the small segment of recession (*PQ*) between the two peaks. Base flow separation can now be completed by drawing *ABC* and *EF*, Fig. 2.22. Obviously, the area *ABCEFTRQPA* equals the direct runoff volume from rainfalls *J* and *K*. One can now proceed to derive the unit hydrograph in the usual manner.

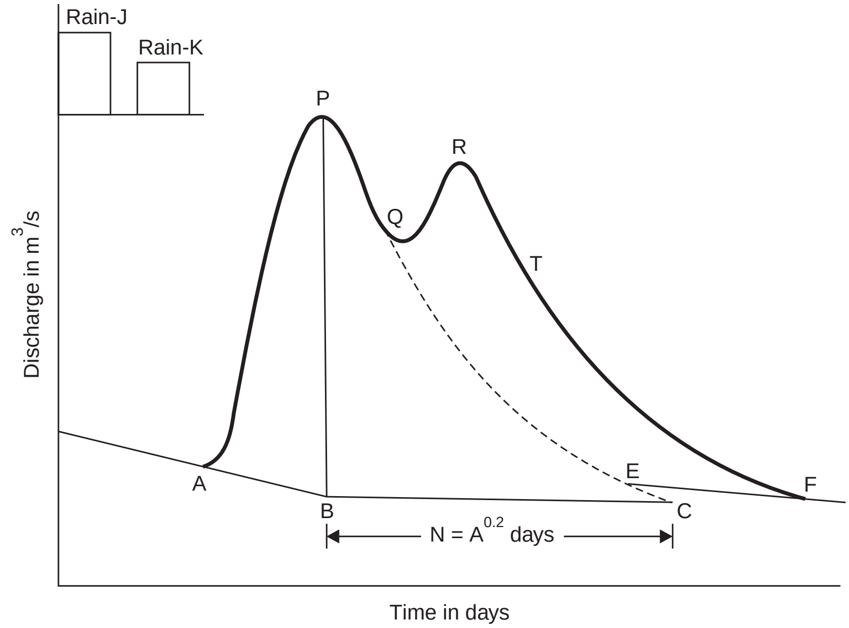

Fig. 2.22 Analysis of complex hydrographs

A unit hydrograph can also be developed through sucessive approximations. A suitable D-hour unit hydrograph is assumed and used to obtain the storm hydrograph corresponding to a given rainfall of the duration D hours which should, obviously, match with the observed storm hydrograph for the given rainfall. If the two storm hydrographs do not match to the acceptable accuracy, the assumed unit hydrograph is suitably modified and the process repeated until a unit hydrograph that gives the best matching is obtained.

2.7.4. Conversion of the Duration of Unit Hydrograph

Preparing unit hydrographs for a given catchment and covering wide range of durations is usually not possible due to lack of data. Therefore, one needs to convert an existing or derived unit hydrograph for one storm duration (say, D hours) to another (say, nD hours) that could be either shorter (to better cope with spatial and intensity variations) or longer (to reduce the computational work and also recognizing the coarseness of the available data). The following two methods are used for the conversion of the duration of a given unit hydrograph:

1. Method of superposition
2. Summation curve (or S-curve) method.

2.7.4.1. Method of Superposition

If n number of D-hour unit hydrographs, each one separated from the previous one by D hour, are added, one would obtain a characteristic hydrograph for n units of rainfall excess and nD-hour duration. Dividing the ordinates of this characteristic hydrograph by n would, obviously, yield a unit hydrograph (with unit rainfall excess) of duration equal to nD hours. Figure 2.23 illustrates the method of superposition which, obviously, requires n to be an integer. When n is not an integer, the summation curve (or S-curve) method is to be used.

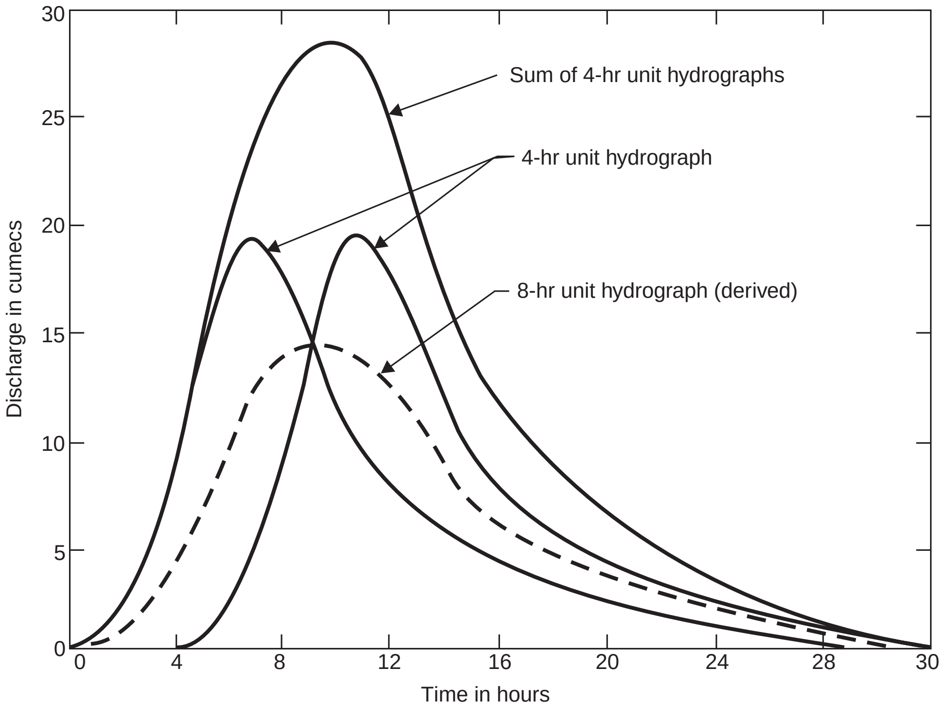

Fig. 2.23 Derivation of unit hydrograph of duration 2*t* from *t-h UH*

2.7.4.2. Summation Curve (or S-Curve) Method

The *S*-curve (or *S*-hydrograph) is a direct runoff hydrograph resulting from a continuous effective rainfall of uniform intensity. The S-curve is obtained by adding together a series of unit hydrographs of, say, *D*-hour duration each lagged by *D* hour in relation to the preceding one (Fig. 2.24). The intensity of effective rainfall for this S-hydrograph would, therefore, be 1/*D* cm/hr. This means that each S-curve applies to a specific duration *D* within which one unit of direct runoff is generated. If the time base of the D-hour unit hydrograph is T hour (Fig. 2.20), then a continuous rainfall producing one unit of runoff in every period of *D* hours would yield a constant outflow at the end of *T* hours. Therefore, one needs to combine only *T*/*D* unit hydrographs to produce an *S*-curve whose equilibrium flow rate Q_e (in m^3/s) would be the product of area of the catchment basin and the intensity of the effective rainfall (or rainfall excess) *i.e.,* 1/*D* cm/hr $\left(=\frac{1}{100\,D}\ \text{m/hr}\right)$.

$$\therefore \qquad Q_e = (A \times 10^6)\left(\frac{1}{100\,D}\right)\ \text{m}^3/\text{hr}$$

$$= \left(\frac{A}{D} \times 10^4\right)\ \text{m}^3/\text{hr} \qquad (2.22)$$

where, A = area of the catchment basin in km^2, and

D = duration in hours of the effective rainfall of the unit hydrograph.

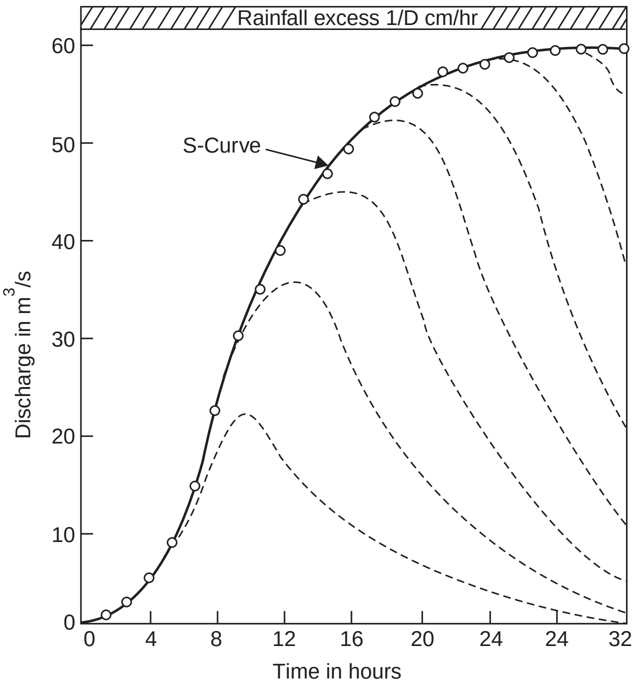

Fig. 2.24 Illustration of *S*-hydrograph (or *S*-curve)

Alternatively,
$$Q_e = (A \times 10^6)\left(\frac{1}{100\,D}\right)\left(\frac{1}{60}\times\frac{1}{60}\right)\ \text{m}^3/\text{s}$$

$$Q_e = 2.78\,\frac{A}{D}\ \text{m}^3/\text{s} \qquad ...(2.23)$$

The *S*-curve ordinates sometimes oscillate at and around the equilibrium discharge. This may be due to the unit hydrograph (used for the derivation of the *S*-curve) which might have been derived from storms not satisfying the requirements of an ideal storm for the derivation of the unit hydrograph. An average *S*-curve can, however, be still drawn so as to attain the equilibrium discharge rather smoothly. The *S*-curve, obtained from a *D*-hour unit hydrograph of a catchment basin, can be used to obtain unit hydrograph (for the same catchment, of course) of another duration, say, *D′* hour as explained in the following steps :

1. Draw two S-curves (obtained from a *D*-hour unit hydrograph) with their initial points displaced on time axis by *D′* hour, Fig. 2.25.

2. The effective rainfall hyetographs (*ERH*) producing these two *S*-curves are also drawn in the same figure. The two *ERH* are also displaced by *D′* hour. The difference between these two effective rainfall hyetographs represents a storm of duration *D′* with an intensity of 1/*D* cm/hr and, hence, a rainfall of magnitude *D′*/*D* cm.

3. The difference between the ordinates of the two *S*-curves at any time [*i.e.* $S(t)$-$S(t - D')$] gives the ordinate of a direct runoff hydrograph at that time, Fig. 2.25. This hydrograph is, obviously, for the storm of duration *D′* with an intensity of 1/*D* cm/hr having rainfall excess of *D′*/*D* cm which is the runoff volume.

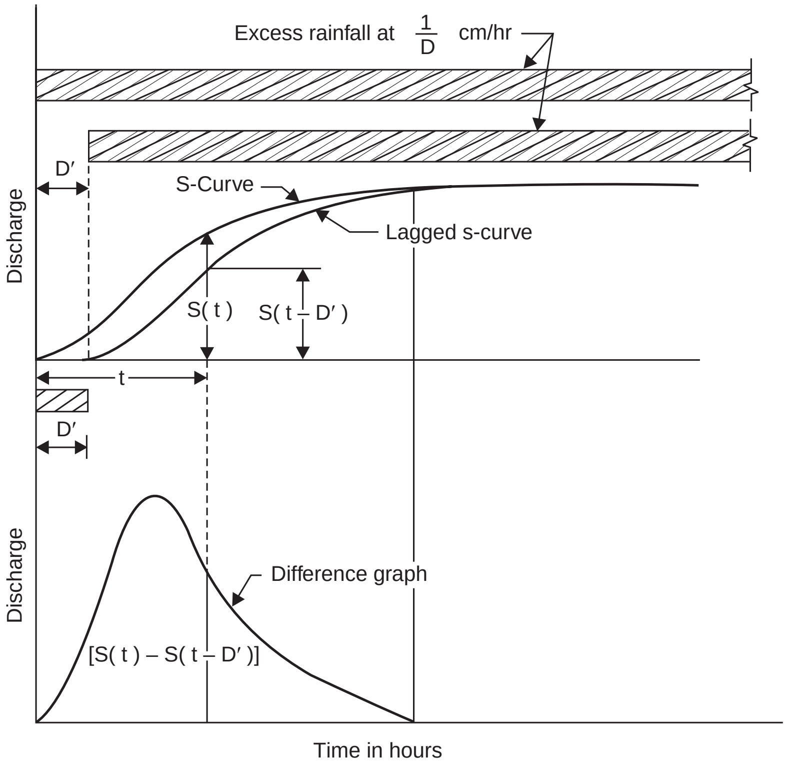

Fig. 2.25 Derivation of *D′* hour *UH* by *S*-curve method

4. Compute the ordinates of the D'-hour unit hydrograph by multiplying the *S*-curve differences [*i.e.,* $S(t) - S(t - D')$] with the ratio D/D'.

Earlier, it has been stated that one requires combining T/D unit hydrographs suitably for obtaining a *S*-curve. However, one can construct *S*-curve without requiring to tabulate and adding T/D unit hydrographs with successive time lags of D hours. The difference of two-*S*-curves (derived from a D-hour unit hydrograph) lagged by D hour itself is nothing but the D-hour unit hydrograph itself. Therefore, the ordinate $U(t)$ of a D-hour unit hydrograph at any time t is given as

$$U(t) = S(t) - S\,(t - D)$$

or
$$S(t) = U(t) + S(t - D) \tag{2.24}$$

The term $S(t - D)$ is called the *S*-curve addition which is an ordinate of *S*-curve itself but at time $(t - D)$. It may be noted that for $t \le D$.

$$S(t) = U(t) \tag{2.25}$$

And for $t > D$, one has to use Eq. (2.24) for constructing *S*-curve.

2.7.4.3. Instantaneous Unit Hydrograph (IUH)

As the duration D of unit hydrograph is reduced, the intensity of rainfall excess *i.e.,* $1/D$ increases, and the unit hydrograph becomes more skewed with its peak occurring earlier, Fig. 2.26. The fictitious case of unit hydrograph of zero duration is known as instantaneous unit hydrograph which represents the direct runoff from the catchment due to an instantaneous precipitation of the rainfall excess volume of 1 cm.

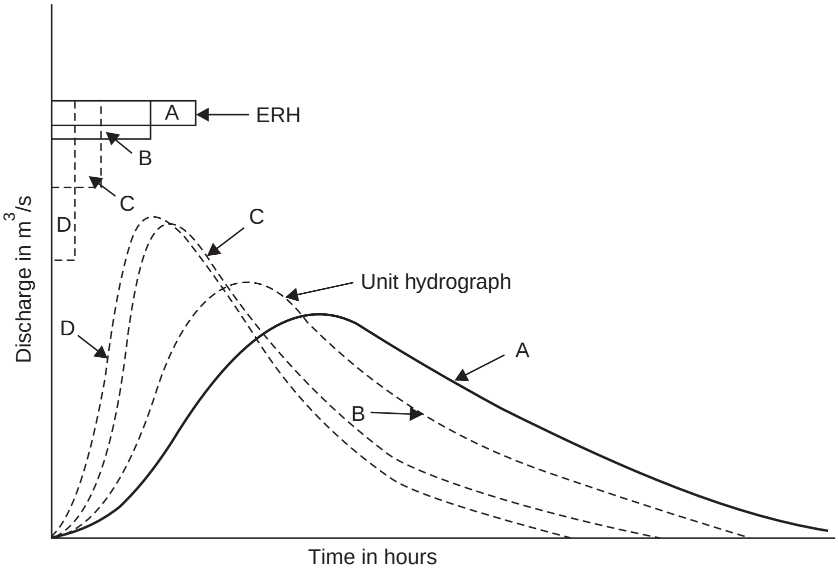

Fig. 2.26 Unit hydrographs of different durations

2.7.4.4. Synthetic Unit Hydrograph

Unit hydrographs can be derived only if suitable records of data for the catchment are available. Many of the catchments, especially the remote ones, remain ungauged. To derive unit hydrographs for such basins, one requires suitable empirical relations of regional validity which relate basin characteristics and the salient features of resulting hydrographs. Such relations usually pertain to time to peak, peak flow and time base of the unit hydrograph. Unit hydrographs derived in this manner are known as synthetic unit hydrograph. Dimensionless unit hydrographs (Q/Q_p versus t/t_{pk}) based on a study of large number of unit hydrographs are also used to derive synthetic unit hydrograph.

2.7.4.5. Use and Limitations of Unit Hydrograph

Unit hydrograph is useful for (*i*) the development of flood hydrographs for extreme rainfalls for use in the design of hydraulic structures, (*ii*) extension of flood-flow records based on rainfall data, and (*iii*) development of flood forecasting and warning system based on rainfall.

In very large catchment basins, storms may not meet the conditions of constant intensity within effective storm duration and uniform areal distribution. Therefore, each storm may give different direct runoff hydrograph under, otherwise, identical conditions. Therefore, unit hydrograph is considered applicable for catchments having area less than about 5000 km^2 (9). Very large catchments are usually divided into smaller sub-basins and the hydrographs of these sub-basins are processed to obtain composite hydrgraph at the basin outlet.

The application of unit hydrograph also requires that the catchment area should not be smaller than about 200 ha as for such small basins there are other factors which may affect the rainfall-runoff relation and the derived unit hydrograph may not be accurate enough.

Example 2.7 The following Table lists the ordinates of a runoff hydrograph in response to a rainfall of 21.90 mm during the first two hours, 43.90 mm in the next two hours, and 30.90 mm during the last two hours of the rainfall which lasted for six hours on July 19, 1995 in

Warasgaon catchment basin whose area is 133.1 km^2 (***Source*:** M.Tech. Dissertation on "Rainfall—runoff modelling of Mutha river system" by S.R. Vhatkar submitted at the Department of Hydrology, University of Rookee in 1996).

Time (hr)	0	2	4	6	8	10	12	14	16	18
Discharge (m^3/s)	0	171	393	522	297	133	51	10	10	10

Obtain the following :

(*a*) ϕ-index

(*b*) unit hydrograph and its duration (say, T hours)

(*c*) time of concentration, T_c

Thereafter, derive the following :

(*i*) Unit hydrographs for $2T$ and $3T$ hours

(*ii*) T-hr unit hydrograph from the derived $2T$-hr unit hydrograph

Solution: On plotting the runoff hydrograph, Fig. 2.27 (*a*), one notices that the stream flow stabilizes at 10 m^3/s. This must be on account of the base flow contribution.

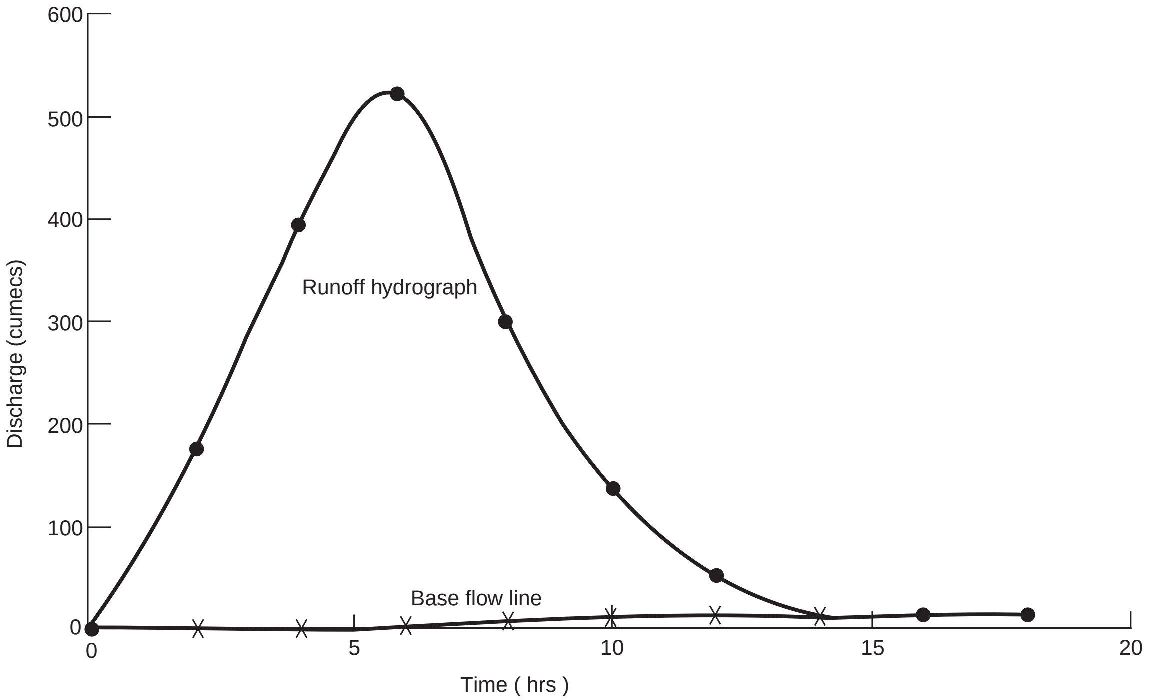

Fig. 2.27 (*a*) Runoff hydrograph and base flow (Example 2.7)

Therefore, treating the base flow as 10 m^3/s at time t = 14 hrs, the base flow line is obtained by assuming linear variation between t = 0 and t = 14 hrs. The values of the base flow (Col. 3 of Table *A* for this example) have been subtracted from the corresponding values of the runoff hydrograph (Col. 2 of Table *A*) to obtain the ordinates of the direct runoff hydrograph (Col. 4 of Table *A*).

(A) Computation of Unit Hydrograph ordinates (Example 2.7)

Time (hr)	*Runoff (cumecs)*	*Base flow (cumecs)*	*DRH (cumecs)*	*UH (cumecs)*	*Adjusted UH (cumecs)*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*
0	0	0	0	0.00	0
2	171	1.43	169.57	20.41	20
4	393	2.86	390.14	46.95	47
6	522	4.29	517.71	62.30	62
8	297	5.71	291.29	35.05	35
10	133	7.14	125.86	15.15	15
12	51	8.57	42.43	5.11	5
14	10	10	0	0.00	0
16	10	10			
18	10	10			
Total			1537	184.96	184

Measured runoff volume

$= (2 \times 3600)\ [0 + 169.57 + 390.14 + 517.71 + 291.29 + 125.86 + 42.43 + 0]$

$= 2 \times 3600 \times 1537$

$= 11066400\ \text{m}^3$

$\therefore$ Runoff depth $= \dfrac{\text{Runoff volume}}{\text{catchment area}} = \dfrac{11066400}{133.1 \times 10^6} = 0.0831$ m

$= 83.1$ mm $= 8.31$ cm.

Therefore, the ordinates of the unit hydrograph (for 1 cm rainfall) (Col. 5 of Table *A*) would be obtained by dividing the DRH values with 8.31. These values have been rounded off suitably (Col. 6 to Table *A*).

Runoff volume for *UH* $= (2 \times 3600)\ [0 + 20 + 47 + 62 + 35 + 15 + 5 + 0]$

$= (2 \times 3600)\ (184)$

$= 1324800\ \text{m}^3$

$\therefore$ Runoff depth from *UH* $= 1324800/(133.1 \times 10^6)$

$= 9.95 \times 10^{-3}$ m

$= 0.995$ cm.

This runoff depth of 0.995 cm for the unit hydrograph is close to 1.0 cm and is, therefore, accepted without any more adjustment. The *UH* so obtained is shown plotted in Fig. 2.27 (*b*). To determine the duration of the derived *UH*, one needs to derive effective rainfall intensity hyetograph for which knowledge of ϕ-index is required.

Total rainfall depth $= 21.90 + 43.10 + 30.90$

$= 95.90$ mm

$= 9.59$ cm

$\therefore$ Losses due to abstractions $= 95.90 - 83.10$

$= 12.80$ mm

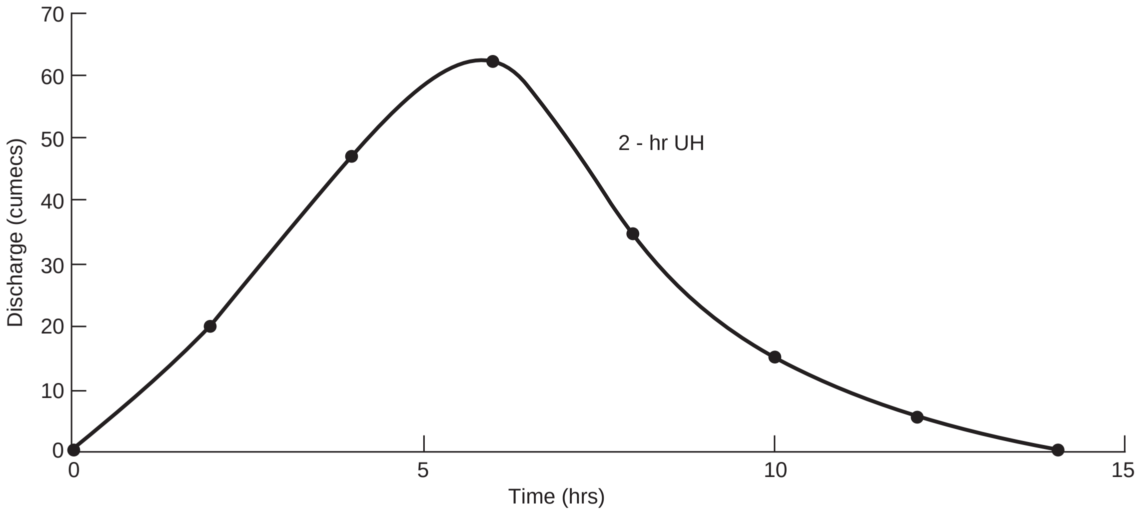

Fig. 2.27 (*b*) 2-hour Unit hydrograph (Example 2.7)

$$\therefore \qquad \phi\text{-index} = \frac{12.80}{6} = 2.13 \text{ mm/hr.}$$

Ordinates of the effective rainfall hyetograph (*i.e.,* plot of effective rainfall intensity versus time) have been worked out as follows :

Time (hr)	*Rainfall depth (mm)*	*Rainfall intensity (mm/hr)*	*ϕ-index (mm/hr)*	*Effective rainfall intensity (mm/hr)*
0-2	21.90	10.95	2.13	8.82
2-4	43.10	21.55	2.13	19.42
4-6	30.90	15.45	2.13	13.32

Effective rainfall hyetograph so obtained is shown in Fig. 2.27 (*c*).

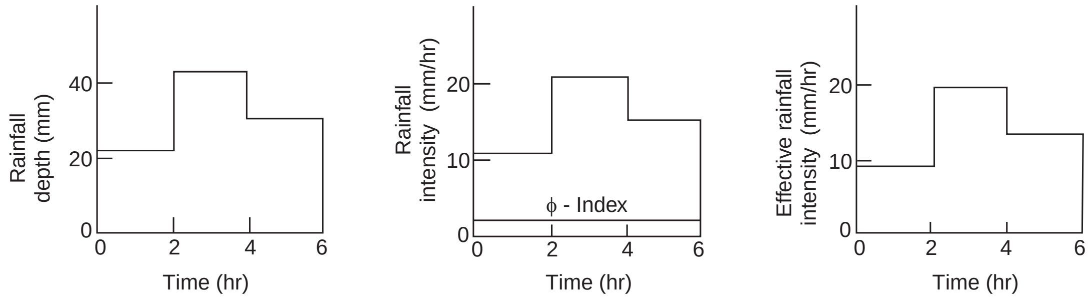

Fig. 2. 27 (*c*) Derivation of effective rainfall hyetograph (Example 2.7)

The duration of the unit hydrograph is expected to be between $t_{pk}/3$ and $t_{pk}/5$. Since t_{pk} (*i.e.,* time to peak) for the present problem is 6 hrs, the duration of the *UH* is expected to be tetween 1.2 to 2 hrs. To decide on the duration of the *UH*, one should draw *S*-hydrographs of different durations around the expected range of the duration. *S*-curve of duration shorter than that of the unit hydrograph would be very smooth while that of longer duration would be a wavy curve. This observation combined with the expected range between $t_{pk}/3$ and $t_{pk}/5$ enables one to choose suitable value for the duration of the unit hydrograph. For the present problem, three *S*-hydrographs for the durations of 1-hr, 2-hr and 3-hr are drawn. The ordinate values are listed in Cols. 3, 4 and 5 of Table *B* and the *S*-curves are as shown in Fig. 2.27 (*d*) . One may adopt 2 hour as the duration of the *UH*.

(B) Computation of S-hydrographs for deciding duration of UH (Example 2.7)

Time (hour)	*UH (cumecs)*	*S-Hydrographs (cumecs)*		
		1-hr	*2-hr*	*3-hr*
(1)	*(2)*	*(3)*	*(4)*	*(5)*
0	0	0	0	0
1	10	10	10	10
2	20	30	20	20
3	33	63	43	33
4	47	110	67	57
5	55	165	98	75
6	62	227	129	95
7	48	275	146	105
8	35	310	164	110
9	25	335	171	120
10	15	350	179	120
11	10	360	181	120
12	5	365	184	125
13	2	367	183	122
14	0	367	184	120
	367			

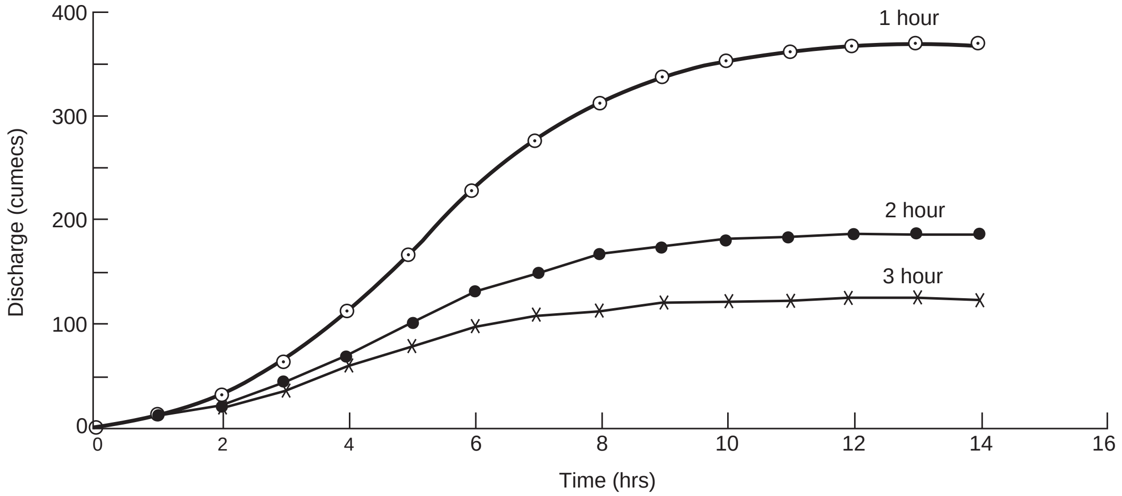

Fig. 2. 27 (*d*) *S*-hydrographs considering different durations of unit hydrograph (Example 2.7)

Time of concentration = Base of *DRH* – Duration of effective rainfall = 14 – 2 = 12 hrs.

For obtaining *UH* of 2*T i.e.,* 4-hr unit duration, one may use method of superposition. For 4-hr *UH*, one would require to superpose two 2-hr *UH*'s separated from each other by 2 hrs. The ordinates of the second *UH* (separated from the previous one by 2 hrs) are listed in Col. 3 of Table *C*. By adding the values of Cols. 2 and 3, one can obtain the ordinates of *DRH* of 2 cm rainfall in a duration of 4 hrs. (Col. 4 of Table *C*). Thus, the ordiantes of 4-hr unit hydrograph (Col. 5) would be obtained by dividing the values of Col. 4 of Table *C* by 2. The 4-hr unit hydrograph is shown in Fig. 2.27 (*e*).

(C) Computations for the derivation of 4-hr UH (Example 2.7)

Time (hours)	*2-hr UH*	*2-hr UH lagged by 2 hrs*	*DRH of 2cm in 4 hrs*	*4-hr UH*
(1)	*(2)*	*(3)*	*(4)*	*(5)*
0	0		0	0.0
2	20	0	20	10.0
4	47	20	67	33.5
6	62	47	109	54.5
8	35	62	97	48.5
10	15	35	50	25.0
12	5	15	20	10.0
14	0	5	5	2.5
16		0	0	0.0
				184.0

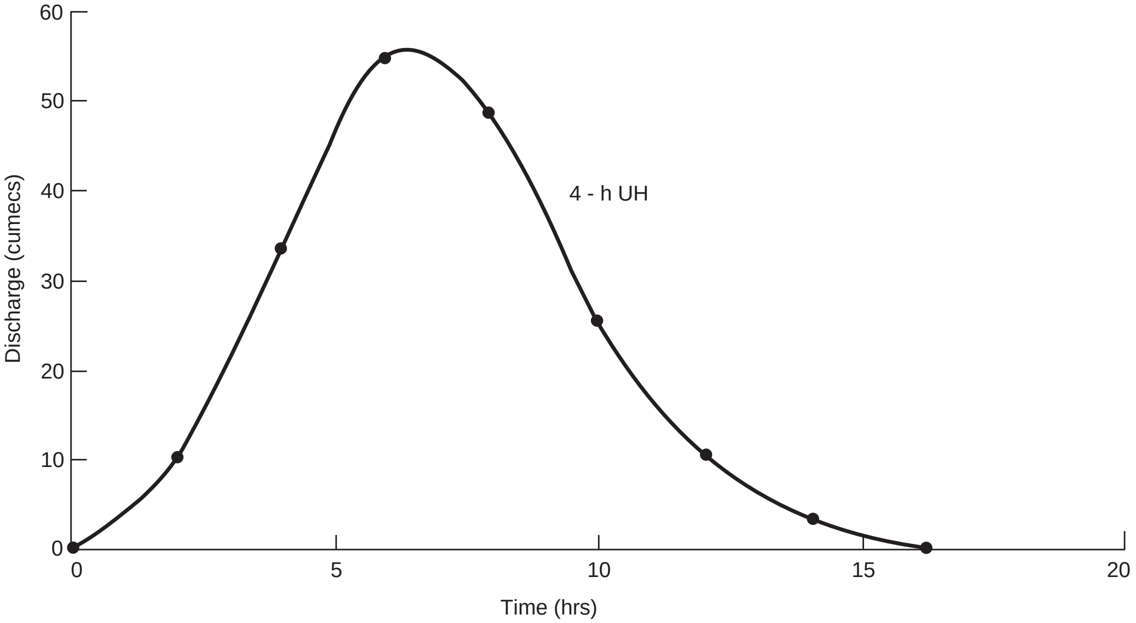

Fig. 2.27 (*e*) 4-hour Unit hydrograph (Example 2.7)

Similarly, for 3*T*-hr *UH i.e.,* 6-hr *UH*, one would require to superpose three 2-hr *UH*'s each separated from the previous one by 2 hrs. The ordinates of these three *UH*'s are in Cols. 2, 3 and 4 of Table *D* which, when added together, yield the ordinates of *DRH* (Col. 5 of Table *D*) of 3 cm rainfall in duration of 6 hrs. The values of Col. 5 of Table *D* are divided by 3 to obtain the ordinates of 6-hr *UH* (Col. 6 of Table *D*), Fig. 2.27 (*f*).

(D) Computation of 6-hr UH

Time (hours)	*2-h Unit Hydrograph*			*DRH of 3 cm in 6 hrs*	*6-hr UH*
	original	*lagged by 2 hr*	*lagged by 4 hr*		
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*
0	0			0	0.00
2	20	0		20	6.67
4	47	20	0	67	22.33
6	62	47	20	129	43.00
8	35	62	47	144	48.00
10	15	35	62	112	37.33
12	5	15	35	55	18.33
14	0	5	15	20	6.67
16		0	5	5	1.67
18			0	0	0.00
					184.00

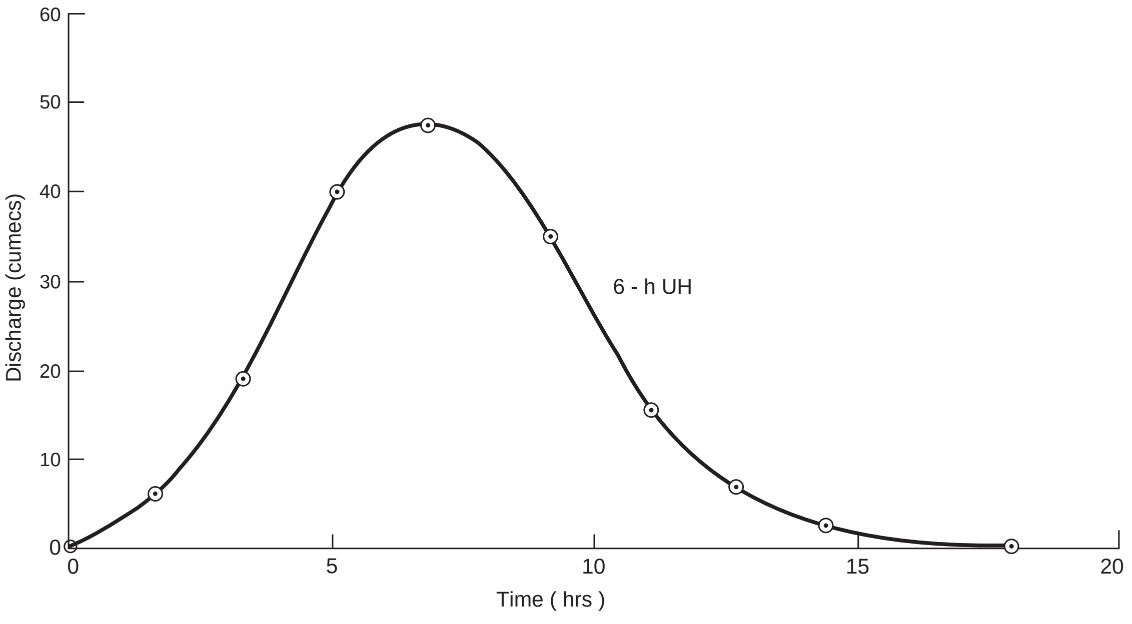

Fig. 2.27 (*f*) 6-hour Unit hydrograph (Example 2.7)

In order to obtain 2-h *UH* from the derived 4-h *UH*, one has to use the *S*-hydrograph method. After having obtained 4-hr *S*-curve (Col. 3 of Table *E*), ordinates of another 4-hr *S*-curve (lagging the previous 4-hr *S*-curve by 2-h), Col. 4 of Table *E*, are subtracted from the previous 4-hr *S*-curve. The difference gives the ordinates of a hydrograph resulting from a rainfall excess of (2/4) cm *i.e.,* 0.5 cm in a duration of 2 hours (Col. 5 of Table *E*). Therefore, the ordinates of the hydrograph (rainfall excess of 0.5 cm in a duration of 2 hours) are divided by 0.5 cm so as to have the hydrograph with rainfall excess of a 1 cm in a duration of 2 hours. (Col. 6 of Table *E*). The ordinates (Col. 6 of Table *E*), as expected, are the same as that of the ordinates of the original 2-hr, *UH* (Col. 6 of Table *A*). That is, this resulting hydrograph, Fig. 2.27 (*b*), is the 2-hr *UH* derived from the 4-h *UH*.

(E) Computation of 2-h UH from the derived 4-hr UH

Time (hours)	*4-hr UH*	*4-hour S-Hydrograph*		*DRH of 0.5 cm in 2 hours*	*2-h UH*
		Original	*lagged by 2 hours*		
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*
0	0.0	0		0.00	0
2	10.0	10	0	10.00	20
4	33.5	33.5	10	23.50	47
6	54.5	64.5	33.5	31.00	62
8	48.5	82	64.5	17.50	35
10	25.0	89.5	82	7.50	15
12	10.0	92	89.5	2.50	5
14	2.5	92	92	0.00	0
16	0.0	92	92	0.00	

Sum of all the ordinates of any *UH* of this example is 184 m³/s which results in direct runoff depth of

$$\frac{184 \times 2 \times 3600}{133.1 \times 10^6} \times 100 \quad i.e., \quad 0.995 \text{ cm} \approx 1.0 \text{ cm}$$

Therefore, the computations are in order.

Also, the correctness of the computations of each *S*-curve can be examined by comparing the computed equilibrium discharge with the value obtained from Eq. (2.23). For example, for 3-hr *S*-curve,

$$Q_e = (2.78)(133.1/3) = 123.34 \text{ m}^3/\text{s}$$

which value compares well with the values of discharge around equilibrium conditions.

2.8. FLOODS

A flood represents an unusual high stage in a river such that the river overflows its banks and, thus, inundates the adjoining land. Floods cause huge loss of life and property besides disrupting all human activities resulting into large economic loss. India suffers greatly on account of floods or hydrologic droughts occurring recurrently in one or other part of the country. Hydrologic drought is a condition (or period) during which stream flows are inadequate to supply for the established uses (domestic, irrigation, hydropower *etc.*) of water under a given water-management system. Complete control over floods and droughts is impossible to achieve.

For the purpose of designing any hydraulic structure, one needs to know the magnitude of the peak flood (or flow) that can be expected with an assigned frequency during the life of the structure.

2.8.1. Estimation of Peak Flood

Floods or flows in a stream depend upon several factors the influence of which on floods cannot be ascertained accurately. Therefore, the floods, like other hydrologic processes, cannot be modelled analytically.

The following alternative methods are used for estimation of the peak flood:

(*i*) Rational method

(*ii*) Empirical method

(*iii*) Unit hydrograph method

(*iv*) Flood—frequency method.

The choice of a method for estimation of the peak flood primarily depends upon the importance of the work and available data.

2.8.1.1. Rational Method

The runoff rate during and after a precipitation of uniform intensity and long duration typically varies as shown in Fig. 2.28. The runoff increases from zero to a constant peak value when the water from the remotest area of the catchment basin reaches the basin outlet. If t_c is the time taken for water from the remotest part of the catchment to reach the outlet and the rainfall continues beyond t_c, the runoff will have attained constant peak value. When the rain stops the runoff starts decreasing. The peak value of runoff, Q_p (m^3/s) is given as

$$Q_p = \frac{1}{3.6} C\,i\,A \tag{2.26}$$

where, C = coefficient of runoff (Table 2.3) depending upon the nature of the catchment surface and the rainfall intensity, i.

i = the mean intensity of rainfall (mm/hr) for a duration equal to or exceeding t_c and an exceedence probability P, and

A = catchment area in km^2.

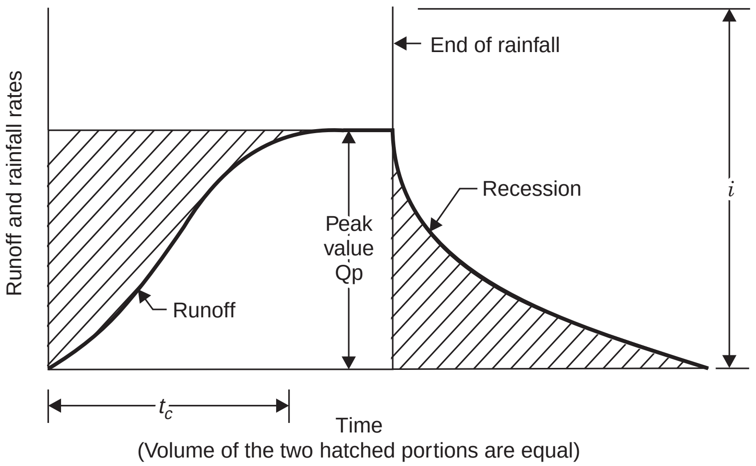

Fig. 2.28 Runoff hydrograph due to uniform rainfall

Table 2.3 : Coefficient of runoff C for different surfaces

Type of surface	*Value of C*
Wooded areas	0.01 – 0.20
Parks, open spaces lawns, meadows	0.05 – 0.30
Unpaved streets, Vacant lands	0.10 – 0.30
Gravel roads and walks	0.15 – 0.30
Macadamized roads	0.25 – 0.60
Inferior block pavements with open joints	0.40 – 0.50
Stone, brick and wood-block pavements with open or uncemented joints	0.40 – 0.70
Stone, brick and wood-block pavements with tightly cemented joints	0.75 – 0.85
Asphalt pavements in good order	0.85 – 0.90
Watertight roof surfaces	0.70 – 0.95

The time t_c (known as time of concentration) can be obtained from Kirpich equation

$$t_c = 0.01947\, L^{0.77}\, S^{-0.386} \tag{2.27}$$

where, t_c is in minutes, L is the maximum length (in metres) of travel of water from the upstream end of the catchment basin to the basin outlet, and S is the slope of the catchment which is equal to $\Delta H/L$ in which ΔH is the difference of elevations of the upstream end of the catchment and the outlet.

The method is suitable for catchments of small size less than about 50 km^2 *i.e.,* 5000 hectares and is often used for peak flow estimation required for deisgn of storm drains, culverts, highway drains etc.

2.8.1.2. Empirical Methods

The empirical relations are based on statistical correlation between the observed peak flow, Q_p (m^3/s) and the area of catchment, A (km^2) in a given region and are, therefore, region-specific. The following empirical relations are often used in India:

(*a*) Dicken's formula, used in the central and northern parts of India, is as follows:

$$Q_p = C_D\, A^{3/4} \tag{2.28}$$

where, C_D (known as Dicken's constant) is selected from the following Table (8):

Region	*Value of C_D*
North Indian plains	6
North Indian hilly regions	11 – 14
Central India	14 – 28
Coastal Andhra and Orissa	22 – 28

(*b*) Ryves formula, used in Tamil Nadu and parts of Karnataka and Andhra Pradesh, is as follows :

$$Q_p = C_R\, A^{2/3} \tag{2.29}$$

The recommended values of C_R are as follows:

C_R = 6.8 for areas within 80 km from the east cost
= 8.5 for areas which are 80 – 160 km from the east coast
= 10.2 for limited areas near hills.

(c) Inglis formula, used in regions of Western Ghats in Maharashtra, is as follows:

$$Q_p = \frac{124\, A}{\sqrt{A + 10.4}} \tag{2.30}$$

(d) Envelope curve technique is used to develop peak flow-area relationship for areas having meager peak flood data. The data of peak flow from large number of catchments (having meteorological characteristics similar to the region for which peak flow-area relationship is sought to be prepared) are plotted against catchment area on a log-log graph paper. The enveloping curve, encompassing all the data points, gives peak flow–area relation for any catchment that has meteorological characteristics similar to the ones of catchments whose data were used to obtain the envolope curve. Equation of the envelope curve would yield empirical formula of the type $Q_p = f(A)$. Two such curves, based on data from large catchments of areas in the range of 10^3 to 10^6 km^2, are shown in Fig. 2.29 (8).

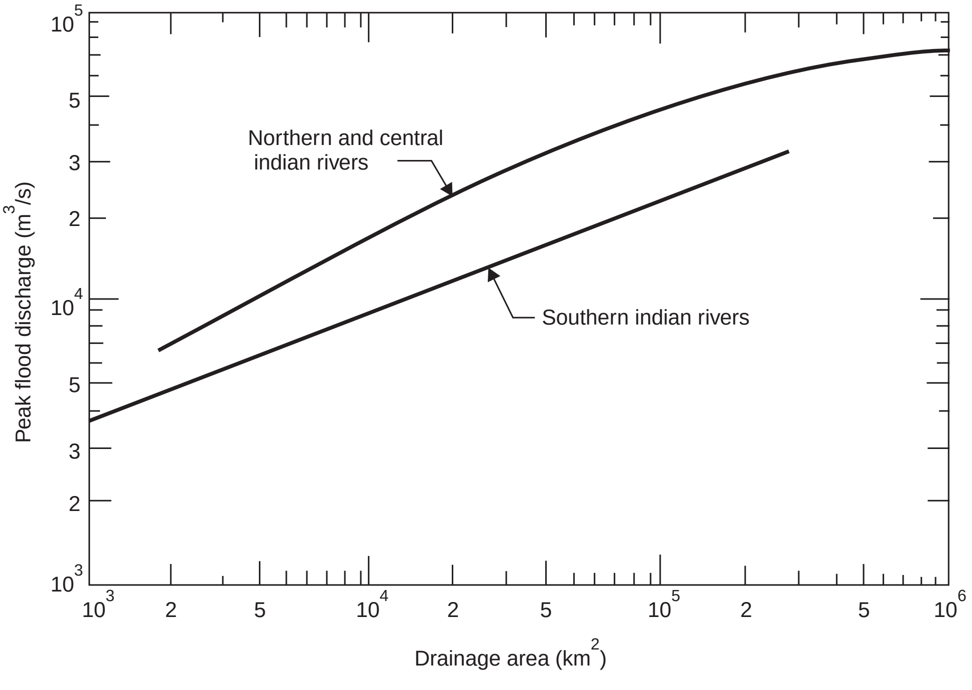

Fig. 2.29 Envelope curves for Indian rivers (8)

Based on data of peak flood in various catchments throughout the world, Baird and McILL wraith have given the following formula for Q_p (8) :

$$Q_p = \frac{3025\, A}{(278 + A)^{0.78}} \tag{2.31}$$

Similarly, CWC (11, 12, and 13) recommended the following relation for the estimation of peak flood flow for small to medium catchments (A < 250 sq. km) :

$$Q_p = A\ [a(t_p)^b] \qquad ...(2.32)$$

in which, t_p is the time (in hours) to peak and is expressed as

$$t_p = c\ [L_c/\sqrt{S}\,]^d \qquad ...(2.33)$$

in which, L_c is the length of the longest stream from the point, opposite to the centroid of the catchment, up to the gauging site in km, S is the slope of the catchment in m/km, and the values of the constants a, b, c, and d in Eqs. (2.32) and (2.33) are region-dependent (Table 2.4).

Table 2.4. Values of the constants *a*, *b*, *c* and *d* for Eqs. (2.32) and (2.33)

Region	a	b	c	d
Mahi and Sabarmati	1.161	– 0.635	0.433	0.704
Lower Narmada and Tapi	1.920	– 0.780	0.523	0.323
Mahanandi	1.121	– 0.660	1.970	0.240
Godavari	1.968	– 0.842	0.253	0.450

2.8.1.3. Unit Hydrograph Method

One can use already known or derived unit hydrograph for a catchment basin to predict the peak flood hydrograph in response to an extreme rainfall *i.e.,* the design storm in the catchment as discussed in Art. 2.7.

2.8.1.4. Flood Frequency Method

The data of annual maximum flood in a given catchment area for a large number of successive years (*i.e.*, data series of the largest flood in each successive year) are arranged in decreasing order of magnitude. The probability P of each flood being equalled or exceeded, also known as the plotting position, is given as

$$P = \frac{m}{N+1} \tag{2.34}$$

where, m is the order number of the relevant flood in the table of annual floods arranged in decreasing order and N is the total number of annual floods in the data. Return period (or recurrence interval) T_r is the reciprocal of the probability, P. Thus,

$$T_r = \frac{1}{P} \tag{2.35}$$

Frequency of flood (or any other hydrologic event) of a given magnitude is the average number of times a flood of given (or higher) magnitude is likely to occur.Thus, the 100-year flood is a flood which has a probability of being equalled or exceeded once in every 100 years. That is, $P = 1/100$ and $T_r = 100$ years.

One can draw a graph between the flood magnitude and its return period (or plotting position) on the basis of the data series of annual floods and fit a curve to obtain probability (or empirical) distribution. This graph may be extrapolated to get the design flood for any other return period. Such plots are used to estimate the floods with shorter return periods. For longer return periods, however, one should fit a theoretical distribution to the flood data. Some of the commonly used theoretical frequency distribution functions for estimating the extreme flood magnitudes are as follows:

(*a*) Gumbel's extreme value distribution

(*b*) The log-Pearson Type *III* distribution, and

(*c*) The log normal distribution.

Flood frequency analysis is a viable method of flood-flow estimation in most situations. But, it requires data for a minimum of 30 years for meaningful predictions. Such analysis gives reliable predictions in regions of relatively uniform climatic conditions from year to year.

2.8.2. Design Flood

Design flood is the flood adopted for the design of a hydraulic structure. It would be, obviously, very costly affair to design any hydraulic structure so as to make it safe against the maximum flood possible in the catchment. Smaller structures such as culverts, storm drainage systems can be designed for relatively small floods (*i.e.,* more frequent floods) as the consequences of a higher than design flood may only cause temporary inconvenience and some repair works without any loss of life and property. However, failure of structures such as spillway would cause huge loss of life and property and, therefore, such structures should be designed for relatively more severe floods having relatively larger return period. Table 2.5 provides guidelines for selecting design floods (14). The terms *PMF* and *SPF* in the Table 2.5 have the following meaning :

PMF, *i.e.,* probable maximum flood is the extreme large flood that is physically possible in a region as a result of severemost (including rare ones) combination of meteorological and hydrological factors. *SPF* is the standard project flood that would result from a severe combination of meteorological and hydrological factors. Usually, SPF is about 40 to 60% of *PMF*.

Table 2.5 Guidelines for selecting design floods (14)

S. No.	*Structure*	*Recommended design flood*
1.	Spillways for major and medium projects with storage more than 60 Mm^3	(*a*) *PMF* determined by unit hydrograph and probable maximum precipitation (*PMP*) (*b*) If (*a*) is not applicable or possible, flood-frequency method with T = 1000 years to 5000 years
2.	Permanent barrage and minor dams with capacity less than 60 Mm^3	(*a*) SPF determined by unit hydrograph and standard project storm (*SPS*) which is usually the largest recorded storm in the region (*b*) Flood with a return period of 100 years. (*a*) or (*b*) whichever gives higher value.
3.	Pickup weirs	Flood with a return period of 100 or 50 years depending on the importance of the project.
4.	Aqueducts (*a*) Waterway (*b*) Foundations and free board	 Flood with T = 50 years Flood with T = 100 years
5.	Project with very scanty or inadequate data	Empirical formulae

The PMF is used for structures such as dams, spillways etc. whose failure would result in huge loss of life and property. Table 2.6 provides guidelines for this purpose (15).

Table 2.6 Design flood for dams

Size/class of dam	*Gross storage (Mm^3)*	*Hydraulic Head (m)*	*Design Flood*
Small	0.5 to 10.00	7.5 to 12.0	100 – year flood
Medium	10.0 to 60.00	12.0 to 30.0	SPF
Large	more than 60.0	more than 30.0	PMF

EXERCISES

2.1 There are five rain gauges uniformly spread in a small watershed. The depths of rainfall observed at these rain gauges and the area of Theissen polygons for the corresponding rain gauges are as follows :

Rain gauge no.	1	2	3	4	5
Rainfall depth (cm)	47.3	46.4	43.8	52.3	48.5
Area of Theissen polygon (1000 m^2)	95	102	98.67	80.52	85.38

Determine the average depth of precipitation.

2.2 The amounts of rainfall for 6 successive days on a catchment were 2, 5, 7, 8, 5, and 1 cm. If the ϕ-index for the storm is 4 cm/day, find the total surface runoff.

2.3 The normal annual precipitation at four rain gauge stations is 120, 100, 84, and 120 cm, respectively. During a particular storm the rain gauge at station A became non-functional. The rainfall depths recorded at B, C, and D are respectively, 10, 7 and 11 cm. Estimate the rainfall depth at station A during the storm.

2.4 In a drainage basin, there exist 6 rain gauges, and average rainfall at these stations is 106, 91, 65, 55, 62, and 48 cm. Determine if these rain gauges are adequate to give reliable measurements of rainfall with an error of less than 10% ? If not, how many additional rain gauges are needed ?

2.5 The ordinates of three unit hydrographs derived from separate storms of 4-hour rains each are as follows :

Hours	*Storm 1*	*Storm 2*	*Storm 3*	*Hours*	*Storm 1*	*Storm 2*	*Storm 3*
0	0	0	0	8	135	165	214
1	115	20	16	9	100	125	164
2	370	120	57	10	70	85	121
3	505	353	173	11	45	55	90
4	395	460	335	12	27	30	60
5	315	400	442	13	15	15	35
6	240	300	400	14	10	3	16
7	175	215	283	15	0	0	0

Find the average unit hydrograph.

2.6 The observed stream discharges resulting from a storm of 3-hr duration are as follows :

Hour	*Day 1*	*Day 2*	*Day 3*	*Hour*	*Day 1*	*Day 2*	*Day 3*
3 a.m	550	4500	1800	3 p.m.	8000	2600	1000
6 a.m	500	3900	1600	6 p.m.	7000	2300	900
9 a.m	6000	3400	1400	9 p.m.	6000	2000	800
Noon	9500	3000	1200	Mid-night	5200	1800	700

The drainage area of the stream is 2000 km^2. Derive the unit hydrograph assuming constant base flow of 500 m^3/s.

2.7 For storm 1 of Exercise 2.5, obtain the S-hydrograph and derive the 2-hr and 6-hr unit hydrographs.

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3

SOIL–WATER RELATIONS AND IRRIGATION METHODS

3.1. SOILS

Soil mainly consists of finely divided organic matter and minerals (formed due to disintegration of rocks). It holds the plants upright, stores water for plant use, supplies nutrients to the plants and helps in aeration. Soils can be classified in many ways, such as on the basis of size (gravel, sand, silt, clay, *etc.*), geological process of formation, and so on. Based on their process of formation (or origin), they can be classified into the following categories:

(*i*) *Residual soils:* Disintegration of natural rocks due to the action of air, moisture, frost, and vegetation results in residual soils.

(*ii*) *Alluvial soils:* Sediment material deposited in bodies of water, deltas, and along the banks of the overflowing streams forms alluvial soils.

(*iii*) *Aeolian soils:* These soils are deposited by wind action.

(*iv*) *Glacial soils:* These soils are the products of glacial erosion.

(*v*) *Colluvial soils:* These are formed by deposition at foothills due to rain wash.

(*vi*) *Volcanic soil:* These are formed due to volcanic eruptions and are commonly called as volcanic wash.

The soils commonly found in India can be classified as follows:

(i) Alluvial Soils: Alluvial soils include the deltaic alluvium, calcareous alluvial soils, coastal alluvium, and coastal sands. This is the largest and most important soil group of India.

The main features of the alluvial soils of India are derived from the deposition caused by rivers of the Indus, the Ganges, and the Brahmaputra systems. These rivers bring with them the products of weathering of rocks constituting the mountains in various degrees of fineness and deposit them as they traverse the plains. These soils vary from drift sand to loams and from fine silts to stiff clays. Such soils are very fertile and, hence, large irrigation schemes in areas of such soils are feasible. However, the irrigation structures themselves would require strong foundation.

(ii) Black Soils: The black soils vary in depth from a thin layer to a thick stratum. The typical soil derived from the Deccan trap is black cotton soil. It is common in Maharashtra, western parts of Madhya Pradesh, parts of Andhra Pradesh, parts of Gujarat, and some parts of Tamil Nadu. These soils may vary from clay to loam and are also called heavy soils. Many black soil areas have a high degree of fertility but some, especially in the uplands, are rather poor. These are suitable for the cultivation of rice and sugarcane. Drainage is poor in such soils.

(iii) Red Soils: These are crystalline soils formed due to meteoric weathering of the ancient crystalline rocks. Such soils are found in Tamil Nadu, Karnataka, Goa, south-eastern Maharashtra, eastern Andhra Pradesh, Madhya Pradesh, Orissa, Bihar, and some districts of West Bengal and Uttar Pradesh. Many of the so-called red soils of south India are not red. Red soils have also been found under forest vegetation.

(iv) Lateritic Soils: Laterite is a formation peculiar to India and some other tropical countries. Laterite rock is composed of a mixture of the hydrated oxides of aluminium and iron with small amounts of manganese oxides. Under the monsoon conditions, the siliceous matter of the rocks is leached away almost completely during weathering. Laterites are found on the hills of Karnataka, Kerala, Madhya Pradesh, the estern Ghats of Orissa. Maharashtra, West Bengal, Tamil Nadu, and Assam.

(v) Desert Soils: A large part of the arid region belonging to western Rajasthan, Haryana, and Punjab lying between the Indus river and the Aravalli range is affected by desert and conditions of geologically recent origin. This part is covered with a mantle of the blown sand which, combined with the arid climate, results in poor soil development. The Rajasthan desert is a vast sandy plain including isolated hills or rock outcrops at places. The soil in Rajasthan improves in fertility from west and north-west to east and north-east.

(vi) Forest Soils: These soils contain high percentage of organic and vegetable matter and are also called humus. These are found in forests and foothills.

Soils suitable for agriculture are called arable soils and other soils are non-arable. Depending upon their degree of arability, these soils are further subdivided as follows:

(*i*) *Class I:* The soils in class I have only a few limitations which restrict their use for cultivation. These soils are nearly level, deep, well-drained, and possess good water-holding capacity. They are fertile and suitable for intensive cropping.

(*ii*) *Class II:* These soils have some limitations which reduce the choice of crops and require moderate soil conservation practices to prevent deterioration, when cultivated.

(*iii*) *Class III:* These soils have severe limitations which reduce the choice of crops and require special soil conservation measures, when cultivated.

(*iv*) *Class IV:* These soils have very severe limitations which restrict the choice of crops to only a few and require very careful management. The cultivation may be restricted to once in three or four years.

Soils of type class *I* to class *IV* are called arable soils. Soils inferior to class *IV* are grouped as non-arable soils. Irrigation practices are greatly influenced by the soil characteristics. From agricultural considerations, the following soil characteristics are of particular significance.

(*i*) Physical properties of soil,

(*ii*) Chemical properties of soil, and

(*iii*) Soil-water relationships.

3.2. PHYSICAL PROPERTIES OF SOIL

The permeability of soils with respect to air, water, and roots are as important to the growth of crop as an adequate supply of nutrients and water. The permeability of a soil depends on the porosity and the distribution of pore spaces which, in turn, are decided by the texture and structure of the soil.

3.2.1. Soil Texture

Soil texture is determined by the size of soil particles. Most soils contain a mixture of sand (particle size ranging from 0.05 to 1.00 mm in diameter), silt (0.002 to 0.05 mm) and clay (smaller than 0.002 mm). If the sand particles dominate in a soil, it is called sand and is a coarse-textured soil. When clay particles dominate, the soil is called clay and is a fine-textured soil. Loam soils (or simply loams) contain about equal amount of sand, silt, and clay and are medium-textured soils.

The texture of a soil affects the flow of water, aeration of soil, and the rate of chemical transformation all of which are important for plant life. The texture also determines the water holding capacity of the soil.

3.2.2. Soil Structure

Volume of space (*i.e.,* the pores space) between the soil particles depends on the shape and size distribution of the particles. The pore space in irrigated soils may vary from 35 to 55 per cent. The term porosity is used to measure the pore space and is defined as the ratio of the volume of voids (*i.e.,* air and water-filled space) to the total volume of soil (including water and air). The pore space directly affects the soil fertility (*i.e.,* the productive value of soil) due to its influence upon the water-holding capacity and also on the movement of air, water, and roots through the soil.

Soils of uniform particle size have large spaces between the particles, whereas soils of varying particle sizes are closely packed and the space between the particles is less. The particles of a coarse-grained soil function separately but those of fine-grained soils function as granules. Each granule consists of many soil particles. Fine-textured soils offer a favourable soil structure permitting retention of water, proper movement of air and penetration of roots which is essential for the growth of a crop.

The granules are broken due to excessive irrigation, ploughing or working under too wet (puddling) or too dry conditions. Such working affects the soil structure adversely. The structure of the irrigated soil can be maintained and improved by proper irrigation practices some of which are as follows (1):

(*i*) Ploughing up to below the compacted layers,

(*ii*) After ploughing, allowing sufficient time for soil and air to interact before preparing the seed bed or giving pre-planting irrigation,

(*iii*) The organic matter spent by the soil for previous crops should be returned in the form of fertilisers, manures, *etc*.,

(*iv*) Keeping cultivation and tillage operations to a minimum, and

(*v*) Adopting a good crop rotation.

Green manures keep the soil fertility high. Crops like hamp, *gwar, moong etc*. are grown on the fields. When these plants start flowering, ploughing is carried out on the fields so that these plants are buried below the ground surface. Their decomposition makes up for the soil deficiencies.

The tendency of cultivators to grow only one type of crop (due to better returns) should be stopped as this cultivation practice leads to the deficiency in the soil of those nutrients which are needed by the crop. If the land is not used for cultivation for some season, the soil recoups its fertility. Alternatively, green manures can be used. Rotation of crops (which means growing different crops on a field by rotation) is also useful in maintaining soil fertility at a satisfactory level.

3.2.3. Depth of Soil

The importance of having an adequate depth of soil for storing sufficient amount of irrigation water and providing space for root penetration cannot be overemphasised. Shallow soils require more frequent irrigations and cause excessive deep percolation losses when shallow soils overlie coarse-textured and highly permeable sands and gravels. On the other hand, deep soils would generally require less frequent irrigations, permit the plant roots to penetrate deeper, and provide for large storage of irrigation water. As a result, actual water requirement for a given crop (or plant) is more in case of shallow soils than in deep soils even though the amount of water actually absorbed by the crop (or plant) may be the same in both types of soils. This is due to the unavoidable water losses at each irrigation.

3.3. CHEMICAL PROPERTIES OF SOIL

For satisfactory crop yield, soils must have sufficient plant nutrients, such as nitrogen, carbon, hydrogen, iron, oxygen, potassium, phosphorus, sulphur, magnesium, and so on. Nitrogen is the most important of all the nutrients. Nitrogenous matter is supplied to the soil from barnyard manure or from the growing of legume crops as green manures, or from commercial fertilisers. Plants absorb nitrogen in the form of soluble nitrates.

Soils having excess (greater than 0.15 to 0.20 per cent) soluble salts are called saline soils and those having excess of exchangeable sodium (more than 15 per cent or pH greater than 8.5) are called alkaline (or sodic) soils. Excessive amounts of useful plant nutrients such as sodium nitrate and potassium nitrate may become toxic to plants. Saline soils delay or prevent crop germination and also reduce the amount and rate of plant growth because of the high osmotic pressures which develop between the soil-water solution and the plants. These pressures adversely affect the ability of the plant to absorb water.

Alkaline (or sodic) soils tend to have inferior soil structure due to swelling of the soil particles. This changes the permeability of the soil. Bacterial environment is also an important feature of the soil-water-plant relationship. The formation of nitrates from nitrogenous compounds is accelerated due to favourable bacterial activity. Bacterial action also converts organic matter and other chemical compounds into forms usable by the plants. Bacterial activity is directly affected by the soil moisture, soil structure, and soil aeration. Compared to humid climate soils, arid soils provide better bacterial environment up to much greater depths because of their open structure. Besides, due to low rainfall in arid regions, leaching (*i.e.,* draining away of useful salts) is relatively less and the arid soils are rich in mineral plant food nutrients, such as calcium and potassium.

Soils become saline or alkaline largely on account of the chemical composition of rocks weathering of which resulted in the formation of soils. Sufficient application of water to the soil surface through rains or irrigation helps in carrying away the salts from the root-zone region of the soil to the rivers and oceans. When proper drainage is not provided, the irrigation water containing excessive quantities of salt may, however, render the soil unsuitable for cultivation. Saline and alkaline soils can be reclaimed by: (*i*) adequate lowering of water table, (*ii*) leaching out excess salts, and (*iii*) proper management of soil so that the amount of salt carried away by the irrigation water is more than the amount brought in by irrigation water.

3.4. SOIL–WATER RELATIONSHIPS

Any given volume V of soil (Fig. 3.1) consists of : (*i*) volume of solids V_s , (*ii*) volume of liquids (water) V_w, and (*iii*) volume of gas (air) V_a. Obviously, the volume of voids (or pore spaces) $V_v = V_w + V_a$. For a fully saturated soil sample, $V_a = 0$ and $V_v = V_w$. Likewise, for a completely dry specimen, $V_w = 0$ and $V_v = V_a$. The weight of air is considered zero compared to the weights of water and soil grains. The void ratio e, the porosity n, the volumetric moisture content w, and the saturation S are defined as

$$e = \frac{V_v}{V_s},\ n = \frac{V_v}{V},\ w = \frac{V_w}{V},\ S = \frac{V_w}{V_v}$$

Therefore, $$w = Sn \qquad \text{...(3.1)}$$

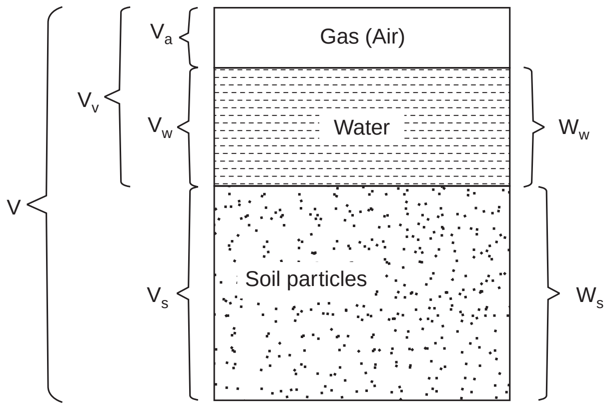


Fig. 3.1 Occupation of space in a soil sample

It should be noted that the value of porosity n is always less than 1.0. But, the value of void ratio e may be less, equal to, or greater than 1.0.

Further, if the weight of water in a wet soil sample is W_w and the dry weight of the sample is W_s , then the dry weight moisture fraction, W is expressed as (2)

$$W = \frac{W_w}{W_s} \qquad (3.2)$$

The bulk density (or the bulk specific weight or the bulk unit weight) γ_b of a soil mass is the total weight of the soil (including water) per unit bulk volume, *i.e.*,

$$\gamma_b = \frac{W_T}{V}$$

in which, $$W_T = W_s + W_w$$

The specific weight (or the unit weight) of the solid particles is the ratio of dry weight of the soil particles W_s to the volume of the soil particles V_s, *i.e.*, W_s/V_s. Thus,

$$G_b\, \gamma_w = \frac{W_s}{V} \qquad i.e., V = \frac{W_s}{G_b \gamma_w}$$

and $$G_s\,\gamma_w = \frac{W_s}{V_s} \quad i.e., \quad V_s = \frac{W_s}{G_s\gamma_w}$$

$$\therefore \quad \frac{V_s}{V} = \frac{G_b}{G_s} \tag{3.3}$$

Here, γ_w is the unit weight of water and G_b and G_s are, respectively, the bulk specific gravity of soil and the relative density of soil grains. Further,

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

$$\therefore \quad G_b = G_s(1 - n) \tag{3.4}$$

Also, $$w = \frac{V_w}{V} = \frac{W_w/\gamma_w}{W_s/(G_b\gamma_w)} = G_b\frac{W_w}{W_s}$$

$$\therefore \quad w = G_b W \tag{3.5}$$

and $$w = G_s(1 - n)W$$

Considering a soil of root-zone depth d and surface area A (*i.e.,* bulk volume = Ad),

$$W_s = V_s G_s \gamma_w = Ad\,(1 - n)\,G_s\gamma_w$$

Therefore, the dry weight moisture fraction, $W = \dfrac{W_w}{W_s}$

$$= \frac{V_w\gamma_w}{Ad\,(1-n)\,G_s\gamma_w}$$

Therefore, the volume of water in the root-zone soil,

$$V_w = W\,Ad\,(1 - n)\,G_s \tag{3.6}$$

This volume of water can also be expressed in terms of depth of water which would be obtained when this volume of water is spread over the soil surface area A.

$\therefore$ Depth of water, $$d_w = \frac{V_w}{A}$$

$$d_w = G_s\,(1 - n)\,Wd \tag{3.7}$$

or $$d_w = w\,d \tag{3.8}$$

Example 3.1 If the water content of a certain saturated soil sample is 22 per cent and the specific gravity is 2.65, determine the saturated unit weight γ_{sat}, dry unit weight γ_d, porosity n and void ratio e.

Solution:

$$W = \frac{W_w}{W_s} = 0.22$$

and $$G_s = \frac{W_s}{V_s\gamma_w} = 2.65$$

$$W_s = 2.65\,\gamma_w\,V_s$$

and $$WW_s = W_w$$

$$= 0.22 \times 2.65\,\gamma_w V_s$$

$$V_w = \frac{W_w}{\gamma_w} = 0.22 \times 2.65 \times V_s = 0.583\ V_s$$

Total volume $V = V_s + V_w$ (as $V_a = 0$ since the sample is saturated)

$$= V_s\,(1 + 0.583)$$

$$= 1.583\ V_s$$

$$n = \frac{V_v}{V} = \frac{0.583\ V_s}{1.583\ V_s} = 36.8\%$$

(since $V_v = V_w$ as the soil sample is saturated)

and $$e = \frac{V_v}{V_s} = 0.583 = 58.3\%$$

and total weight $$W = W_w + W_s$$

$$= 0.22 \times 2.65 \times \gamma_w V_s + 2.65\ \gamma_w V_s$$

$$= 3.233\ \gamma_w V_s$$

$$\gamma_{sat} = \frac{W}{V} = \frac{3.233\ \gamma_w V_s}{1.583\ V_s}$$

$$= 20.032 \text{ kN/m}^3 \quad \text{(since } \gamma_w = 9810 \text{ N/m}^3\text{)}$$

$$\gamma_d = \frac{W_s}{V} = \frac{2.65\ \gamma_w V_s}{1.583\ V_s}$$

$$= 16.422 \text{ kN/m}^3.$$

Example 3.2 A moist clay sample weighs 0.55 N. Its volume is 35 cm^3. After drying in an oven for 24 hours, it weights 0.50 N. Assuming specific gravity of clay as 2.65, compute the porosity n, degree of saturation S, original moist unit weight, and dry unit weight.

Solution:

$$W_T = 0.55 \text{ N}$$

$$W_s = 0.50 \text{ N}$$

$$W_w = 0.05 \text{ N}$$

$$V_s = \frac{W_s}{\gamma_s} = \frac{0.5}{2.65 \times 9810}$$

$$= 1.923 \times 10^{-5} \text{ m}^3 = 19.23 \text{ cm}^3$$

$$V_w = \frac{W_w}{\gamma_w} = \frac{0.05}{9810}$$

$$= 5.1 \times 10^{-6} \text{ m}^3 = 5.10 \text{ cm}^3$$

$$V_v = V - V_s = 35 - 19.23$$

$$= 15.77 \text{ cm}^3$$

Porosity, $$n = \frac{V_v}{V} = \frac{15.77}{35} = 45.06\%$$

Degree of saturation, $$S = \frac{V_w}{V_v} = \frac{5.10}{15.77} = 32.34\%$$

Moist unit weight, $\gamma = \dfrac{0.55}{35} = 0.016 \text{ N/m}^3$

Dry unit weight, $\gamma_d = \dfrac{0.50}{35} = 0.014 \text{ N/m}^3$.

Example 3.3 A moist soil sample has a volume of 484 cm^3 in the natural state and a weight of 7.94N. The dry weight of the soil is 7.36 N and the relative density of the soil particles is 2.65. Determine the porosity, soil moisture content, volumetric moisture content, and degree of saturation.

Solution:

$$G_b = \frac{7.36}{484 \times 10^{-6} \times 9810} = 1.55$$

The porosity, $n = 1 - \dfrac{G_b}{G_s}$

$$= 1 - \frac{1.55}{2.65} = 0.415 = 41.5\%$$

The soil moisture fraction,

$$W = \frac{7.94 - 7.36}{7.36} = 0.0788 = 7.88\%$$

The volumetric moisture content,

$$G_b\, W = 1.55\ (0.0788)$$
$$= 12.214\%$$

Degree of saturation, $S = \dfrac{w}{n} = \dfrac{12.214}{41.5} = 0.2943 = 29.43\%$

3.5. ROOT-ZONE SOIL WATER

Water serves the following useful functions in the process of plant growth:

(*i*) Germination of seeds,

(*ii*) All chemical reactions,

(*iii*) All biological processes,

(*iv*) Absorption of plant nutrients through their aqueous solution,

(*v*) Temperature control,

(*vi*) Tillage operations, and

(*vii*) Washing out or dilution of salts.

Crop growth (or yield) is directly affected by the soil moisture content in the root zone. The root zone is defined as the volume of soil or fractured rock occupied or occupiable by roots of the plants from which plants can extract water (3). Both excessive water (which results in waterlogging) and deficient water in the root-zone soil retard crop growth and reduce the crop yield.

Soil water can be divided into three categories:

(*i*) Gravity (or gravitational or free) water,

(*ii*) Capillary water, and

(*iii*) Hygroscopic water.

Gravity water is that water which drains away under the influence of gravity. Soon after irrigation (or rainfall) this water remains in the soil and saturates the soil, thus preventing circulation of air in void spaces.

The *capillary water* is held within soil pores due to the surface tension forces (against gravity) which act at the liquid-vapour (or water-air) interface.

Water attached to soil particles through loose chemical bonds is termed *hygroscopic water*. This water can be removed by heat only. But, the plant roots can use a very small fraction of this moisture under drought conditions.

When an oven-dry (heated to 105°C for zero per cent moisture content) soil sample is exposed to atmosphere, it takes up some moisture called *hygroscopic moisture*. If more water is made available, it can be retained as capillary moisture due to surface tension (*i.e.,* intermolecular forces). Any water, in excess of maximum capillary moisture, flows down freely and is the gravitational (or gravity) water.

The water remaining in the soil after the removal of gravitational water is called the field capacity. *Field capacity* of a soil is defined as the moisture content of a deep, permeable, and well-drained soil several days after a thorough wetting. Field capacity is measured in terms of the moisture fraction, $W_{fc} = (W_w/W_s)$ of the soil when, after thorough wetting of the soil, free drainage (at rapid rate) has essentially stopped and further drainage, if any, occurs at a very slow rate. An irrigated soil, *i.e.,* adequately wetted soil, may take approximately one (in case of sandy soil) to three (in case of clayey soil) days for the rapid drainage to stop. This condition corresponds to a surface tension of one-tenth bar (in case of sandy soils) to one-third bar (in case of clayey soils). Obviously, the field capacity depends on porosity and soil moisture tension. The volumetric moisture content at the field capacity w_{fc} becomes equal to $G_b\, W_{fc}$.

Plants are capable of extracting water from their root-zone soil to meet their transpiration demands. But, absence of further addition to the soil moisture may result in very low availability of soil water and under such a condition the water is held so tightly in the soil pores that the rate of water absorption by plants may not meet their transpiration demands and the plants may either wilt or even die, if not supplied with water immediately and well before the plants wilt. After wilting, however, a plant may not regain its strength and freshness even if the soil is saturated with water. *Permanent wilting point* is defined as the soil moisture fraction, W_{wp} at which the plant leaves wilt (or droop) permanently and applying additional water after this stage will not relieve the wilted condition. The soil moisture tension at this condition is around 15 bars (2). The moisture content at the permanent wilting condition will be higher in a hot climate than in a cold climate. Similarly, the percentage of soil moisture at the permanent wilting point of a plant will be larger in clayey soil than in sandy soil. The permanent wilting point is, obviously, at the lower end of the available moisture range and can be approximately estimated by dividing the field capacity by a factor varying from 2.0 (for soils with low silt content) to 2.4 (for soils with high silt content). The permanent wilting point also depends upon the nature of crop. The volumetric moisture content at the permanent wilting point, w_{wp} becomes G_bW_{wp}. Figure 3.2 shows different stages of soil moisture content in a soil and the corresponding conditions.

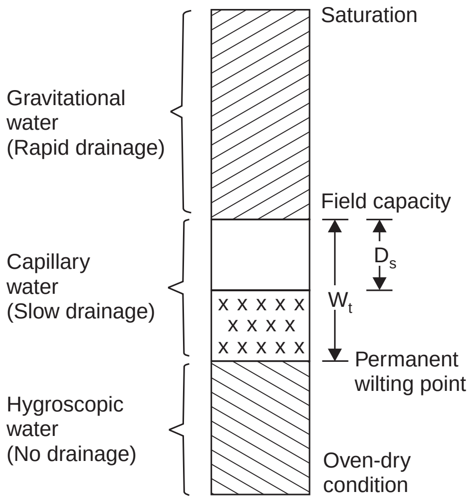


Fig. 3.2 Different stages of soil moisture content in a soil

The difference in the moisture content of the soil between its field capacity and the permanent wilting point within the root zone of the plants is termed *available moisture*. It represents the maximum moisture which can be stored in the soil for plant use. It should be noted that the soil moisture content near the wilting point is not easily extractable by the plants. Hence, the term *readily available moisture* is used to represent that fraction of the available moisture which can be easily extracted by the plants. Readily available moisture is approximately 75% of the available moisture.

The total available moisture d_t (in terms of depth) for a plant (or soil) is given by

$$d_t = (w_{fc} - w_{wp})\, d \tag{3.9}$$

in which, d is the depth of the root zone.

It is obvious that soil moisture can vary between the field capacity (excess amount would drain away) and the permanent wilting point. However, depending upon the prevailing conditions, soil moisture can be allowed to be depleted below the field capacity (but not below the permanent wilting point in any case), before the next irrigation is applied. The permissible amount of depletion is referred to as the *management allowed deficit* D_m which primarily depends on the type of crop and its stage of growth (2). Thus,

$$D_m = f_m\, d_t \tag{3.10}$$

in which, f_m is, obviously, less than 1 and depends upon the crop and its stage of growth. At a time when the soil moisture content is w, the soil-moisture deficit D_s is given as

$$D_s = (w_{fc} - w)\, d \tag{3.11}$$

Example 3.4 For the following data, calculate the total available water and the soil moisture defict.

Soil depth (cm)	G_b	W_{fc}	W_{wp}	W
0-15	1.25	0.24	0.13	0.16
15-30	1.30	0.28	0.14	0.18
30-60	1.35	0.31	0.15	0.23
60-90	1.40	0.33	0.15	0.26
90-120	1.40	0.31	0.14	0.28

Solution:

Depth of soil layers, d (mm)	$w_{fc} = G_b W_{fc}$	$w_{wp} = G_b W_{wp}$	$d_t = d \times (w_{fc} - w_{wp})$ *(mm)*	$w = G_b W$	$D_s = d \times (w_{fc} - w)$ *(mm)*
150	0.3	0.1625	20.625	0.2	15.0
150	0.364	0.182	27.300	0.234	19.5
300	0.4185	0.2025	64.800	0.3105	32.4
300	0.462	0.21	75.600	0.364	29.4
300	0.434	0.196	71.400	0.392	12.6
		Total	259.725		108.9

Example 3.5 The field capacity and permanent wilting point for a given 0.8 m root-zone soil are 35 and 10 per cent, respectively. At a given time, the soil moisture in the given soil is 20 per cent when a farmer irrigates the soil with 250 mm depth of water. Assuming bulk specific gravity of the soil as 1.6, determine the amount of water wasted from the consideration of irrigation.

Solution:

At the time of application of water,

Soil moisture deficit, $D_s = (W_{fc} - W)\, d\, G_b$

$$= (0.35 - 0.20)\,(0.8)\,(1.6)$$

$$= 0.192 \text{ m}$$

Therefore, the amount of water wasted

$$= 0.250 - 0.192$$

$$= 58 \text{ mm}$$

$$= \frac{58}{250} \times 100 = 23.2\%$$

3.6. INFILTRATION

Infiltration is another important property of soil which affects surface irrigation. It not only controls the amount of water entering the soil but also the overland flow. Infiltration is a complex process which depends on: (*i*) soil properties, (*ii*) initial soil moisture content, (*iii*) previous wetting history, (*iv*) permeability and its changes due to surface water movement, (*v*) cultivation practices, (*vi*) type of crop being sown, and (*vii*) climatic effects. In an initially dry soil, the infiltration rate is high at the beginning of rain (or irrigation), but rapidly decreases with time until a fairly steady state infiltration is reached (Fig. 3.3). This constant rate of infiltration is also termed the *basic infiltration rate* and is approximately equal to the permeability of the saturated soil.

The moisture profile under ponded infiltration into dry soil, Fig. 3.4, can be divided into the following five zones (4):

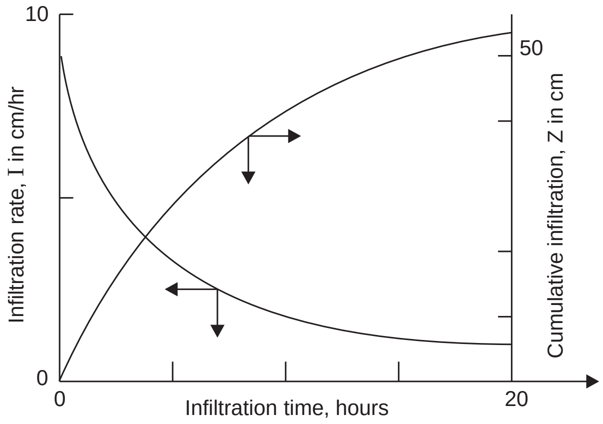


Fig. 3.3 Variation of infiltration rate, I and cumulative infiltration, Z with time

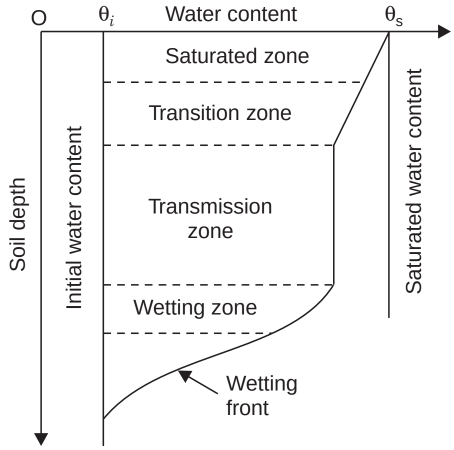


Fig. 3.4 Soil-moisture profile during ponded infiltration

(*i*) The saturated zone extending up to about 1.5 cm below the surface and having a saturated water content.

(*ii*) The transition zone which is about 5 cm thick and is located below the saturated zone. In this zone, a rapid decrease in water content occurs.

(*iii*) The transmission zone in which the water content varies slowly with depth as well as time.

(*iv*) The wetting zone in which sharp decrease in water content is observed.

(*v*) The wetting front is a region of very steep moisture gradient. This represents the limit of moisture penetration into the soil.

Table 3.1 lists the ranges of porosity, field capacity, permanent wilting point, and basic infiltration rate (or permeability) for different soil textures.

Table 3.1 Representative properties of soil

Soil texture	*Porosity (%)*	*Field capacity (%)*	*Permanent wilting point (%)*	*Basic infiltration rate (cm/hr)*
Sand	32-42	5-10	2-6	2.5-25
Sandy loam	40-47	10-18	4-10	1.3-7.6
Loam	43-49	18-25	8-14	0.8-2.0
Clay loam	47-51	24-32	11-16	0.25-1.5
Silty clay	49-53	27-35	13-17	0.03-0.5
Clay	51-55	32-40	15-22	0.01-0.1

3.7. CONSUMPTIVE USE (OR EVAPOTRANSPIRATION)

The combined loss of water from soil and crop by vaporisation is identified as evapotranspiration (3). Crops need water for transpiration and evaporation. During the growing period of a crop, there is a continuous movement of water from soil into the roots, up the stems and leaves, and out of the leaves to the atmosphere. This movement of water is essential for carrying plant food from the soil to various parts of the plant. Only a very small portion (less than 2 per cent) of water absorbed by the roots is retained in the plant and the rest of the absorbed water, after performing its tasks, gets evaporated to the atmosphere mainly through the leaves and stem. This process is called *transpiration*. In addition, some water gets evaporated to the atmosphere directly from the adjacent soil and water surfaces and from the surfaces of the plant leaves (*i.e.,* the intercepted precipitation on the plant foliage). The water needs of a crop thus consists of transpiration and evaporation and is called *evapotranspiration or consumptive use*.

Consumptive use refers to the water needs of a crop in a specified time and is the sum of the volume of transpirated and evaporated water. *Consumptive use* is defined as the amount of water needed to meet the water loss through evapotranspiration. It generally applies to a crop but can be extended to a field, farm, project or even a valley. Consumptive use is generally measured as volume per unit area or simply as the depth of water on the irrigated area. Knowledge of consumptive use helps determine irrigation requirement at the farm which should, obviously, be the difference between the consumptive use and the effective precipitation.

Evapotranspiration is dependent on climatic conditions like temperature, daylight hours, humidity, wind movement, type of crop, stage of growth of crop, soil moisture depletion, and other physical and chemical properties of soil. For example, in a sunny and hot climate, crops need more water per day than in a cloudy and cool climate. Similarly, crops like rice or sugarcane need more water than crops like beans and wheat. Also, fully grown crops need more water than crops which have been just planted.

While measuring or calculating potential evapotranspiration, it is implicitly assumed that water is freely available for evaporation at the surface. Actual evapotranspiration, in the absence of free availability of water for evaporation will, obviously, be less and is determined by: (*i*) the extent to which crop covers the soil surface, (*ii*) the stage of crop growth which affects the transpiration and soil surface coverage, and (*iii*) soil water supply.

Potential evapotranspiration is measured by growing crops in large containers, known as lysimeters, and measuring their water loss and gains. Natural conditions are simulated in these containers as closely as possible. The operator measures water added, water retained by the soil, and water lost through evapotranspiration and deep percolation. Weighings can be made with scales or by floating the lysimeters in water. Growth of roots in lysimeters confined to the dimensions of lysimeters, the disturbed soil in the lysimeters and other departures from natural conditions limit the accuracy of lysimeter measurements of potential evapotranspiration.

Potential evapotranspiration from a cropped surface can be estimated either by correlating potential evapotranspiration with water loss from evaporation devices or by estimations based on various climatic parameters. Correlation of potential evapotranspiration assumes that the climatic conditions affecting crop water loss (D_{et}) and evaporation from a free surface of water (E_p) are the same. Potential evapotranspiration D_{et} can be correlated to the pan evaporation E_p as (3),

$$D_{et} = KE_p \tag{3.12}$$

in which, K is the crop factor for that period. Pan evaporation data for various parts of India are published by the Meteorological Department. The crop factor K depends on the crop as well as its stage of growth (Table 3.2). The main limitations of this method are the differences in physical features of evaporation surfaces compared with those of a crop surface.

Table 3.2 Values of crop factor *K* from some major crops

Percentage of crop growing season since sowing	*Maize, cotton, potatoes, peas and sugarbeets*	*Wheat, barley and other small grains*	*Sugarcane*	*Rice*
0	0.20	0.08	0.50	0.80
10	0.36	0.15	0.60	0.95
25	0.75	0.33	0.75	1.10
50	1.00	0.65	1.00	1.30
75	0.85	0.90	0.85	1.15
100	0.20	0.20	0.50	0.20

In the absence of pan evaporation data, the consumptive use is generally computed as follows:

(*i*) Compute the seasonal (or monthly) distribution of potential evapotranspiration, which is defined as the evapotranspiration rate of a well-watered reference crop which completely shades the soil surface (2). It is thus an indication of the climatic evaporation demand of a vigorously growing crop. Usually, grass and alfalfa (a plant with leaves like that of clover and purple flowers used as food for horses and cattle) are taken as reference crops.

(*ii*) Adjust the potential evapotranspiration for the type of crop and the stage of crop growth. Factors such as soil moisture depletion are ignored so that the estimated values of the consumptive use are conservative values to be used for design purposes.

Thus, evapotranspiration of a crop can be estimated by multiplying potential evapotranspiration by a factor known as *crop coefficient*.

Potential evapotranspiration can be computed by one of the several methods available for the purpose. These methods range in sophistication from simple temperature correlation (such as the Blaney-Criddle formula) to equations (such as Penman's equation) which account for radiation energy as well. Blaney-Criddle formula for the consumptive use has been used extensively and is expressed as (1)

$$u = kf \tag{3.13}$$

in which, u = consumptive use of crop in mm,

k = empirical crop consumptive use coefficient (Table 3.3), and

f = consumptive use factor.

The quantities u, k, and f are determined for the same period (annual, irrigation season, growing season or monthly). The consumptive use factor f is expressed as

$$f = \frac{p}{100}(1.8t + 32) \tag{3.14}$$

in which, t = mean temperature in °C for the chosen period, and

p = percentage of daylight hours of the year occurring during the period.

Table 3.4 lists the values of p for different months of a year for 0° north latitude. The value of the consumptive use is generally determined on a monthly basis and the irrigation system must be designed for the maximum monthly water needs. It should be noted that Eq. (3.13) was originally in FPS system with appropriate values of k. Similarly, Eq. (3.14) too had a different form with t in Fahrenheit.

Table 3.3 Consumptive use coefficient for some major crops (1)

Crop	*Lenght of normal growing season or period*	*Consumptive use coefficient, k*	
		*For the growing period**	*Monthly (maximum value)***
Corn (maize)	4 months	19.05 to 21.59	20.32 to 30.48
Cotton	7 months	15.24 to 17.78	19.05 to 27.94
Potatoes	3-5 months	16.51 to 19.05	21.59 to 25.40
Rice	3-5 months	25.40 to 27.94	27.94 to 33.02
Small grains	3 months	19.05 to 21.59	21.59 to 25.40
Sugarbeet	6 months	16.51 to 19.05	21.59 to 25.40
Sorghums	4-5 months	17.78 to 20.32	21.59 to 25.40
Orange and lemon	1 year	11.43 to 13.97	16.21 to 19.05

*The lower values are for more humid areas and the higher values are for more arid climates.

** Dependent upon mean monthly temperature and stage of growth of crop.

Table 3.4 Per cent daylight hours for northern hemispere (0-50° latitude) (1)

Latitude North (in degr-ees)	*Jan.*	*Fab.*	*March*	*April*	*May*	*June*	*July*	*Aug.*	*Sep.*	*Oct.*	*Nov.*	*Dec.*
0	8.50	7.66	8.49	8.21	8.50	8.22	8.50	8.49	8.21	8.50	8.22	8.50
5	8.32	7.57	8.47	8.29	8.65	8.41	8.67	8.60	8.23	8.42	8.07	8.30
10	8.13	7.47	8.45	8.37	8.81	8.60	8.86	8.71	8.25	8.34	7.91	8.10
15	7.94	7.36	8.43	8.44	8.98	8.80	9.05	8.83	8.28	8.26	7.75	7.88

(Contd.)...

20	7.74	7.25	8.41	8.52	9.15	9.00	9.25	8.96	8.30	8.18	7.58	7.66
25	7.53	7.14	8.39	8.61	9.33	9.23	9.45	9.09	8.32	8.09	7.40	7.42
30	7.30	7.03	8.38	8.72	9.53	9.49	9.67	9.22	8.33	7.99	7.19	7.15
32	7.20	6.97	8.37	8.76	9.62	9.59	9.77	9.27	8.34	7.95	7.11	7.05
34	7.10	6.91	8.36	8.80	9.72	9.70	9.88	9.33	8.36	7.90	7.02	6.92
36	6.99	6.85	8.35	8.85	9.82	9.82	9.99	9.40	8.37	7.85	6.92	6.79
38	6.87	6.79	8.34	8.90	9.92	9.95	10.10	9.47	8.38	7.80	6.82	6.66
40	6.76	6.72	8.33	8.95	10.02	10.08	10.22	9.54	8.39	7.75	6.72	6.52
42	6.63	6.65	8.31	9.00	10.14	10.22	10.35	9.62	8.40	7.69	6.62	6.37
44	6.49	6.58	8.30	9.06	10.26	10.38	10.49	9.70	8.41	7.63	6.49	6.21
46	6.34	6.50	8.29	9.12	10.39	10.54	10.64	9.79	8.42	7.57	6.36	6.04
48	6.17	6.41	8.27	9.18	10.53	10.71	10.80	9.89	8.44	7.51	6.23	5.86
50	5.98	6.30	8.24	9.24	10.68	10.91	10.99	10.00	8.46	7.45	6.10	5.65

Table 3.5 gives typical values of the water needs of some major crops for the total growing period of some of the crops (5). This table also indicates the sensitivity of the crop to water shortages or drought. High sensitivity to drought means that the crop cannot withstand water shortages, and that such shortages should be avoided.

Table 3.5 Indicative values of crop water needs and sensitivity to drought (5)

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

Example 3.6 Using the Blaney-Criddle formula, estimate the yearly consumptive use of water for sugarcane for the data given in the first four columns of Table 3.6.

Solution:

According to Eqs. (3.13) and (3.14),

$$u = k\frac{p}{100}(1.8\,t + 32)$$

Values of monthly consumptive use calculated from the above formula have been tabulated in the last column of Table 3.6. Thus, yearly consumptive use = Σu = 1.75 m.

Table 3.6 Data and solution for Example 3.6

Month	*Mean monthly temperature, t°C*	*Monthly crop coefficient, k*	*Per cent sunshine hours, p*	*Monthly consumptive use, u (mm)*
January	13.10	19.05	7.38	78.14
February	15.70	20.32	7.02	85.96
March	20.70	21.59	8.39	125.46
April	27.00	21.59	8.69	151.22
May	31.10	22.86	9.48	190.66
June	33.50	24.13	9.41	209.58
July	30.60	25.40	9.60	212.34
August	29.00	25.40	9.60	205.31
September	28.20	24.13	8.33	166.35
October	24.70	22.86	8.01	140.01
November	18.80	21.59	7.25	103.06
December	13.70	19.05	7.24	78.15

3.8. IRRIGATION REQUIREMENT

Based on the consumptive use, the growth of all plants can be divided into three stages, *viz.*, vegetative, flowering, and fruiting. The consumptive use continuously increases during the vegetative stage and attains the peak value around the flowering stage; thereafter, the consumptive use decreases. It should be noted that different crops are harvested during different stages of crop growth. For example, leafy vegetables are harvested during the vegetative stage and flowers are harvested during the flowering stage. Most crops (such as potatoes, rice, corn, beans, bananas, *etc*.) are harvested during the fruiting stage.

At each precipitation, a certain volume of water is added to the crop field. Not all of the rainfall can be stored within the root zone of the soil. The part of the precipitation which has gone as surface runoff, percolated deep into the ground or evaporated back to the atomosphere does not contribute to the available soil moisture for the growth of crop. Thus, *effective precipitation* is only that part of the precipitation which contributes to the soil moisture available for plants. In other words, the effective rainfall is the water retained in the root zone and is obtained by subtracting the sum of runoff, evaporation, and deep percolation from the total rainfall.

If, for a given period, the consumptive use exceeds the effective precipitation, the difference has to be met by irrigation water. In some cases irrigation water has to satisfy leaching requirements too. Further, some of the water applied to the field necessarily flows away as surface runoff and/or percolates deep into the ground and/or evaporates to the atmosphere. Therefore, irrigation requirement is the quantity of water, exclusive of precipitation and regardless of its source, required by a crop or diversified pattern of crops in a given period of time of their normal growth under field conditions. It includes evapotranspiration not met by effective precipitation and other economically unavoidable losses such as surface runoff and deep percolation. Irregular land surfaces, compact impervious soils or shallow soils over a gravel stratum of high permeability, small or too large irrigation streams, absence of an attendant during irrigation, long irrigation runs, improper land preparation, steep ground slopes and such other factors contribute to large losses of irrigation water which, in turn, reduce irrigation efficiency. Irrigation efficiency is the ratio of irrigation water consumed by crops of an irrigated field to the water diverted from the source of supply. Irrigation efficiency is usually measured at the field entrance (3). Water application efficiency is the ratio of the average depth added to the root-zone storage to the average depth applied to the field. Obviously, irrigation efficiency measured at the field and the water application efficiency would be the same. Thus, the field irrigation requirement FIR is expressed as (2)

$$\text{FIR} = \frac{D_{et} - (D_p - D_{pl})}{E_a} \tag{3.15}$$

in which, D_{et} = depth of evapotranspiration,

D_p = depth of precipitation,

D_{pl} = depth of precipitation that goes as surface runoff and/or infiltrates into the ground and/or intercepted by the plants,

and E_a = irrigation efficiency or application efficiency.

In the absence of any other information, the following values can be used as a guide for E_a in different methods of surface irrigation for different types of soils:

Soil class	*Irrigation efficiency (%)*
Sand	60
Sandy loam	65
Loam	70
Clay loam	75
Heavy clay	80

If no other information is available, the following formulae can be used to estimate the effective rainfall depth, D_{pe} provided that the ground slope does not exceed 5%.

$$D_{pe} = 0.8\, D_p - 25 \qquad \text{if } D_p > 75 \text{ mm/month}$$

$$D_{pe} = 0.6\, D_p - 10 \qquad \text{if } D_p < 75 \text{ mm/month}$$

D_{pe} is always equal to or greater than zero and never negative. Both D_p ad D_{pe} are in mm/month in the foregoing formulae.

Example 3.7 Using the data given in the first four columns of Table 3.7 for a given crop, determine the field irrigation requirement for each month assuming irrigation efficiency to be 60 per cent.

Table 3.7 Data and solution for Example 3.5

Month	*Crop factor, K*	*Pan evaporation, E_p (mm)*	*Effective rainfall, $D_p - D_{pl}$ (mm)*	*Consumptive use, D_{et} (mm)*	*FIR (mm)*
November	0.20	118.0	6.0	23.60	29.33
December	0.36	96.0	16.0	34.56	30.93
January	0.75	90.0	20.0	67.50	79.17
February	0.90	105.0	15.0	94.50	132.50
March	0.80	140.0	2.0	112.00	183.33

Solution:

According to Eqs. (3.12) and (3.15)

$$D_{et} = KE_p$$

and

$$\text{FIR} = \frac{D_{et} - (D_p - D_{pl})}{E_a}$$

Given $E_a = 0.6$

Field irrigation requirement calculated for each month of the crop-growing season has been tabulated in the last column of Table 3.7.

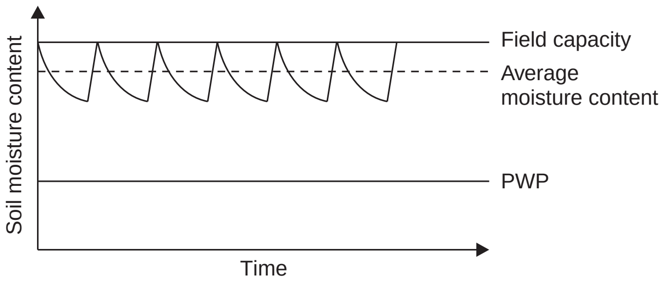
3.9. FREQUENCY OF IRRIGATION

Growing crops consume water continuously. However, the rate of consumption depends on the type of crop, its age, and the atmospheric conditions all of which are variable factors. The aim of each irrigation is to fulfil the needs of the crop for a period which may vary from few days to several weeks. The frequency of irrigation primarily depends on: (*i*) the water needs of the crop, (*ii*) the availability of water, and (*iii*) the capacity of the root-zone soil to store water. Shallow-rooted crops generally require more frequent irrigation than deep-rooted crops. The roots of a plant in moist soil extract more water than the roots of the same plant in drier soil.

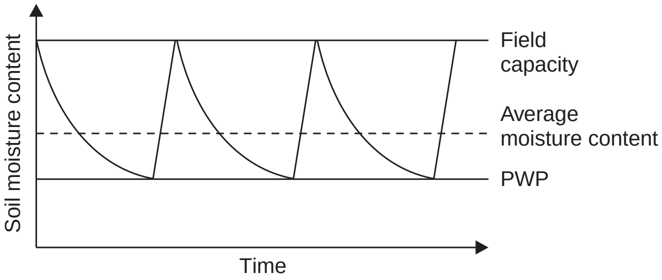
A moderate quantity of soil moisture is beneficial for good crop growth. Both excessive and deficient amount of soil moisture retard the crop growth and thus the yield. Excessive flooding drives out air which is essential for satisfactory crop growth. In case of deficient moisture, the plant has to spend extra energy to extract the desired amount of water.

Many of the crops have an optimum soil moisture content at which the yield is maximum; if the moisture content is less or more than this amount, the yield reduces. Wheat has a well-defined optimum moisture content of around 40 cm. However, there are other crops in which the yield initially increases at a much faster rate with the increase in the soil moisture content and the rate of increase of the yield becomes very small at higher moisture content. In such cases, the soil moisture is kept up to a level beyond which the increase in production is not worth the cost of the additional water supplied.

It should be noted that, because of the capacity of a soil to store water, it is not necessary to apply water to the soil every day even though the consumptive use takes place continuously. The soil moisture can vary between the field capacity and the permanent wilting point. The average moisture content will thus depend on the frequency of irrigation and quantity of water applied. As can be seen from Fig. 3.5, frequent irrigation (even of smaller depths) keeps the average mositure content closer to the field capacity. On the other hand, less frequent irrigation of larger depths of water will keep the average moisture content on the lower side.



(*a*) MORE FREQUENT IRRIGATION



(*b*) LESS FREQUENT IRRIGATION

Fig. 3.5 Effect of frequency of irrigation on average moisute content

For most of the crops, the yield remains maximum if not more than 50 per cent of the available water is removed during the vegetative, flowering, and the intitial periods of the fruiting stage. During the final period of the fruiting stage, 75 per cent of the available moisture can be depleted without any adverse effect on the crop yield.

The frequency of irrigation (or irrigation interval) is so decided that the average moisture content is close to the optimum and at each irrigation the soil moisture content is brought to the field capacity. Alternatively, the frequency of irrigation can be decided so as to satisfy the daily consumptive use requirement which varies with stage of growth. Thus, frequency of irrigation is calculated by dividing the amount of soil moisture which may be depleted (*i.e.,* allowable depletion below field capacity and well above permanent wilting point) within the root-zone soil by the rate of consumptive use. Thus,

$$\text{Frequency of irrigation} = \frac{\text{Allowable soil moisture depletion}}{\text{Rate of consumptive use}} \tag{3.16}$$

The depth of watering at each irrigation to bring the moisture content w to the field capacity w_{fc} in a soil of depth d can be determined from the following relation:

$$\text{Depth of water to be applied} = \frac{(w_{fc} - w)d}{E_a} \tag{3.17}$$

Example 3.8 During a particular stage of the growth of a crop, consumptive use of water is 2.8 mm/day. Determine the interval in days between irrigations, and depth of water to be applied when the amount of water available in the soil is: (*i*) 25%, (*ii*) 50% (*iii*) 75%, and (*iv*) 0% of the maximum depth of available water in the root zone which is 80 mm. Assume irrigation efficiency to be 65%.

Solution:

(*i*) Frequency of irrigation $= \dfrac{80 \times (1 - 0.25)}{2.8}$

$= 21.43$ days

$= 21$ days (say)

(*ii*) Depth of water to be applied $= \dfrac{80 \times (1 - 0.25)}{0.65}$

$= 92.31$ mm

$= 93.00$ mm (say).

Other calculations have been shown in the following table:

	Amount of soil moisture depleted to			
	25%	50%	75%	0%
Frequency of irrigation (days)	21	14	7	28
Depth of water to be applied (mm)	93	62	31	124

3.10. METHODS OF IRRIGATION

Any irrigation system would consist of the following four subsystems (2):

(*i*) The water supply subsystem which may include diversion from rivers or surface ponds or pumped flow of ground water.

(*ii*) The water delivery subsystem which will include canals, branches, and hydraulic structures on these.

(*iii*) The water use subsystems, which can be one of the four main types, namely, (*a*) surface irrigation, (*b*) subsurface irrigation, (*c*) sprinkler irrigation, and (*d*) trickle irrigation.

(*iv*) The water removal system *i.e.,* the drainage system.

In this section, the water use subsystems have been described.

3.10.1. Water Use Subsystems

Irrigation water can be applied to croplands using one of the following irrigation methods (1):

(*i*) Surface irrigation which includes the following:

(*a*) Uncontrolled (or wild or free) flooding method,

(*b*) Border strip method,

(*c*) Check method,

(*d*) Basin method, and

(*e*) Furrow method.

(*ii*) Subsurface irrigation

(*iii*) Sprinkler irrigation

(*iv*) Trickle irrigation

Each of the above methods has some advantages and disadvantages, and the choice of the method depends on the following factors (2):

(*i*) Size, shape, and slope of the field,

(*ii*) Soil characteristics,

(*iii*) Nature and availability of the water supply subsystem,

(*iv*) Types of crops being grown,

(*v*) Initial development costs and availability of funds, and

(*vi*) Preferences and past experience of the farmer.

The design of an irrigation system for applying water to croplands is quite complex and not amenable to quantitative analysis. Principal criteria for the design of a suitable irrigation method are as follows (3):

(*i*) Store the required water in the root-zone of the soil,

(*ii*) Obtain reasonably uniform application of water,

(*iii*) Minimise soil erosion,

(*iv*) Minimise run-off of irrigation water from the field,

(*v*) Provide for beneficial use of the runoff water,

(*vi*) Minimise labour requirement for irrigation,

(*vii*) Minimise land use for ditches and other controls to distribute water,

(*viii*) Fit irrigation system to field boundaries,

(*ix*) Adopt the system to soil and topographic changes, and

(*x*) Facilitate use of machinery for land preparation, cultivating, furrowing, harvesting, and so on.

3.10.2. Surface Irrigation

In all the surface methods of irrigation, water is either ponded on the soil or allowed to flow continuously over the soil surface for the duration of irrigation. Although surface irrigation is the oldest and most common method of irrigation, it does not result in high levels of performance. This is mainly because of uncertain infiltration rates which are affected by year-to-year changes in the cropping pattern, cultivation practices, climatic factors, and many other factors. As a result, correct estimation of irrigation efficiency of surface irrigation is difficult. Application efficiencies for surface methods may range from about 40 to 80 per cent.

(*a*) *Uncontrolled Flooding*

When water is applied to the cropland without any preparation of land and without any levees to guide or restrict the flow of water on the field, the method is called 'uncontrolled', wild or 'free' flooding. In this method of flooding, water is brought to field ditches and then admitted at one end of the field thus letting it flood the entire field without any control.

Uncontrolled flooding generally results in excess irrigation at the inlet region of the field and insufficient irrigation at the outlet end. Application efficiency is reduced because of either deep percolation (in case of longer duration of flooding) or flowing away of water (in case of shorter flooding duration) from the field. The application efficiency would also depend on the depth of flooding, the rate of intake of water into the soil, the size of the stream, and topography of the field.

Obviously, this method is suitable when water is available in large quantities, the land surface is irregular, and the crop being grown is unaffected because of excess water. The advantage of this method is the low initial cost of land preparation. This is offset by the disadvantage of greater loss of water due to deep percolation and surface runoff.

(*b*) *Border Strip Method*

Border strip irrigation (or simply 'border irrigation') is a controlled surface flooding method of applying irrigation water. In this method, the farm is divided into a number of strips which can be 3-20 metres wide and 100-400 metres long. These strips are separated by low levees (or borders). The strips are level between levees but slope along the length according to natural slope. If possible, the slope should be between 0.2 and 0.4 per cent. But, slopes as flat as 0.1 per cent and as steep as 8 per cent can also be used (1). In case of steep slope, care should be taken to prevent erosion of soil. Clay loam and clayey soils require much flatter slopes (around 0.2%) of the border strips because of low infiltration rate. Medium soils may have slopes ranging from 0.2 to 0.4%. Sandy soils can have slopes ranging from 0.25 to 0.6%.

Water from the supply ditch is diverted to these strips along which it flows slowly towards the downstream end and in the process it wets and irrigates the soil. When the water supply is stopped, it recedes from the upstream end to the downsteam end.

The border strip method is suited to soils of moderately low to moderately high intake rates and low erodibility. This method is suitable for all types of crops except those which require prolonged flooding which, in this case, is difficult to maintain because of the slope. This method, however, requires preparation of land involving high initial cost.

(*c*) *Check Method*

The check method of irrigation is based on rapid application of irrigation water to a level or nearly level area completely enclosed by dikes. In this method, the entire field is divided into a number of almost levelled plots (compartments or '*Kiaries*') surrounded by levees. Water is admitted from the farmer's watercourse to these plots turn by turn. This method is suitable for a wide range of soils ranging from very permeable to heavy soils. The farmer has very good control over the distribution of water in different areas of his farm. Loss of water through deep percolation (near the supply ditch) and surface runoff can be minimised and adequate irrigation of the entire farm can be achieved. Thus, application efficiency is higher for this method. However, this method requires constant attendance and work (allowing and closing the supplies to the levelled plots). Besides, there is some loss of cultivable area which is occupied by the levees. Sometimes, levees are made sufficiently wide so that some 'row' crops can be grown over the levee surface.

(*d*) *Basin Method*

This method is frequently used to irrigate orchards. Generally, one basin is made for one tree. However, where conditions are favourable, two or more trees can be included in one basin.

(*e*) *Furrow Method*

In the surface irrigation methods discussed above, the entire land surface is flooded during each irrigation. An alternative to flooding the entire land surface is to construct small channels along the primary direction of the movement of water and letting the water flow through these channels which are termed 'furrows', 'creases' or 'corrugation'. Furrows are small channels having a continuous and almost uniform slope in the direction of irrigation. Water infiltrates through the wetted perimeter of the furrows and moves vertically and then laterally to saturate the soil. Furrows are used to irrigate crops planted in rows.

Furrow lengths may vary from 10 metres to as much as 500 metres, although, 100 metres to 200 metres are the desirable lengths and more common. Very long furrows may result in excessive deep percolation losses and soil erosion near the upstream end of the field. Preferable slope for furrows ranges between 0.5 and 3.0 per cent. Many different classes of soil have been satisfactorily irrigated with furrow slope ranging from 3 to 6 per cent (1). In case of steep slopes, care should be taken to control erosion. Spacing of furrows for row crops (such as corn, potatoes, sugarbeet, *etc*.) is decided by the required spacing of the plant rows. The furrow stream should be small enough to prevent the flowing water from coming in direct contact with the plant. Furrows of depth 20 to 30 cm are satisfactory for soils of low permeability. For other soils, furrows may be kept 8 to 12 cm deep.

Water is distributed to furrows from earthen ditches through small openings made in earthen banks. Alternatively, a small-diameter pipe of light weight plastic or rubber can be used to siphon water from the ditch to the furrows without disturbing the banks of the earthen ditch.

Furrows necessitate the wetting of only about half to one-fifth of the field surface. This reduces the evaporation loss considerably. Besides, puddling of heavy soils is also lessened and it is possible to start cultivation soon after irrigation. Furrows provide better on-farm water management capabilities for most of the surface irrigation conditions, and variable and severe topographical conditions. For example, with the change in supply conditions, number of simultaneously supplied furrows can be easily changed. In this manner, very high irrigation efficiency can be achieved.

The following are the disadvantages of furrow irrigation:

(*i*) Possibility of increased salinity between furrows,

(*ii*) Loss of water at the downstream end unless end dikes are used,

(*iii*) The necessity of one extra tillage work, *viz.,* furrow construction,

(*iv*) Possibility of increased erosion, and

(*v*) Furrow irrigation requires more labour than any other surface irrigation method.

3.10.3. Subsurface Irrigation

Subsurface irrigation (or simply subirrigation) is the practice of applying water to soils directly under the surface. Moisture reaches the plant roots through capillary action. The conditions which favour subirrigation are as follows (1):

(*i*) Impervious subsoil at a depth of 2 metres or more,

(*ii*) A very permeable subsoil,

(*iii*) A permeable loam or sandy loam surface soil,

(*iv*) Uniform topographic conditions, and

(*v*) Moderate ground slopes.

In natural subirrigation, water is distributed in a series of ditches about 0.6 to 0.9 metre deep and 0.3 metre wide having vertical sides. These ditches are spaced 45 to 90 metres apart.

Sometimes, when soil conditions are favourable for the production of cash crops (*i.e.,* high-priced crops) on small areas, a pipe distribution system is placed in the soil well below the surface. This method of applying water is known as artificial subirrigation. Soils which permit free lateral movement of water, rapid capillary movement in the root-zone soil, and very slow downward movement of water in the subsoil are very suitable for artificial subirrigation. The cost of such methods is very high. However, the water consumption is as low as one-third of the surface irrigation methods. The yield also improves. Application efficiency generally varies between 30 and 80 per cent.

3.10.4. Sprinkler Irrigation

Sprinkling is the method of applying water to the soil surface in the form of a spray which is somewhat sililar to rain. In this method, water is sprayed into the air and allowed to fall on the soil surface in a uniform pattern at a rate less than the infiltration rate of the soil. This method started in the beginning of this century and was initially limited to nurseries and orchards. In the beginning, it was used in humid regions as a supplemental method of irrigation. This method is popular in the developed countries and is gaining popularity in the developing countries too.

Rotating sprinkler-head systems are commonly used for sprinkler irrigation. Each rotating sprinkler head applies water to a given area, size of which is governed by the nozzle size and the water pressure. Alternatively, perforated pipe can be used to deliver water through very small holes which are drilled at close intervals along a segment of the circumference of a pipe. The trajectories of these jets provide fairly uniform application of water over a strip of cropland along both sides of the pipe. With the availability of flexible PVC pipes, the sprinkler systems can be made portable too.

Sprinklers have been used on all types of soils on lands of different topography and slopes, and for many crops. The following conditions are favourable for sprinkler irrigation (1):

(*i*) Very previous soils which do not permit good distribution of water by surface methods,

(*ii*) Lands which have steep slopes and easily erodible soils,

(*iii*) Irrigation channels which are too small to distribute water efficiently by surface irrigation, and

(*iv*) Lands with shallow soils and undulating lands which prevent proper levelling required for surface methods of irrigation.

Besides, the sprinkler system has several features. For example, small amounts of water can be applied easily and frequently by the sprinkler system. Light and frequent irrigations are very useful during the germination of new plants, for shallow-rooted crops and to control soil temperature. Measurement of quantity of water is easier. It causes less interference in cultivation and other farming operations. While sprinkler irrigation reduces percolation losses, it increases evaporation losses. The frequency and intensity of the wind will affect the efficiency of any sprinkler system. Sprinkler application efficiencies should always be more than 75 per cent so that the system is economically viable.

The sprinkler method is replacing the surface/gravity irrigation methods in all developed countries due to its higher water application/use efficiency, less labour requirements, adaptability to hilly terrain, and ability to apply fertilizers in solution. In India too, the gross area under sprinkler irrigation has increased from 3 lakh hectares in 1985 to 5.80 lakh hectares in 1989. The total number of sprinkler sets in India now exceeds one lakh.

3.10.5. Trickle Irrigation

Trickle irrigation (also known as drip irrigation) system comprises main line (37.5 mm to 70 mm diameter pipe), submains (25 mm to 37.5 mm diameter pipe), laterals (6 mm to 8 mm diameter pipe), valves (to control the flow), drippers or emitters (to supply water to the plants), pressure gauges, water meters, filters (to remove all debris, sand and clay to reduce clogging of the emitters), pumps, fertiliser tanks, vacuum breakers, and pressure regulators. The drippers are designed to supply water at the desired rate (1 to 10 litres per hour) directly to the soil. Low pressure heads at the emitters are considered adequate as the soil capillary forces cause the emitted water to spread laterally and vertically. Flow is controlled manually or set to automatically either (*i*) deliver desired amount of water for a predetermined time, or (*ii*) supply water whenever soil moisture decreases to a predetermined amount. A line sketch of a typical drip irrigation system is shown in Fig. 3.6. Drip irrigation has several advantages. It saves water, enhances plant growth and crop yield, saves labour and energy, controls weed growth, causes no erosion of soil, does not require land preparation, and also improves fertilizer application efficiency. However, this method of irrigation does have some economic and technical limitations as it requires high skill in design, installation, and subsequent operation.

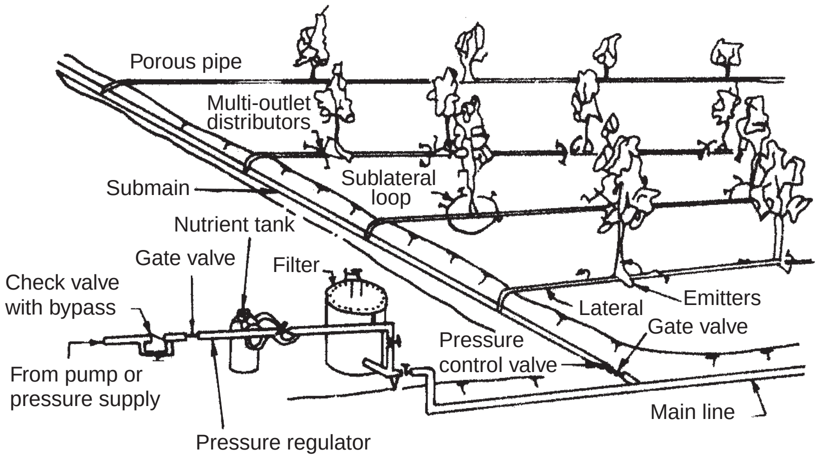


Fig. 3.6 Line sketch of a typical drip irrigation system

Trickle irrigation enables efficient water application in the root zone of small trees and widely spaced plants without wetting the soil where no roots exist. In arid regions, the irrigation efficiency may be as high as 90 per cent and with very good management it may approach the ideal value of 100 per cent. The main reasons for the high efficiency of trickle irrigation are its capability to produce and maintain continuously high soil moisture content in the root zone and the reduction in the growth of weeds (due to limited wet surface area) competing with the crop for water and nutrients. Insect, disease, and fungus problems are also reduced by minimising the wetting of the soil surface.

Due to its ability to maintain a nearly constant soil moisture content in the root zone, Fig. 3.7, trickle irrigation results in better quality and greater crop yields. Fruits which contain considerable moisture at the time of harvesting (such as tomatoes, grapes, berries, *etc*.) respond very well to trickle irrigation. However, this method is not at all suitable (from practical as well as economic considerations) for closely planted crops such as wheat and other cereal grains.

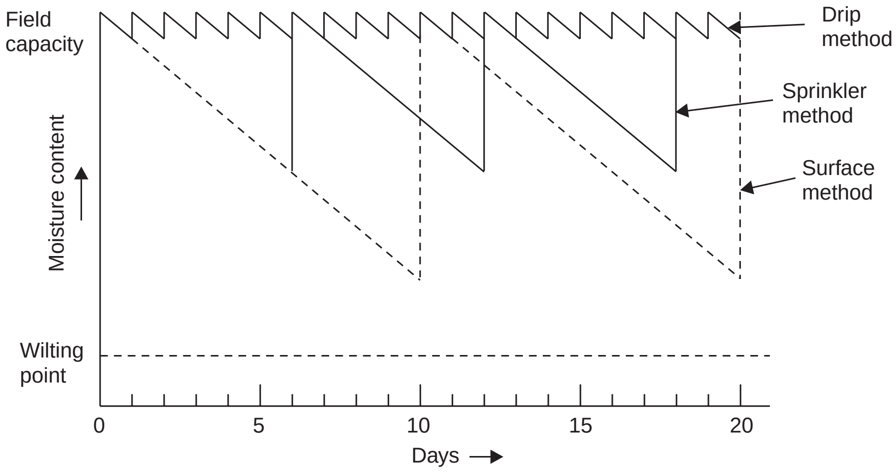


Fig. 3.7 Moisture availability for crops in different irrigation methods

One of the major problems of trickle irrigation is the clogging of small conduits and openings in the emitters due to sand and clay particles, debris, chemical precipitates, and organic growth. In trickle irrigation, only a part of the soil is wetted and, hence it must be ensured that the root growth is not restricted. Another problem of trickle irrigation is on account of the dissolved salt left in the soil as the water is used by the plants. If the rain water flushes the salts near the surface down into the root zone, severe damage to the crop may result. In such situations, application of water by sprinkler or surface irrigation may become necessary.

Because of the obvious advantages of water saving and increased crop yield associated with the drip irrigation, India has embarked on a massive programme for popularising this method. The area under drip irrigation in India is about 71000 hectares against a world total of about 1.8 million hectares (6). The area coverage is the highest in Maharashtra. (about 33000 hectares) followed by Andhra Pradesh and Karnataka. Cost of drip irrigation system in India varies from about Rs. 15000 to 40000 per hectare. The benefit-cost ratio (excluding the benefit of saving in water) for drip irrigation system varies between 1.3 to 2.6. However, for grapes this ratio is much higher and may be as high as 13.

3.11. QUALITY OF IRRIGATION WATER

Needs of a healthy human environment place high demands on the quality of irrigation water. Irrigation water must not have direct or indirect undesirable effects on the health of human beings, animals, and plants. The irrigation water must not damage the soil and not endanger the quality of surface and ground waters with which it comes into contact. The presence of toxic substances in irrigation water may threaten the vegetation besides degrading the suitability of soil for future cultivation. Surface water, ground water, and suitably treated waste waters are generally used for irrigation purposes. In examining the quality of irrigation water, attention is focussed on the physical, chemical, and biological properties of such water.

The effect of undissolved substances in irrigation water on the soil fertility depends on their size, quantity, and nutrient content as well as the type of soil. Fine-grained soil particles in irrigation water may improve the fertility of light soils, but may adversely affect the permeability and aeration characteristics of heavier soils. The undissolved substances may settle in the irrigation systems and thus result either in the reduction of their capacity or even failure of some installations such as pumping plants. The use of water having upleasant odour for irrigation may unfavourably affect the farmers.

The quality of irrigation water depends mainly on the type and content of dissolved salts. The problem arises when the total salt content of irrigation water is so high that the salts accumulate in the root zone. The plant then draws water and nutrients from the saline soil with great difficulty which affects the plant growth. The presence of toxic salts is harmful to the plants.

The suitability of irrigation water from biological considerations is usually decided by the type and extent of biological animation.

Special care should be taken when waste water is to be used for irrigation. Waste waters for irrigation can be classified as municipal wastes, industrial wastes, and agricultural wastes. Waste waters selected for irrigation must be suitable and their use must be permissible from the sanitary and agricultural considerations as well as from the point of view of smooth operation of the irrigation system. Fields should be located in the immediate vicinity of the waste water resources. The requirement of a minimum distance of about 200 m between the irrigated area and residential buildings must be observed. Also, at a wind velocity of more than 3.5 m/s, sprinkler irrigation, using waste water, should not be used.

EXCERCISES

3.1 Describe important physical and chemical properties of soil which are important from considerations of irrigation.

3.2 What is the meaning of consumptive use? On what factors does it depend? How would one calculate the consumptive use for a given crop?

3.3 The field capacity and permanent wilting point for a given soil are 35 and 15 per cent, respectively. Determine the storage capacity of soil within the root zone of the soil which may be taken as 80 cm. At a given time the soil moisture in the field is 20 per cent and a farmer applies 25.0 cm of water. What part of this water would be wasted? Assume porosity of soil as 40 per cent and relative density as 2.65.

3.4 Determine the frequency of irrigation for the following data:

Field capacity of soil	30%
Permanent wilting point	10%
Management allowed deficit	45%
Effective root zone depth	0.75 m
Consumptive use	12 mm/day
Apparent specific gravity of soil (including the effect of porosity)	1.6

3.5 The consumptive use for a given crop is 90 mm. Determine the field irrigation requirement if the effective rainfall and the irrigation efficiency in the area are 15 mm and 60 per cent, respectively.

3.6 The following data were obtained in determining the soil moisture content at successive depths in the root zone prior to applying irrigation water:

Depth of sampling	*Weight of moist soil sample*	*Weight of oven-dried soil sample*
0-25 cm	1.35 N	1.27 N
25-50 cm	1.36 N	1.26 N
50-75 cm	1.23 N	1.15 N
75-100 cm	1.11 N	1.02 N

The bulk specific gravity of the soil in the root zone is 1.5. The available moisture-holding capacity of the soil is 17.8 cm/m depth of soil.

Determine (*i*) the moisture content at different depths in the root zone, (*ii*) the moisture content in the entire root zone prior to irrigation, and (*iii*) depth of water to be applied to bring the moisture content to the field capacity.

3.7 For the following data pertaining to a cultivated land, determine irrigation interval and amount of irrigation water needed at each irrigation so that the moisture content at any stage does not fall below 40 per cent of the maximum available moistures.

Field capacity of soil	= 35%
Permanent wilting point	= 12%
Porosity of soil	= 0.42
Depth of root-zone soil	= 1.20 m
Consumptive use	= 12 mm per day
Application efficiency	= 60%

3.8 Describe various methods of irrigation mentioning their advantages, disadvantages and applicability to different field conditions.

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4

GROUND WATER AND WELLS

4.1. GROUND WATER RESOURCES

The amount of water stored in the earth's crust may be of the order of 8 billion cubic kilometres, half of which is at depths less than 800 m (1). This water inside the earth is about 35 times the combined storage of all the world's rivers, fresh water lakes, reservoirs, and inland seas, and is about one-third the volume of water stored in the arctic and antarctic ice fields, the glaciers of Greenland, and the great mountain systems of the world (2). All of this ground water, however, cannot be utilised because of physiographic limitations.

The estimate of the present ground water resource in India (3) is of the order of 650 cubic kilometres (as against 1880 cubic km for surface water resources), out of which utilisable ground water is assessed at around 420 cubic km (as against 690 cubic km for surface water resources); see Table 1.4. *Ground water* is that part of the subsurface water which occurs within the saturated zone of the earth's crust where all pores are filled with water (2). Ground water has also been referred to as that part of the subsurface water which can be lifted or which flows naturally to the earth's surface. A hole or shaft, usually vertical, is excavated in the earth to lift ground water to the earth's surface and is termed a well. A well can also be used for disposal of water, artificial recharge, draining out agricultural lands, and relieving pressures under hydraulic structures. The Chinese are known to be the first to have drilled deep wells using bamboo rods tipped with iron (2). The rods were lifted and dropped manually and the method was similar to the method now known as cable tool drilling. Ground water flows to the earth's surface through naturally discharging springs and streams and rivers which are sustained by ground water itself when overland runoff is not present. Following significant features of ground water should always be kept in mind while managing ground water (2):

(*i*) Ground water is a huge water resource, but is exhaustible and is unevenly available.

(*ii*) Ground water and surface water resources are interrelated and, hence, should be considered together.

(*iii*) Excessive and continued exploitation of ground water must be avoided as natural replenishment of the ground water resource is a very slow process.

(*iv*) Ground water is generally better than surface water in respect of biological characteristics. On the other hand, surface water is generally better than ground water in terms of chemical characteristics.

(*v*) Ground water may be developed in stages on "pay-as-you-go" or "pay-as-you-grow" basis. Surface water development usually needs large initial capital investment.

(*vi*) Underground reservoirs storing ground water are more advantageous than surface reservoirs.

(*a*) There is no construction cost involved in underground reservoirs. But, well construction, pumps and energy for pumping water, and maintenance of pumps and wells require money.

(*b*) Underground reservoirs do not silt up, but surface siltation of recharge areas may appreciably reduce recharge rates.

(*c*) The evaporation from underground reservoirs is much less.

(*d*) Underground reservoirs do not occupy the land surface which may be useful for some other purposes.

(*vii*) Ground water is generally of uniform temperature and mineral quality and is free of suspended impurities.

(*viii*) Ground water source has indefinite life, if properly managed.

Ground water source is replenished through the processes of infiltration and percolation. *Infiltration* is the process by which the precipitation and surface water move downward into the soil. *Percolation* is the vertical and lateral movement through the various openings in the geological formations. Natural sources of replenishment include rainwater, melting snow or ice and water in stream channels, and lakes or other natural bodies of water. Rainwater may infiltrate into the ground directly or while flowing over the land enroute to a river, or stream, or other water bodies. Artificial sources of replenishment (or recharge) include the following (2):

(*i*) Leakage from reservoirs, conduits, septic tanks, and similar water related structures.

(*ii*) Irrigation, or other water applications including deliberate flooding of a naturally porous area.

(*iii*) Effluents discharged to evaporation or percolation ponds.

(*iv*) Injection through wells or other similar structures.

4.2. WELL IRRIGATION

In view of the large amount of utilisable ground water, higher agricultural yield of tubewell-irrigated lands in comparison to that of canal-irrigated lands (see Table 1.3), and favourable impact of its use on waterlogging, it is only logical to develop ground water resources for irrigation and other activities. Most of the existing canal systems in India are of a protective nature, *i.e.,* they provide protection against famine. They were not designed to promote intensive farming. Well irrigation ensures more reliable irrigation and, therefore, enables the farmers to grow more remunerative crops with improved yield. The following are the main requirements for the success of well irrigation:

(*i*) Presence of a suitable aquifer which can yield good quality water in sufficient quantity.

(*ii*) Availability of energy, preferably electric power, for pumps.

(*iii*) Well distributed demand for irrigation throughout the year.

(*iv*) Suitable configuration of command area with the highest ground around the centre of the command area.

In general, well irrigation is more efficient than canal irrigation. The following are the comparative features of the two types of irrigation:

(*i*) In the canal irrigation system, major structures, such as headworks, main and branch canals, etc. must be constructed prior to the start of proportionate agricultural activity which grows gradually because of the availability of irrigation facility. But, wells can be constructed gradually to keep pace with the development of the agricultural activities of the area.

(*ii*) Transit losses in well irrigation are much less than those in canal irrigation system.

(*iii*) Isolated patches of high lands can be better served by well irrigation.

(*iv*) Well irrigation offers an effective anti-waterlogging measure of the affected lands and reduces the chances of waterlogging of canal-irrigated lands.

(*v*) Well irrigation ensures relatively more reliable supply of water at the time of need. This results in better yield. Besides, farmers can switch over to more remunerative crops due to the availability of assured supply.

(*vi*) Well irrigation needs energy for pumping. Installation and maintenance of pumps and the cost of running the pumps make well irrigation costlier.

(*vii*) Failure of power supply at the time of keenest demand may adversely affect the yield in case of well irrigation systems.

It is thus obvious that both irrigation systems have advantages as well as disadvantages. Therefore, both must be used in a judicious manner to obtain maximum benefits, such that there is no waterlogging and the ground water resource can be maintained indefinitely.

4.3. OCCURRENCE OF GROUND WATER

The subsurface medium within which ground water occurs is either porous or fractured or both. The subsurface occurrence of ground water can be divided into two zones (Fig. 4.1): (*i*) the vadose zone or unsaturated zone or zone of aeration, and (*ii*) the phreatic zone or saturated zone or zone of saturation. In the saturated zone, all pores or voids are filled with water whereas in the unsaturated zone, pores contain gases (mainly air and water vapours) in addition to water.

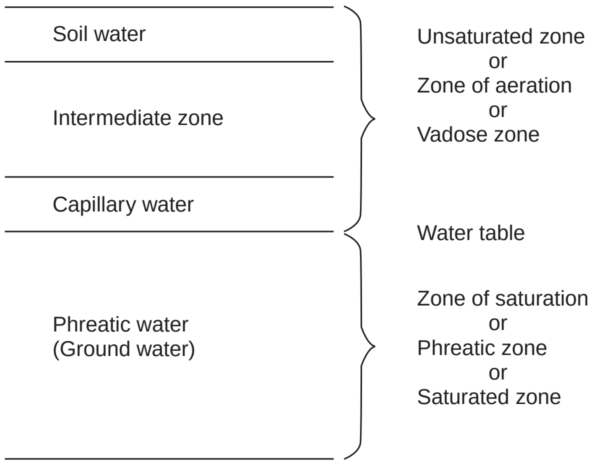

Fig. 4.1 Vertical distribution of subsurface water

The water table is defined as the upper limit of the saturated zone. However, it should be noted that all the pores near the base of the capillary water zone (which itself may range from practically nothing in coarse material to about 2.5 m or more in clay materials) may be completely saturated. The number of pores filled with water decreases in the upward direction of the capillary water zone. One can, therefore, expect the upper limit of actual saturation to be an irregular surface. *Water table* should, therefore, be redefined as the upper limit of saturation at atmospheric pressure.

The saturated zone containing interconnected pores may exceed depths penetrated by oil wells (more than 12,000 m). However, freshwater (part of the hydrologic cycle) is found only up to depths of about 800 m (2).

A saturated geologic formation capable of yielding water economically in sufficient quantity is known as an *aquifer* (or water-bearing formation or ground water reservoir). Ground water constantly moves through an aquifer under local hydraulic gradients. Thus, aquifers perform storage as well as conduit functions. Ground water may exist in aquifers in two different manners: (*i*) unconfined, and (*ii*) confined. The unconfined condition occurs when the water table is under atmospheric pressure and is free to rise or fall with changes in the volume of the stored water. An aquifer with unconfined conditions is referred to as an unconfined or water table aquifer. An aquifer which is separated from the unsaturated zone by an impermeable or very less permeable formation is known as confined aquifer (or artesian aquifer or pressure aquifer). Ground water in a confined aquifer is under pressure which is greater than the atmospheric pressure. The water level in a well penetrating a confined aquifer indicates the piezometric pressure at that point and will be above the bottom of the upper confining formation. Such wells are known as artesian wells and if the water level rises above the land surface, a *flowing well* results (Fig. 4.2).

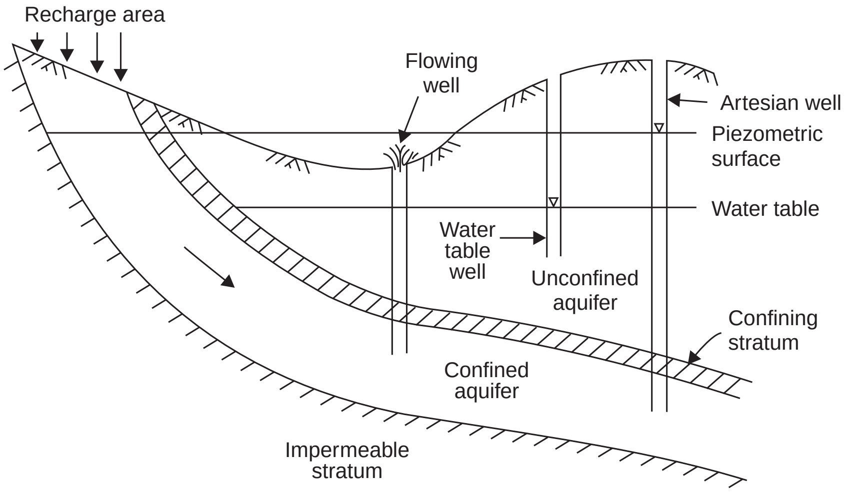

Fig. 4.2 Aquifers and wells

Water released from an unconfined aquifer is the result of dewatering or draining of the aquifer material. In the case of confined aquifer, the release of water is the result of a slight expansion of water and a very small compression of the porous medium (2).

The availability, movement, and quality of ground water depend mainly on the characteristics of the medium. The following characteristics of the medium affect the availability and movement of ground water.

Porosity can be defined as the ratio of the volume of pores to the total volume of the porous medium. It ranges from 0 to 50 per cent for most of the rock materials. For aquifer considerations, porosities less than 5% are considered small, those between 5% and 20% are considered medium and those greater than 20% are considered large (2). Porosity is, obviously, an inherent characteristic of the material independent of the presence or absence of water. For ground water studies, the interconnected pore space which can be drained by gravity should be used for determining the porosity and such porosity is known as effective porosity.

The *specific yield* of a soil formation is defined as the ratio of the volume of water which the soil formation, after being saturated, will yield by gravity to the volume of the soil formation.

The *specific retention* of a soil formation is defined as the ratio of the volume of water which the soil formation, after being saturated, will retain against the pull of gravity to the volume of the soil formation.

These definitions of specific yield and specific retention implicitly assume complete drainage. Obviously, the sum of the specific yield and the specific retention would be equal to the porosity of the given soil formation. The product of the average specific yield of a saturated water-bearing formation and its total volume gives the volume of water which can be recovered from the formation by gravity drainage. It may be noted that the time factor is not included in the definition of specific yield. However, the gravity draining of a formation decreases with time and may continue for years. Fine-grained materials may have lesser specific yield than coarse materials even though their porosity may be greater (Fig. 4.3 and Table 4.1).

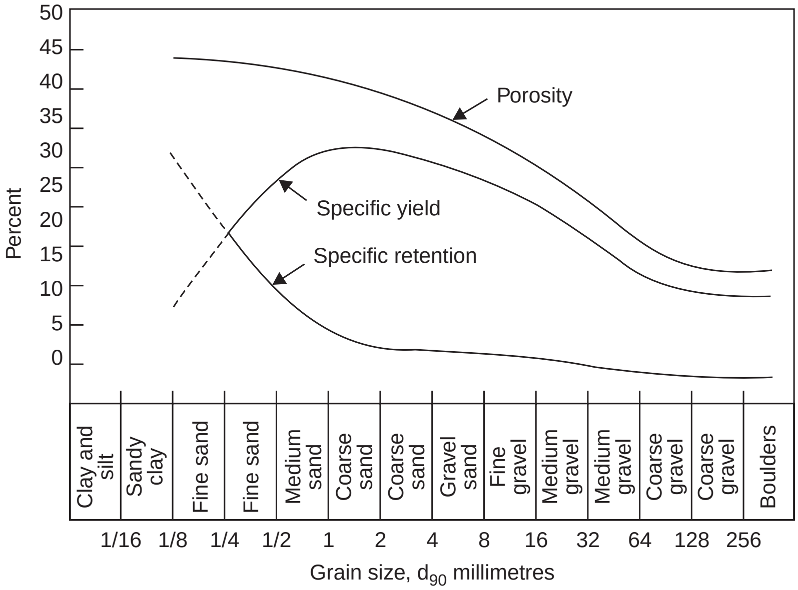

Fig. 4.3 Typical variation of porosity, specific yield, and specific retention with grain size (4)

Table 4.1 Representative porosity and specific yield of selected earth material

Material	*Porosity %*	*Specific yield %*
Clay	45 – 55	1 – 10
Sand	25 – 40	10 – 30
Gravel	25 – 40	15 – 30
Sand and gravel	10 – 35	15 – 25
Sandstone	5 – 30	5 – 15
Shale	0 – 10	0.5 – 5
Limestone	1 – 20	0.5 – 5

In case of confined aquifers there is no dewatering or draining of the material unless the hydraulic head drops below the top of the aquifer. Therefore, the concept of specific yield does not apply to confined aquifers and an alternative term, storage coefficient or storativity is used for confined aquifers. *Storativity* or *storage coefficient* is defined as the volume of water an aquifer would release from or take into storage per unit surface area of the aquifer for a unit change in head. Its value is of the order 5×10^{-2} to 1×10^{-5} (2). For the same drop in head, the yield from an unconfined aquifer is much greater than that from a confined aquifer.

The *permeability* of a porous medium describes the ease with which a fluid will pass through it. Therefore, it depends on the characteristics of the medium as well as the flowing fluid. It would be logical to use another term which reflects only the medium characteristics. This term is named intrinsic permeability, and is independent of the properties of the flowing fluid and depends only on the characteristics of the medium. It is proportional to the square of the representative grain diameter of the medium, and the constant of proportionality depends on porosity, packing, size distribution, and shape of grains.

The permeability of a medium is measured in terms of hydraulic conductivity (also known as the coefficient of permeability) which is equal to the volume of water which flows in unit time through a unit cross-sectional area of the medium under a unit hydraulic gradient at the prevailing temperature. The hydraulic conductivity, therefore, has the dimensions of $[L/T]$ and is usually expressed as metres per day or metres per hour. It should be noted that an unsaturated medium would have lower hydraulic conductivity because of the resistance to a flow of water offered by the air present in the void spaces.

The *transmissivity*, a term generally used for confined aquifers, is obtained by multiplying the hydraulic conductivity of an aquifer with the thickness of the saturated portion of the aquifer. It represents the amount of water which would flow through a unit width of the saturated portion of the aquifer under a unit hydraulic gradient and at the prevailing temperature.

Example 4.1 A ground water basin consists of 20 km^2 of plains. The maximum fluctuation of ground water table is 3 m. Assuming a specific yield of 15 per cent, determine the available ground water storage.

Solution: Ground water storage = area of basin × depth of fluctuation × specific yield

$$= 20 \times 10^6 \times 3 \times 0.15 = 9 \times 10^6 \text{ m}^3$$

Example 4.2 In an aquifer whose area is 100 ha, the water table has dropped by 3.0 m. Assuming porosity and specific retention of the aquifer material to be 30 per cent and 10 per cent, respectively, determine the specific yield of the aquifer and the change in ground water storage.

Solution: Porosity = specific yield + specific retention

∴ Specific yield = porosity – specific retention = 30 – 10 = 20%.

Reduction in ground water storage = $100 \times 10^4 \times 3.0 \times 0.2$

$= 60 \times 10^4 \text{ m}^3$.

4.4. FLOW OF WATER THROUGH POROUS MEDIA

Ground water flows whenever there exists a difference in head between two points. This flow can either be laminar or turbulent. Most often, ground water flows with such a small velocity that the resulting flow is laminar. Turbulent flow occurs when large volumes of water converge through constricted openings as in the vicinity of wells.

Based on a series of experiments conducted in the vertical pipe filled with sand, Henry Darcy, a French engineer, in 1856 concluded that the rate of flow, Q through a column of saturated sand is proportional to the difference in hydraulic head, Δh, between the ends of the column and to the area of flow cross-section A, and inversely proportional to the length of the column, L. Thus,

$$Q = KA \frac{\Delta h}{L} \tag{4.1}$$

Here, K is the constant of proportionality and is equal to the hydraulic conductivity of the medium. Equation (4.1) is known as Darcy's law and can also be written as

$$V = K \frac{\Delta h}{L} \tag{4.2}$$

in which, V is the specific discharge (or the apparent velocity of flow) and $\frac{\Delta h}{L}$ is the hydraulic gradient. Expressed in general terms, Darcy's law, Eq. (4.2), becomes

$$V = -K \frac{dh}{ds} \tag{4.3}$$

in which, dh/ds is the hydraulic gradient which is negative, since h decreases in the positive direction of the flow. Thus, flow along the three principal co-ordinate axes can be described as

$$u = -K_x \frac{\partial h}{\partial x} \tag{4.4a}$$

$$v = -K_y \frac{\partial h}{\partial y} \tag{4.4b}$$

and

$$w = -K_z \frac{\partial h}{\partial z} \tag{4.4c}$$

Here, u, v, and w are the velocity components in the x-, y-, and z-directions, respectively, and K_x, K_y, and K_z are hydraulic conductivities (coefficients of permeability) in these directions.

In Darcy's law, the velocity is proportional to the first power of the hydraulic gradient and is, therefore, applicable to laminar flows only. For a flow through porous medium, Reynolds number R_e can be expressed as

$$R_e = \frac{Vd\rho}{\mu}$$

Here, d is the representative average grain diameter which approximately represents the average pore diameter, *i.e.,* the flow dimension. ρ and μ are, respectively, the mass density and the dynamic viscosity of the flowing water. An upper limit of Reynolds number ranging between 1 and 10 has been suggested as the limit of validity of Darcy's law (4). A range rather than a unique value of R_e has been specified in view of the possible variety of grain shapes, grain-size distribution, and their packing conditions. For natural ground water motion, R_e is usually less than unity and Darcy's law is, therefore, usually applicable.

When Darcy's law is substituted in the continuity equation of motion, one obtains the equation governing the flow of water through a porous medium. The resulting equations for confined and unconfined aquifers are, respectively, as follows (5):

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2} = \frac{S}{T}\frac{\partial h}{\partial t} \tag{4.5}$$

and

$$\frac{\partial^2 H^2}{\partial x^2} + \frac{\partial^2 H^2}{\partial y^2} = \frac{2n}{K}\frac{\partial H}{\partial t} \tag{4.6}$$

Here, H represents the hydraulic head in unconfined aquifer and n is the porosity of the medium. Equations (4.5) and (4.6) are, respectively, known as Boussinesq's and Dupuit's equations. Both these equations assume that the medium is homogeneous, isotropic, and water is incompressible. Equation (4.5) also assumes that large pressure variations do not occur. Equation (4.6) further assumes that the curvature of the free surface is sufficiently small for the vertical components of the flow velocity to be negligible in comparison to the horizontal component. For steady flow, Eqs. (4.5) and (4.6) become

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2} = 0 \tag{4.7a}$$

$$\frac{\partial^2 H^2}{\partial x^2} + \frac{\partial^2 H^2}{\partial y^2} = 0 \tag{4.7b}$$

4.5. WELL HYDRAULICS

A well is a hydraulic structure which, if properly designed and constructed, permits economic withdrawal of water from an aquifer (6). When water is pumped from a well, the water table (or the piezometric surface in case of a confined aquifer) is lowered around the well. The surface of a lowered water table resembles a cone and is, therefore, called the *cone of depression.* The horizontal distance from the centre of a well to the practical limit of the cone of depression is known as the *radius of influence* of the well. It is larger for wells in confined aquifers than for those in unconfined aquifers. All other variables remaining the same, the radius of influence is larger in aquifers with higher transmissivity than in those with lower transmissivity. The difference, measured in the vertical direction, between the initial water table (or the piezometric surface in the confined aquifer) and its lowered level due to pumping at any location within the radius of influence is called the *drawdown* at that location. *Well yield* is defined as the volume of water discharge, either by pumping or by free flow, per unit time. Well yield per unit drawdown in the well is known as the *specific capacity* of the well.

With the continued pumping of a well, the cone of depression continues to expand in an extensive aquifer until the pumping rate is balanced by the recharge rate. When pumping and recharging rates balance each other, a steady or equilibrium condition exists and there is no further drawdown with continued pumping. In some wells, the equilibrium condition may be attained within a few hours of pumping, while in others it may not occur even after prolonged pumping.

4.5.1. Equilibrium Equations

For confined aquifers, the governing equation of flow, Eq. (4.7*a*), can be written in polar cylindrical coordinates (r, θ, z) as

$$\frac{1}{r}\left(\frac{\partial}{\partial r} r \frac{\partial h}{\partial r}\right) + \frac{\partial^2 h}{r^2 \partial \theta^2} + \frac{\partial^2 h}{\partial z^2} = 0 \tag{4.8}$$

If one assumes radial symmetry (*i.e.,* h is independent of θ) and the aquifer to be horizontal and of constant thickness (*i.e.,* h is independent of z), Eq. (4.8) reduces to

$$\frac{d}{dr}\left(r \frac{dh}{dr}\right) = 0 \tag{4.9}$$

For flow towards a well, penetrating the entire thickness of a horizontal confined aquifer, Eq. (4.9) needs to be solved for the following boundary conditions (Fig. 4.4):

(*i*) at $r = r_0$, $h = h_0$ (r_0 is the radius of influence)

(*ii*) at $r = r_w$, $h = h_w$ (r_w is the radius of well)

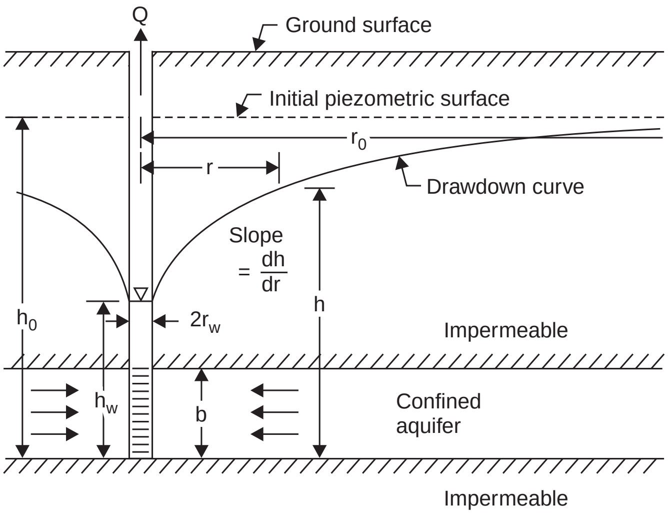

Fig. 4.4 Radial flow to a well penetrating an extensive confined aquifer

On integrating Eq. (4.9) twice with respect to r, one obtains

$$r \frac{dh}{dr} = C_1$$

and $$h = C_1 \ln r + C_2 \tag{4.10}$$

in which C_1 and C_2 are constants of intergration to be obtained by substituting the boundary conditions in Eq. (4.10) which yields

$$h_0 = C_1 \ln r_0 + C_2$$

and $$h_w = C_1 \ln r_w + C_2$$

Hence, $$C_1 = \frac{h_0 - h_w}{\ln (r_0/r_w)}$$

and $$C_2 = h_0 - \frac{h_0 - h_w}{\ln (r_0/r_w)} \ln r_0$$

Also, $$C_2 = h_w - \frac{h_0 - h_w}{\ln (r_0/r_w)} \ln r_w$$

Finally, $$h = h_0 - \frac{h_0 - h_w}{\ln (r_0/r_w)} \ln (r_0/r) \quad (4.11)$$

and also, $$h = h_w + \frac{h_0 - h_w}{\ln (r_0/r_w)} \ln (r/r_w) \quad (4.12)$$

Further, the discharge Q through any cylinder of radius r and height equal to the thickness of the aquifer B is expressed as

$$Q = -K(2\pi r B)\frac{dh}{dr}$$
$$= -2\pi T\left(r\frac{dh}{dr}\right)$$
$$= -2\pi T C_1$$

$\therefore$ $$Q = -2\pi T \frac{h_0 - h_w}{\ln (r_0/r_w)} \quad (4.13)$$

Thus, Eqs. (4.11) and (4.12) can be rewritten as

$$h = h_0 + \frac{Q}{2\pi T} \ln (r_0/r) \quad (4.14)$$

and $$h = h_w - \frac{Q}{2\pi T} \ln (r/r_w) \quad (4.15)$$

It should be noted that the coordinate r is measured positive away from the well and that the discharge towards the well is in the negative direction of r. Therefore, for a discharging well, Q is substituted as a negative quantity in Eqs. (4.13) through (4.15). If the drawdown at any radial distance r from the well is represented by s, then

$$s = h_0 - h = -\frac{Q}{2\pi T} \ln r_0/r \quad (4.16)$$

and the well drawdown s_w is given as

$$s_w = h_0 - h_w = -\frac{Q}{2\pi T} \ln r_0/r_w \quad (4.17)$$

For unconfined aquifers, one can similarly obtain the following equations starting from the Dupuit's equation, Eq. (4.7*b*):

$$H^2 = H_0^2 - \frac{H_0^2 - H_w^2}{\ln (r_0/r_w)} \ln r_0/r \quad (4.18)$$

$$H^2 = H_w^2 + \frac{H_0^2 - H_w^2}{\ln (r_0/r_w)} \ln (r/r_w) \quad (4.19)$$

$$Q = -\pi K \frac{H_0^{\,2} - H_w^{\,2}}{\ln (r_0/r_w)} \tag{4.20}$$

$$H^2 = H_0^{\,2} + \frac{Q}{\pi K} \ln (r_0/r) \tag{4.21}$$

$$H^2 = H_w^{\,2} - \frac{Q}{\pi K} \ln (r/r_w) \tag{4.22}$$

Example 4.3 A well with a radius of 0.3 m, including gravel envelope and developed zone, completely penetrates an unconfined aquifer with $K = 25$ m/day and initial water table at 30 m above the bottom of the aquifer. The well is pumped so that the water level in the well remains at 22 m above the bottom of the aquifer. Assuming that pumping has essentially no effect on water table height at 300 m from the well, determine the steady-state well discharge. Neglect well losses.

Solution: From Eq. (4.20),

$$Q = -\pi K \frac{H_0^{\,2} - H_w^{\,2}}{\ln (r_0/r_w)}$$

$$= -\frac{3.14 \times 25 \times (30^2 - 22^2)}{\ln\left(\dfrac{300}{0.3}\right)}$$

$$= -\,4729.84 \text{ m}^3/\text{day}.$$

Negative sign indicates pumping well.

4.5.2. Non-Equilibrium Equations

For an unsteady flow in confined aquifer, Eq. (4.5) can be written in polar cylindrical coordinates (r, θ, z) as

$$\frac{1}{r}\left(\frac{\partial}{\partial r} r \frac{\partial h}{\partial r}\right) + \frac{\partial^2 h}{r^2 \partial \theta^2} + \frac{\partial^2 h}{\partial z^2} = \frac{S}{T}\frac{\partial h}{\partial t} \tag{4.23}$$

which reduces to

$$\frac{\partial^2 h}{\partial r^2} + \frac{1}{r}\frac{\partial h}{\partial r} = \frac{S}{T}\frac{\partial h}{\partial t} \tag{4.24}$$

if one assumes radial symmetry and the aquifer to be horizontal and of constant thickness. The general form of solution of Eq. (4.24) is $h(r, t)$. For unsteady flow towards a well penetrating the entire thickness of a confined aquifer, Eq. (4.24) needs to be solved for the following boundary conditions:

(*i*) $h(\infty, t) = h_0$

(*ii*) $2\pi r_w B \left(K \dfrac{\partial h}{\partial r}\right)_{r=r_w} = -Q$ for $t > 0$ (flux condition)

$$\therefore \quad \left(r_w \frac{\partial h}{\partial r}\right)_{r=r_w} = -\frac{Q}{2\pi T}$$

which can be approximated as

$$\left(r\frac{\partial h}{\partial r}\right)_{r\to 0} = -\frac{Q}{2\,\pi T}$$

(*iii*) $h(r, 0) = h_0$ (initial condition).

Theis (7) obtained a solution of Eq. (4.24) by assuming that the well is replaced by a mathematical sink of constant strength. The solution is expressed as

$$s = h_0 - h = -\frac{Q}{4\,\pi T}\int_u^{\infty}\frac{e^{-u}}{u}\,du \tag{4.25}$$

in which $u = \dfrac{r^2 S}{4Tt}$

where, t is the time since the beginning of pumping. Equation (4.25) is also written as

$$s = -\frac{Q}{4\pi T}\,W(u) \tag{4.26}$$

in which, $W(u)$ is known as the well function (Table 4.2) and is expressed as a function of u in the form of the following convergent series:

$$W(u) = -\,0.5772 - \ln u + u - \frac{u^2}{2\times 2\,!} + \frac{u^3}{3\times 3\,!} - \frac{u^4}{4\times 4\,!} + \ldots \tag{4.27}$$

An approximate form of the Theis equation (*i.e.,* Eq. (4.25)) was obtained by Cooper and Jacob (8) dropping the third and higher order terms of the series of Eq. (4.27). Thus,

$$s = -\frac{Q}{4\,\pi T}\,[-\,0.5772 - \ln u]$$

or

$$s = -\frac{Q}{4\,\pi T}\left[\ln\frac{0.25\,Tt}{r^2 S}\right]$$

$\therefore$

$$s = -\frac{0.183\,Q}{T}\left[\log\frac{2.25\,Tt}{r^2 S}\right] \tag{4.28}$$

For values of u less than 0.05, Eq. (4.28) gives practically the same results as obtained by Eq. (4.26). Note that Q is to be substituted as a negative quantity for a pumping well.

Because of the non-linear form of Eq. (4.6), its solution is difficult. Boulton (9) has presented a solution for fully penetrating wells in an unconfined aquifer. The solution is valid if the water depth in the well exceeds 0.5 H_0. The solution is

$$s = \frac{Q}{2\,\pi K H_0}\,(1 + C_k)\,V\,(t', r') \tag{4.29}$$

in which, C_k is a correction factor which can be taken as zero for t' less than 5, and according to Table 4.3 for t' greater than 5 (when C_k depends only on r'). $V\,(t', r')$ is Boulton's well function dependent on r' and t' defined as

$$t' = \frac{Kt}{SH_0}$$

and

$$r' = \frac{r}{H_0}$$

Table 4.2 Well function $W(u)$ for different values of u

u	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0
$\times 1$	0.219	0.049	0.013	0.0038	0.0011	0.00036	0.00012	0.000038	0.000012
$\times 10^{-1}$	1.82	1.22	0.91	0.70	0.56	0.45	0.37	0.31	0.26
$\times 10^{-2}$	4.04	3.35	2.96	2.68	2.47	2.30	2.15	2.03	1.92
$\times 10^{-3}$	6.33	5.64	5.23	4.95	4.73	4.54	4.39	4.26	4.14
$\times 10^{-4}$	8.63	7.94	7.53	7.25	7.02	6.84	6.69	6.55	6.44
$\times 10^{-5}$	10.94	10.24	9.84	9.55	9.33	9.14	8.99	8.86	8.74
$\times 10^{-6}$	13.24	12.55	12.14	11.85	11.63	11.45	11.29	11.16	11.04
$\times 10^{-7}$	15.54	14.85	14.44	14.15	13.93	13.75	13.60	13.46	13.34
$\times 10^{-8}$	17.84	17.15	16.74	16.46	16.23	16.05	15.90	15.76	15.65
$\times 10^{-9}$	20.15	19.45	19.05	18.76	18.54	18.35	18.20	18.07	17.95
$\times 10^{-10}$	22.45	21.76	21.35	21.06	20.84	20.66	20.50	20.37	20.25
$\times 10^{-11}$	24.75	24.06	23.65	23.36	23.14	22.96	22.81	22.67	22.55
$\times 10^{-12}$	27.05	26.36	25.96	25.67	25.44	25.26	25.11	24.97	24.86
$\times 10^{-13}$	29.36	28.66	28.26	27.97	27.75	27.56	27.41	27.28	27.16
$\times 10^{-14}$	31.66	30.97	30.56	30.27	30.05	29.87	29.71	29.58	29.46
$\times 10^{-15}$	33.96	33.27	32.86	32.58	32.35	32.17	32.02	31.88	31.76

Table 4.3 Values of C_k for $t' > 5$

r'	0.03	0.04	0.06	0.08	0.1	0.2	0.4	0.6	0.8	1	2	4
C	– 0.27	– 0.24	– 0.19	– 0.16	– 0.13	– 0.05	– 0.02	0.05	0.05	0.05	0.03	0

Values of the function V have been tabulated in Table 4.4. The approximate values of V can also be calculated as follows:

For $t' < 0.01$ and $t'/r' > 10$.

$$V \approx \ln (2t'/r')$$

For $t' < 0.01$,

$$V \approx \sin h^{-1} (t'/r') - \frac{1}{\sqrt{1+r'^2}}$$

For $t' < 0.05$,

$$V \approx \sin h^{-1} \left[\frac{1}{r'}\right] + \sin h^{-1} [t'/r'] - \sin h^{-1} \left[\frac{1+t'}{r'}\right]$$

For $t' > 5.0$,

$$V \approx \frac{1}{2} W(u)$$

in which,

$$u = \frac{r^2 n_e}{4Tt}$$

Here, n_e is the effective porosity of the aquifer.

The well equations mentioned in this section are valid only for a single well of small diameter having no storage capability and fully penetrating an extensive aquifer. The equations will be modified for the effect of partial penetration, well storage, bounded aquifers, interference of adjacent wells, and multi-layer aquifer systems.

Example 4.4 A fully penetrating artesian well is pumped at a rate Q = 1500 m^3/day from an aquifer whose storage coefficient and transmissivity are 4×10^{-4} and 0.145 m^2/min, respectively. Find the drawdowns at a distance 3 m from the production well after one hour of pumping and at a distance of 350 m after one day of pumping.

Solution:

At r = 3 m and t = 1 h,

$$u = \frac{r^2 s}{4\,Tt} = \frac{3 \times 3 \times 4 \times 10^{-4}}{4 \times 0.145 \times (1 \times 60)} = 1.03 \times 10^{-4}$$

$$\therefore \quad w(u) = 8.62$$

From Eq. (4.26)

$$\therefore \quad s = -\frac{Q}{4\,\pi T} W(u) = \frac{1500/(24 \times 60)}{4 \times 3.14 \times 0.145} \times 8.62$$

$$= 4.93 \text{ m}$$

Similarly, at r = 350 m and t = 1 day

Table 4.4 Function $V(t', r')$ for different values of t' and r'

t'	Value of r'																	
	0.001	0.002	0.003	0.004	0.005	0.006	0.007	0.008	0.009	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.01	2.99	2.30	1.90	1.64	1.42	1.28	1.15	1.04	0.95	0.875	0.474	0.322	0.240	0.192	0.158	0.135	0.118	0.104
0.02	3.68	2.97	2.58	2.30	2.09	1.92	1.76	1.64	1.52	1.42	0.860	0.610	0.468	0.378	0.316	0.270	0.236	0.210
0.03	4.08	3.40	3.00	2.70	2.46	2.28	2.13	2.00	1.88	1.79	1.18	0.860	0.675	0.555	0.465	0.400	0.350	0.310
0.04	4.35	3.68	3.26	2.98	2.75	2.58	2.42	2.29	2.17	2.06	1.42	1.07	0.850	0.710	0.600	0.525	0.460	0.410
0.05	4.58	3.90	3.49	3.20	2.96	2.79	2.64	2.50	2.38	2.28	1.60	1.24	1.010	0.850	0.725	0.630	0.560	0.500
0.06	4.76	4.06	3.65	3.36	3.15	2.96	2.80	2.68	2.56	2.45	1.78	1.40	1.15	0.970	0.840	0.735	0.650	0.585
0.07	4.92	4.20	3.80	3.51	3.30	3.12	2.96	2.82	2.70	2.60	1.91	1.54	1.28	1.09	0.950	0.835	0.740	0.670
0.08	5.08	4.34	3.94	3.65	3.42	3.24	3.09	2.95	2.84	2.72	2.04	1.65	1.39	1.20	1.04	0.925	0.825	0.750
0.09	5.18	4.47	4.05	3.75	3.54	3.35	3.20	3.05	2.95	2.84	2.14	1.75	1.50	1.29	1.14	1.02	0.910	0.825
0.1	5.24	4.54	4.14	3.85	3.63	3.45	3.30	3.15	3.04	2.94	2.25	1.85	1.58	1.38	1.22	1.09	0.985	0.890
0.2	5.85	5.15	4.78	4.50	4.28	4.10	3.93	3.80	3.66	3.56	2.87	2.46	2.20	1.98	1.80	1.65	1.52	1.42
0.3	6.24	5.50	5.12	4.85	4.61	4.43	4.28	4.14	4.01	3.90	3.24	2.84	2.54	2.32	2.14	1.98	1.85	1.74
0.4	6.45	5.75	5.35	5.08	4.85	4.67	4.50	4.38	4.26	4.15	3.46	3.05	2.76	2.54	2.36	2.20	2.07	1.96
0.5	6.65	6.00	5.58	5.25	5.00	4.85	4.70	4.55	4.45	4.30	3.65	3.24	2.95	2.72	2.52	2.38	2.24	2.14
0.6	6.75	6.10	5.65	5.40	5.15	4.98	4.82	4.68	4.56	4.45	3.76	3.37	3.09	2.85	2.67	2.50	2.38	2.26
0.7	6.88	6.20	5.80	5.50	5.25	5.08	4.92	4.80	4.68	4.55	3.90	3.50	3.20	2.99	2.80	2.64	2.50	2.38
0.8	7.00	6.25	5.85	5.60	5.35	5.20	5.00	4.90	4.80	4.65	3.96	3.55	3.26	3.05	2.86	2.71	2.58	2.46
0.9	7.10	6.35	6.00	5.70	5.50	5.30	5.12	5.00	4.90	4.75	4.05	3.65	3.36	3.15	2.96	2.80	2.66	2.55
1	7.14	6.45	6.05	5.75	5.55	5.35	5.20	5.05	4.95	4.83	4.10	3.74	3.45	3.22	3.04	2.90	2.75	2.64
2	7.60	6.88	6.45	6.15	5.92	5.75	5.60	5.50	5.35	5.25	4.59	4.18	3.90	3.68	3.50	3.34	3.20	3.09
3	7.85	7.15	6.70	6.45	6.20	6.00	5.85	5.75	5.60	5.50	4.82	4.42	4.12	3.90	3.72	3.57	3.45	3.31
4	8.00	7.28	6.85	6.58	6.35	6.15	6.00	5.90	5.75	5.70	4.95	4.55	4.26	4.04	3.86	3.70	3.59	3.46
5	8.15	7.35	7.00	6.65	6.50	6.25	6.10	6.00	5.85	5.80	5.05	4.68	4.40	4.19	4.00	3.85	3.71	3.60
6	8.20	7.50	7.10	6.75	6.55	6.35	6.20	6.10	5.95	5.85	5.20	4.78	4.50	4.26	4.09	3.92	3.80	3.69
7	8.25	7.55	7.15	6.85	6.62	6.40	6.30	6.20	6.05	5.95	5.25	4.85	4.58	4.35	4.18	4.00	3.90	3.78
8	8.30	7.60	7.20	6.90	6.70	6.50	6.35	6.25	6.10	6.05	5.30	4.92	4.65	4.40	4.25	4.10	3.95	3.82
9	8.32	7.65	7.25	7.00	6.75	6.55	6.40	6.30	6.15	6.10	5.35	5.00	4.70	4.49	4.30	4.15	4.00	3.90
10	8.35	7.75	7.35	7.05	6.80	6.60	6.45	6.35	6.20	6.14	5.40	5.02	4.80	4.52	4.35	4.19	4.05	3.92

(Contd.)...

Table 4.4 (Contd..)

t'	Values of r'													
	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1	2	3	4	5
0.01	0.093	0.0430	0.0264	0.0180	0.0132	0.0100	0.0078	0.0062	0.0049	0.0040	0.00057	0.00015		
0.02	0.187	0.0865	0.0530	0.0365	0.0268	0.0205	0.0160	0.0125	0.0100	0.0081	0.00118	0.00020		
0.03	0.278	0.130	0.0800	0.0550	0.0405	0.0310	0.0240	0.0190	0.0150	0.0122	0.00184	0.00032		
0.04	0.368	0.174	0.107	0.0735	0.0540	0.0415	0.0322	0.0255	0.0202	0.0165	0.00244	0.00043		
0.05	0.450	0.215	0.133	0.0920	0.0675	0.0520	0.0400	0.0320	0.0255	0.0206	0.00305	0.00055		
0.06	0.530	0.257	0.160	0.110	0.0810	0.0610	0.0478	0.0380	0.0305	0.0250	0.00365	0.00065		
0.07	0.610	0.298	0.186	0.130	0.0950	0.0725	0.0565	0.0450	0.0360	0.0292	0.00430	0.00078		
0.08	0.680	0.340	0.214	0.148	0.108	0.0825	0.0645	0.0510	0.0412	0.0336	0.00500	0.00090		
0.09	0.750	0.378	0.236	0.164	0.122	0.0930	0.0730	0.0585	0.0470	0.0380	0.00570	0.00105		
0.1	0.815	0.415	0.260	0.180	0.134	0.103	0.0805	0.0640	0.0515	0.0420	0.00635	0.00118		
0.2	1.32	0.750	0.500	0.359	0.268	0.208	0.165	0.132	0.107	0.0880	0.0145	0.00278		
0.3	1.64	1.02	0.700	0.515	0.392	0.308	0.246	0.200	0.164	0.135	0.0238	0.00490		
0.4	1.86	1.22	0.870	0.650	0.510	0.405	0.328	0.268	0.220	0.182	0.0350	0.00750	0.00160	0.00038
0.5	2.03	1.37	1.00	0.770	0.610	0.490	0.400	0.330	0.275	0.230	0.0450	0.0104	0.00240	0.00056
0.6	2.16	1.49	1.12	0.875	0.700	0.570	0.468	0.390	0.325	0.276	0.0580	0.0138	0.00320	0.00080
0.7	2.28	1.60	1.22	0.965	0.775	0.640	0.525	0.445	0.375	0.320	0.0715	0.0175	0.00425	0.00108
0.8	2.36	1.69	1.30	1.04	0.850	0.715	0.600	0.500	0.425	0.364	0.0840	0.0212	0.00525	0.00140
0.9	2.45	1.75	1.38	1.11	0.920	0.775	0.650	0.550	0.475	0.404	0.0980	0.0260	0.00630	0.00165
1	2.54	1.85	1.45	1.18	0.975	0.825	0.700	0.595	0.510	0.444	0.113	0.0310	0.00840	0.00235
2	2.97	2.29	1.88	1.60	1.38	1.22	1.07	0.950	0.840	0.750	0.259	0.0950	0.0330	0.0115
3	3.20	2.50	2.10	1.82	1.60	1.42	1.28	1.15	1.05	0.960	0.388	0.165	0.0700	0.0275
4	3.36	2.66	2.25	1.97	1.75	1.58	1.42	1.30	1.20	1.10	0.495	0.235	0.112	0.0535
5	3.49	2.78	2.38	2.09	1.87	1.69	1.54	1.42	1.30	1.21	0.580	0.300	0.150	0.0715
6	3.59	2.90	2.47	2.18	1.95	1.78	1.65	1.52	1.40	1.30	0.660	0.360	0.195	0.0990
7	3.66	2.96	2.55	2.25	2.04	1.85	1.70	1.58	1.48	1.38	0.730	0.415	0.230	0.125
8	3.74	3.00	2.60	2.32	2.11	1.94	1.79	1.66	1.55	1.44	0.790	0.465	0.272	0.155
9	3.80	3.09	2.67	2.39	2.17	2.00	1.85	1.72	1.60	1.50	0.850	0.515	0.307	0.182
10	3.84	3.12	2.74	2.45	2.24	2.05	1.90	1.77	1.65	1.55	0.890	0.550	0.340	0.210

$$u = \frac{(350)^2 \times 4 \times 10^{-4}}{4 \times 0.145 \times 24 \times 60} = 5.87 \times 10^{-2}$$

$\therefore$ $$w(u) = 2.316$$

and $$s = \frac{1500/(24 \times 60)}{4\,\pi \times 0.145} \times 2.316$$

$$= 1.32 \text{ m.}$$

4.5.3. Well Interference

If the zone of influence of two adjacent wells overlap (*i.e.,* the wells are spaced at distances smaller than the sum of their radii of influence), the wells affect each other's drawdown and discharge. This effect is due to what is known as well interference. As a result of well interference, even though the total output (*i.e.,* the discharge) of a multiple well system increases, the efficiency of each well (measured in terms of the discharge per unit drawdown) of the system decreases. Since the equation of flow in a confined aquifer is a linear one, one can use the principle of superposition to obtain the resulting drawdown at a point in a well field in which number of wells are being pumped simultaneously. This means, if s_i is the total drawdown at i^{th} observation well on acount of pumping of N wells located in the well field, then

$$s_i = \sum_{j=1}^{N} s_{ij} \tag{4.30}$$

in which, s_{ij} is the drawdown at i^{th} observation well on account of pumping of j^{th} well as if there were no interference effects.

Considering steady flow conditions for two wells in a confined aquifer located distance B apart, the drawdown in the two wells s_{w1} and s_{w2} can be expressed as

$$s_{w1} = \frac{Q_1}{2\,\pi T} \ln \frac{r_0}{r_w} + \frac{Q_2}{2\,\pi T} \ln \frac{r_0}{B}, \tag{4.31}$$

$$s_{w2} = \frac{Q_1}{2\,\pi T} \ln \frac{r_0}{B} + \frac{Q_2}{2\,\pi T} \ln \frac{r_0}{r_w} \tag{4.32}$$

If $Q_1 = Q_2 = Q$, then

$$s_{w1} = s_{w2} = h_0 - h_w = \frac{Q}{2\,\pi T} \ln \frac{r_0^{\,2}}{Br_w}$$

This means,

$$\frac{Q}{h_0 - h_w} = \frac{2\,\pi T}{\ln \dfrac{r_0^{\,2}}{Br_w}} \tag{4.33}$$

Since,

$$\ln \frac{r_0^{\,2}}{Br_w} > \ln \frac{r_0}{r_w},$$

it is obvious that the efficiency of an individual well has reduced. On the other hand, Eq. (4.33) can also be written as

$$\frac{2\,Q}{h_0 - h_w} = \frac{2\,\pi T}{\ln\,[r_0/\,(Br_w)^{0.5}]} \tag{4.34}$$

Since $(Br_w)^{0.5} >> r_w$, the value of $(h_0 - h_w)$ is relatively less for a discharge of $2\,Q$ compared to the value of $(h_0 - h_w)$ when only a single well were to pump a discharge of $2\,Q$. This shows that the efficiency of a multiple well system is higher compared to that of a single well. But, the efficiency of an individual well in a multiple well system is reduced.

4.5.4. Wells Near Aquifer Boundaries

The equations for radial flow towards well assume infinite extent of aquifer. However, in practice, there would be situations when a well may be located near hydrogeologic boundaries and the derived equations would not be applicable as such. The influence of such boundaries on ground water movement can be determined by the image well method.

The image well method assumes straight line boundaries and replaces the real bounded field of flow with a fictitious field of flow with simple boundary conditions such that the flow patterns in the two cases are the same. Consider a pumping well located in the vicinity of a stream (*i.e.,* recharge or permeable boundary). Obviously, the drawdown at the stream on account of pumping well would be zero. This real flow system is now assumed to be replaced with a fictitious flow system, Fig. 4.5. In addition to the real pumping well, the fictitious flow system has, in place of the boundary, an image well (which is a recharging one *i.e.,* the one which pumps water into the aquifer) with the same capacity as that of the real well but located across the real boundary on a perpendicular thereto and at the same distance as the real well from the boundary. Obviously, this fictitious system would result in zero drawdown at the location of the boundary. This means that the flow condition of the real flow system is satisfied by the flow condition of the fictitious flow system. If the boundary is a barrier (*i.e.,* impermeable) boundary, the method remains the same but the image well is also a pumping well. It should be noted that in the fictitious system the real and image wells operate simultaneously and the drawdowns can be obtained by considering the fictitious system as a multiple well system. When an aquifer is delimited by two or more boundaries, the effect of the other boundaries on each of the image wells is also to be considered. As a result, there would be several images, Fig. 4.6. When the image wells are too far from the region of interest, their influence on the flow system in the region of interest is negligible and are, therefore, not included in the computations.

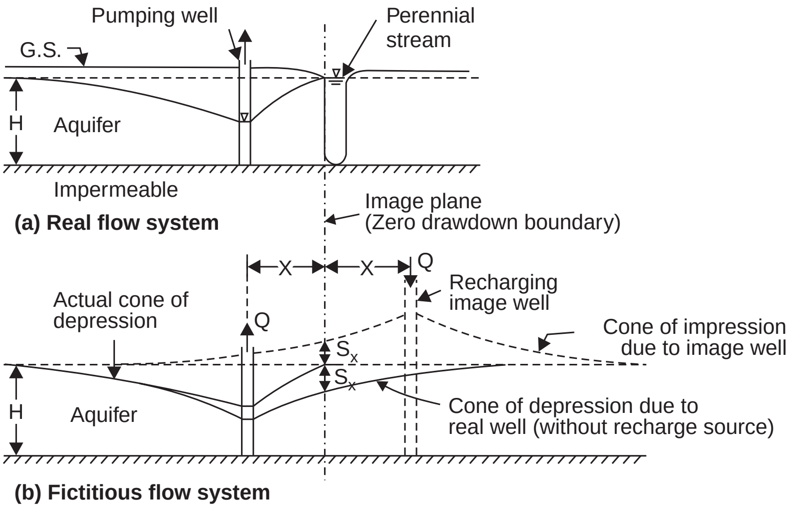

Fig. 4.5 Simulation of recharge boundary

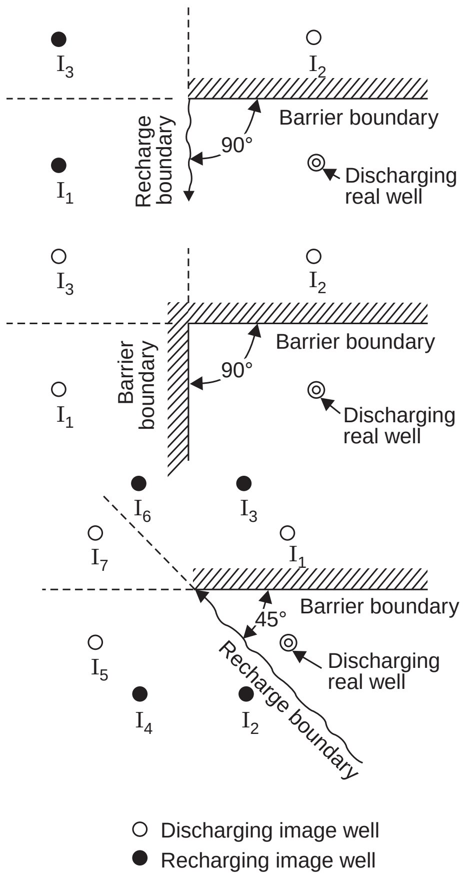

Fig. 4.6 Image well system for different pairs of boundaries

4.6. GROUND WATER EXPLORATION

It is known that everywhere on the earth there is some water under the surface. Ground water planners, however, need to know whether the conditions of the available ground water would permit its economic withdrawal through wells. The purpose of ground water exploration is to delineate the water-bearing formations, estimate their hydrogeologic characteristics and determine the quality of water present in these formations. Some of the exploration methods are briefly discussed in the following paragraphs.

4.6.1. Remote Sensing

Aerial photography, imaging (infra-red and radar) and low frequency electromagnetic aerial methods are included in the "remote sensing" methods of ground water exploration.

Valuable information associated with precipitation, evapotranspiration, interception, infiltration, and runoff can be inferred from aerial photographs by mapping the water area, geology and soil types, seepage areas, vegetation cover, and many other features (10). Satellite photographs can also be used for this purpose.

Recent developments in the nonvisible portion of the electromagnetic spectrum have resulted in several imaging techniques which are capable of mapping earth resources. Infrared imagery is sensitive to the differential head capacity of the ground and can map soil moisture, ground water movement, and faults (11). Radar imagery works in the 0.01–3 m wavelength range and can penetrate vegetation cover to provide subsurface information, such as soil moisture at shallow depths (12).

Buried subsurface channels and salt water intrusion fronts can be successfully located by using recently developed aerial electromagnetic exploration methods which operate in the frequency range of 3.0 to 9 kHz (13).

4.6.2. Surface Geophysical Methods

Surface geophysical methods reveal specific details of the physical characteristics of the local subsurface environment. This information can be interpreted suitably for the purpose of delineating the pre-glacial drainage pattern, mapping the location and extent of buried permeable deposits, direct exploration for ground water, and mapping of freshwater and salt water contact (14). The electrical resistivity method and seismic refraction method are the surface geophysical methods commonly used for ground water exploration.

(*i*) ***Electrical Resistivity Method***

The electrical resistivity of a rock depends on porosity, salinity of the fluid in the pore spaces, straightness or tortuosity of the interconnected pore spaces, presence of solid conductors,

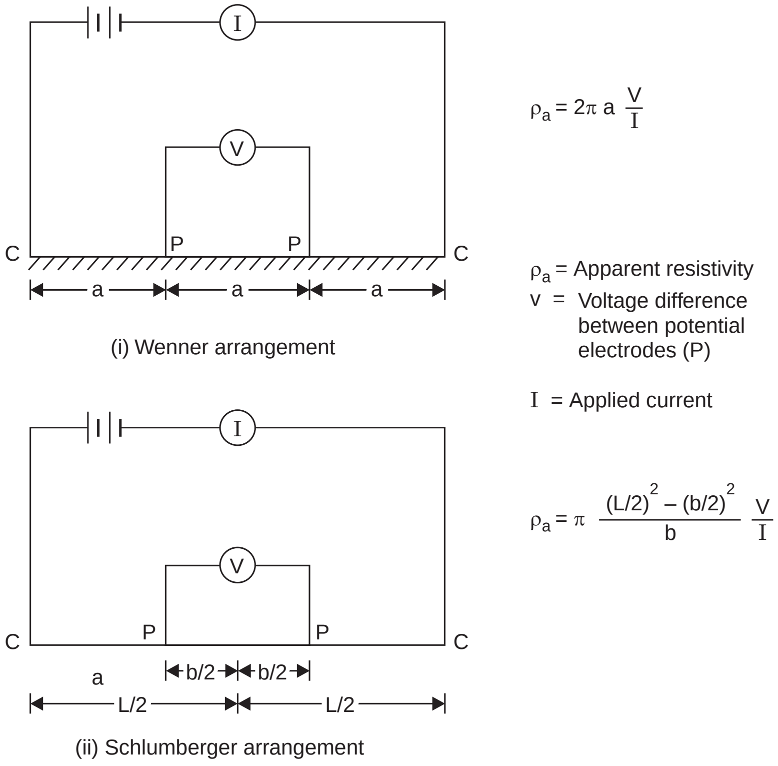

Fig. 4.7 Electrode arrays for electrical resistivity method

such as clays or metallic minerals, and temperature (2). In the electrical resistivity method, electrical current is injected into the ground through two metal stakes (electrodes) and the resulting voltage between two other metal stakes is measured. The depth of measurement is decided by the distance and the arrangement pattern of the four electrodes (Fig. 4.7) and the standard calibration curves. The changes in the electrical resistance of different earth layers are thus determined. Table 4.5 lists a typical order of values of resistivity for some common soils. Using the table and the plot of electrical resistivity versus depth, one can determine the type of subsurface layers at different depths. The electrical resistivity would vary with the salinity of the water included in the pores of earth material. Therefore, one should be careful in interpreting the results. It is advisable to prepare tables, similar to Table 4.5, or histograms of the resistivity for different regions and use these for the interpretation of resistivity measurements.

Table 4.5 Typical values of electrical resistivity for some soils (6)

Earth material	*Electrical resistivity (ohm-metres)*
Clay	1 – 100
Loam	4 – 40
Clayey soil	100 – 380
Sandy soil	400 – 4000
Loose sand	1000 – 180,000
River sand and gravel	100 – 4000
Chalk	4 – 100
Limestones	40 – 3000
Sandstones	20 – 20,000
Basalt	200 – 1000
Crystalline rocks	10^3 – 10^6

(*ii*) ***Seismic Refraction Method***

This geophysical method employs seismic waves to determine variations in the thickness of the unconfined aquifer and the zone where the most permeable strata are likely to exist. The method is based on the velocity variation of artificially generated seismic waves in the ground. Seismic waves are generated either by hammering on a metal plate, or by dropping a heavy ball, or by using explosives. The time between the initiation of a seismic wave on the ground and its first arrival at a detector (seismometre) placed on the ground is measured. For the seismic refraction method, one is interested only in the arrival of the critically refracted ray, *i.e.* the ray which encounters the boundary at such an angle that when it refracts in the lower medium, it travels parallel to the boundary at a higher velocity (2). The critically refracted ray travelling along the boundary radiates wavefronts in all directions and some of which return to the surface (Fig. 4.8). Using the appropriate formulas and the time-distance graph, one can determine the depth of the bedrock. Some representative values of refracted seismic wave velocities in different soils are given in Table 4.6. This method is more precise than the electrical resistivity method in the determination of the depth to bedrock (2). The depth of

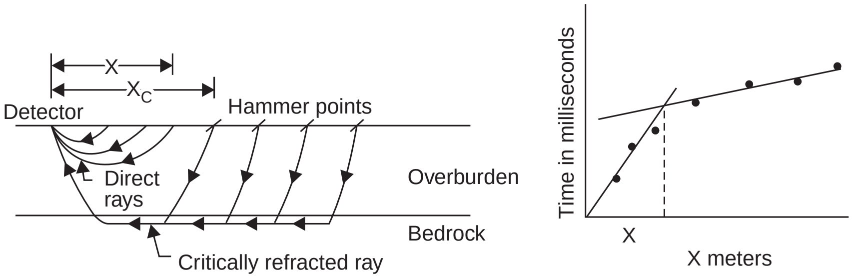

Fig. 4.8 Seismic-refracted rays and time-distance graph

Table 4.6 Representative values of velocity of seismic refracted waves in some soils (15)

Material	*Velocity (m/s)*
Gravel, rubble or dry sand	457–915
Wet sand	610–1830
Clay	915–2740
Water (depnding on temperature and salinity)	1430–1680
Sea water	1460–1520
Sandstone	1830–3960
Shale	2740–4270
Chalk	1830–3960
Limestone	2130–6100
Salt	4270–5180
Granite	4570–5790
Metamorphic rocks	3050–7010

water table in sand gravel formation can also be determined accurately because of the sudden change in seismic velocity at the water table. One important requirement for the seismic refraction method to give accurate results is that the formations must be successively denser with increasing depths.

4.6.3. Well Logging Methods

Surface methods of ground water exploration do not give exact quantitative information about the subsurface environment. Quantitative information about subsurface strata can only be obtained by subsurface investigations which are conducted by personnel working on the surface and the equipment being lowered underground. The equipment extending into the ground measures one of several geophysical quantities, such as electrical resistivity, self-potential, temperature, gamma rays, and so on. Based on these measurements, well logs are prepared. For obtaining electrical resistivity log, one or more electrodes suspended on a conductor cable are lowered into a borehole filled with drilling fluid (6). An electric current is passed between these electrodes and other electrodes placed on the ground. The logging instrument measures the resistance to a flow of current between the electrodes. Thus the electrical resistivity is measured at different depths. The resistivity of any stratum depends primarily on its characteristics and the mineral content of water contained in the stratum.

Self potentials (or spontaneous potentials) are naturally-occurring electrical potentials which result from chemical and physical changes at the contacts between different types of subsurface geologic materials (6). For measuring the self potential at any depth, an electrode is lowered into an uncased borehole filled with drilling fluid by means of an electric cable connected to one end of a millivoltmeter. The other end of this millivoltmeter is connected to a ground terminal at the surface which is usually placed in a mud pit. No external source of current is required.

In gamma logging, natural radiation coming from different strata encountered in the borehole is measured. Such a log can yield qualitative information about subsurface strata.

4.6.4. Test Drilling

All geophysical exploration methods – surface as well as subsurface – and remote sensing methods are quicker and economic but yield results which may be interpreted in more than one way. Test drilling, however, provides the most positive information about the subsurface conditions. Test drilling can predict the true geohydrologic character of subsurface formations by drilling through them, obtaining samples, recording geologic logs, and conducting aquifer tests (2). The following data are usually obtained in test drilling:

(*i*) Identification, location, and elevation of the site of each hole,

(*ii*) Geologic log of the strata penetrated,

(*iii*) Representative samples of strata penetrated,

(*iv*) Depth to static water level in each permeable stratum, and

(*v*) Water quality samples and aquifer test data from water-bearing formations.

Rotary drilling and cable tool drilling are commonly used methods of drilling wells. The rotary drilling method is fast and is the most economical method of drilling wells in unconsolidated formations. However, accurate logging of cuttings is relatively difficult and the depth of water level cannot be predicted accurately unless electric logs have been taken for this purpose. Care should be taken in distinguishing between valid cuttings carried in mud suspension and the cuttings which have been delayed in reaching the surface. Cable tool drilling is suitable for drilling to moderate depths. Sampling of geologic materials is relatively more accurate and presents fewer difficulties. Cable tool drilling is, however, a more time-consuming method.

A good lithologic well log presents variation in the geohydrologic character of subsurface formation with depth and also the depth of the water table.

4.7. PUMPING TESTS (Or AQUIFER TESTS)

Aquifer characteristics and its performance can be best described by its hydraulic conductivity, transmissivity, and storativity. These quantities can be determined by analysing the data collected during aquifer tests or pumping tests. Measurements during an aquifer test include water levels at observation wells (before the start of pumping, at intervals during pumping, and for some time after pumping), the discharge rate, and the time of any variation in the discharge rate (2).

If the observations correspond to equilibrium conditions, one can use Eq. (4.16) for confined aquifers and Eq. (4.21) for unconfined aquifers to determine the hydraulic conductivity. Thus, for two observation wells located at distances r_1 and r_2 ($r_2 > r_1$) from the pumping well, Eq. (4.16) yields

$$K = -\frac{Q \log (r_2/r_1)}{2.73\, B\, (h_2 - h_1)} \tag{4.35}$$

in which, Q is negative for the pumping well. Similarly, Eq. (4.21) would yield

$$K = -\frac{Q \log (r_2/r_1)}{1.366\, (H_2^{\,2} - H_1^{\,2})} \tag{4.36}$$

For non-equilibrium conditions in confined aquifer, Eq. (4.28) would yield

$$T = \frac{0.183\, Q}{(s_2 - s_1)} \log \frac{t_2}{t_1} \tag{4.37}$$

Here, s_1 and s_2 are the drawdowns in an observation well (r distance away from the pumping well) at two different times t_1 and t_2 (from the beginning of pumping), respectively. If t_2 is chosen as 10 t_1 and $s_2 - s_1$ for this case be denoted by Δs, Eq. (4.37) is reduced to

$$T = \frac{0.183\, Q}{\Delta s} \tag{4.38}$$

Having known T, the storativity S can be determined from Eq. (4.28) by substituting suitable values of t and s obtained from the time-drawdown graph as illustrated in the following example.

Example 4.5 A well pumps water at a rate of 2500 m^3/day from a confined aquifer. Drawdown measurements in an observation well 120 m from the pumping well are as follows:

Time since pump started in minutes	*Drawdown s in metres*	*Time since pump started in minutes*	*Drawdown s in metres*
1	0.05	14	0.40
1.5	0.08	18	0.44
2	0.12	24	0.48
2.5	0.14	30	0.52
3	0.16	50	0.61
4	0.20	60	0.64
5	0.23	80	0.68
6	0.27	100	0.73
8	0.30	120	0.76
10	0.34	150	0.80
12	0.37		

Determine the aquifer characteristics S and T assuming that Eq. (4.28) is valid.

Solution:

From the time-drawdown graph (Fig. 4.9)

$$\Delta s = 0.39 \text{ m}$$

Using Eq. (4.38),

$$T = \frac{0.183\,(2500)}{0.39} = 1173.1 \text{ m}^2/\text{day}$$

Substituting s = 0.74 m for t = 100 min = 0.07 day in Eq. (4.28)

$$0.73 = \frac{0.183\,(2500)}{1173.11} \log \frac{2.25\,(1173.1)\,(0.07)}{(120)^2 S}$$

$$\therefore \qquad S = 1.72 \times 10^{-4}$$

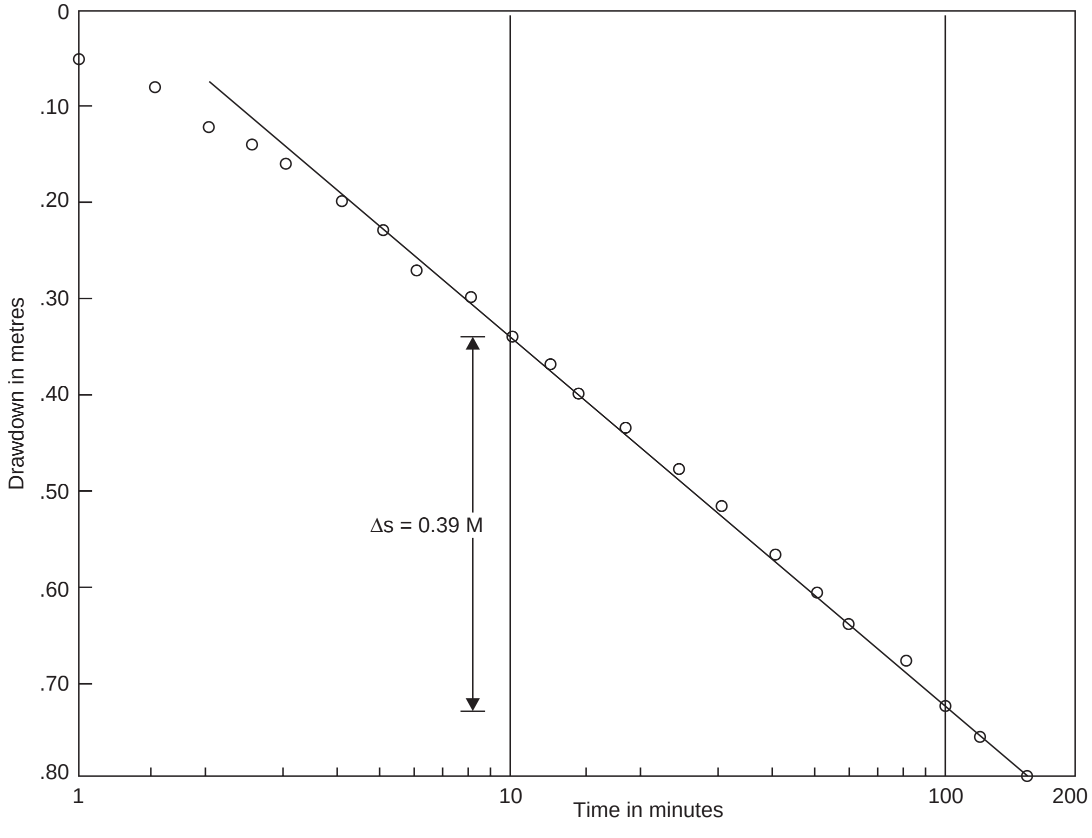

Fig. 4.9 Time-drawdown graph (Example 4.5)

4.8. DESIGN OF WATER WELLS

Well design is the process of specifying the physical materials and dimensions for various well components. The main objectives of well design are (6):

(*i*) To obtain the highest yield with a minimum drawdown consistent with aquifer capability and well requirement,

(*ii*) To obtain good quality water with proper protection from contamination,

(*iii*) To obtain sand-free water.

(*iv*) To ensure long life (30–40 years) of well, and

(*v*) To have reasonable installation, maintenance, and operation costs.

The designer needs the following hydrogeologic information for making the design (6):

(*i*) Stratigraphic information concerning the aquifer and overlying formations,

(*ii*) Transmissivity and storage coefficient of the aquifer,

(*iii*) The present and long-term water balance (*i.e.,* inflow and outflow) conditions in the aquifer,

(*iv*) Grain size analyses of unconsolidated aquifer materials and identification of rocks and minerals, and

(*v*) Water quality.

A water well has two main components – the casing and the intake portion. The casing serves as a vertical conduit for water flowing upward and also houses the pumping equipment. Some of the borehole length may, however, be left uncased if the well is constructed in consolidated rock. The intake portion in unconsolidated and semi-consolidated aquifers is usually screened. The well screen prevents fine aquifer material from entering the well with water and also serves to retain the loose formation material. In consolidated rock aquifer, the intake portion of the well may simply be an open borehole drilled into the aquifer.

Standard design procedure for a water well involves the following steps:

(*i*) Selection of strata to be screened,

(*ii*) Design of well casing and housing pipe, and

(*iii*) Design of well screen.

Before starting a well design project it is worthwhile for the designer to study the design, construction, and maintenance of other wells in the area. The design practices may vary in different regions because of the hydrogeologic conditions.

4.8.1. Selection of Strata to be Screened

The samples collected during drilling are sieve-analysed and a lithologic well log is prepared. This log describes the characteristics (type of material, size distribution, values of d_{10} or d_{17}, d_{50}, d_{60}, *etc.*, and uniformity coefficient d_{60}/d_{10}) of different subsurface strata. The lithologic log helps determine the thickness and permeability of each aquifer. The aquifers to be screened are thus decided.

4.8.2. Design of Well Casing and Housing Pipe

The well casing should meet the following requirements (16):

(*i*) It should have a smooth exterior to minimise frictional resistance between the casing and the subsurface formations.

(*ii*) It should be of adequate size to permit the passage of drilling tools, operation of well development equipment, and installation of pumps. Its size must also assure the uphole velocity of 1.5 m/s or less so that the head loss is small.

(*iii*) The walls of the casing pipe must be of sufficient thickness and suitable material to resist stresses and corrosive action of ground water environment. The life of the casing pipe should be about 30 to 40 years after its installation. Cupronickel alloys, copper-bearing steel, stainless steel, P.V.C. pipes and fibre glass-reinforced epoxy pipes are the desirable types for casing material.

(*iv*) The field joints of the casing pipe must be leak-proof and have adequate strength.

The casing pipe, when used as a housing pipe, should have sufficiently large diameter at the housing elevation to accommodate the pump with enough clearance for its installation and operation. The housing pipe should have its diameter at least 5.0 cm greater than the nominal diameter of the pump and is set a few metres below the lowest drawdown level taking into account seasonal fluctuations and future development of ground water in the area. Table 4.7 presents recommended sizes of casing (*i.e.,* well diameter) for different well yields.

Table 4.7 Recommended well diameters for different pumping rates (6)

Anticipated pumping rate (m^3/day)	*Nominal size of pump bowls (mm)*	*Optimum size of well casing (mm)*	*Smallest size of well casing (mm)*
Less than 540	102	152 ID	127 ID
410– 950	127	203 ID	152 ID
820– 1910	152	254 ID	203 ID
1640– 3820	203	305 ID	254 ID
2730– 5450	254	356 OD	305 OD
4360– 9810	305	406 OD	356 OD
6540– 16400	356	508 OD	406 OD
10900– 20700	406	610 OD	508 OD
16400– 32700	508	762 OD	610 OD

4.8.3. Design of Well Screen

The design of a well screen (*i.e.,* its length, slot, open area, diameter, and material) is the most important aspect of a well design. The basic requirements of a well screen are as follows (16):

(*i*) It should be corrosion resistant,

(*ii*) It should be strong enough to prevent collapse,

(*iii*) It should prevent excessive movement of sand into the well, and

(*iv*) It should have minimum resistance to the flow of water into the well.

4.8.3.1. Length of Well Screen

The intake portion of a well must, obviously, be placed in the zones of the maximum hydraulic conductivity. Such zones are determined by interpreting the lithologic log, visual inspection and sieve analysis of the samples collected during drilling, laboratory tests for hydraulic conductivity and the results of pumping tests. The optimum length of the well screen depends primarily on the nature of the aquifer stratification and the permissible drawdown.

In the case of a homogeneous unconfined aquifer of thickness less than 45 m the screening of the bottom one-third to one-half of the aquifer is recommended (6). In thick and deep aquifers, however, as much as 80 per cent of the aquifer may be screened to obtain a higher specific capacity and greater efficiency even though the resulting yield may be less. These guidelines are applicable to non-homogeneous unconfined aquifers also. However, screen sections are positioned in the most permeable layers of the lower portions of the aquifer (leaving depth of about 0.3 m at the upper and lower ends of the screen to prevent finer material of the transition zone from moving into the well) so that maximum drawdown is available. Wherever possible, the total screen length should be approximately one-third of aquifer thickness.

For homogeneous confined aquifers, the central 80 to 90 percent of the aquifer thickness should be screened assuming that the water level in the well would always be above the upper boundary of the aquifer. In case of non-homogeneous confined aquifer, 80 to 90 per cent of the most permeable aquifer layers should be screened.

If the effective size of two strata are the same, the stratum with lower uniformity coefficient (*i.e.,* relatively poorly graded) is more permeable and should, therefore, be screened.

4.8.3.2. Well-Screen Slot Openings

Well screen slot openings primarily depend on the size distribution of the aquifer material and also on whether the well is naturally developed or filter-packed (*i.e.,* artificially gravel-packed). Wells in aquifers with coarse-grained (d_{10} > 0.25 mm) and non-homogeneous material can be developed naturally. But wells in aquifers with fine-grained and homogeneous material are best developed using a filter pack (or gravel pack) outside the well screen.

In a naturally developed well, the screen slot size is selected so that most of the finer aquifer materials in the vicinity of the borehole are brought into the screen and pumped from the well during development. The process creates a zone of graded formation materials extending 0.3 to 0.6 m outward from the screen (6). The slot size for the screen of such wells can be selected from Table 4.8.

Table 4.8 Selection of slot size for well screen (17)

Uniformity coefficient of the aquifer being tapped	*Condition of the overlying material*	*Slot size in terms of aquifer material size*
> 6	fairly firm; would not easily cave in	d_{70}
> 6	soft; would easily cave in	d_{50}
= 3	fairly firm; would not easily cave in	d_{60}
= 3	soft; would easily cave in	d_{40}

If more than one aquifer is tapped, and the average size of the coarsest aquifer is less than four times the average size of the finest aquifer, the slot size should correspond with the finest aquifer. Otherwise, slot size must vary and correspond with the sizes of aquifer material (16). A more conservative slot size should be selected if: (*i*) there is some doubt about the reliability of the samples, (*ii*) the aquifer is thin and overlain by fine-grained loose material, (*iii*) the development time is at a premium, and (*iv*) the formation is well-sorted. Under these conditions, slot sizes which will retain 40 to 50 per cent of the aquifer material (*i.e.,* d_{60} to d_{50}) should be preferred (6).

In filter-packed wells, the zone in the immediate vicinity of the well screen is made more permeable by removing some formation material and replacing it with specially graded material. This filter pack or gravel pack separates the screen from the aquifer material and increases the effective hydraulic diameter of the well. A filter pack is so designed that it is capable of retaining 90 per cent of the aquifer material after development. Well screen openings should be such that they can retain 90 per cent of the filter pack material (6). The filter pack material must be well graded to yield a highly porous and permeable zone around the well screen. The uniformity coefficient of the filter pack should be 2.0 or less so that there is less segregation during placing and lower head loss through the pack. The filter pack material should be clean and well-rounded. Clean material requires less development time and also results in little loss of material during development. Well-rounded grains make the filter pack more permeable which reduces drawdown and increases the yield. Further, the filter pack must contain 90 to 95% quartz grains so that there is no loss of volume caused by the dissolution of materials. For minimum head loss through the filter pack and minimum sand movement, the pack-aquifer ratio (*i.e.*, the ratio of the average size of the filter pack material to the average size of the aquifer material) should be as follows (16):

	Pack-aquifer Ratio
(*i*) Uniform aquifer with uniform filter pack	9–12.5
(*ii*) Non-uniform aquifer with uniform filter pack	11–15.5

The thickness of the filter pack designed in this manner should be between 15 and 20 cm.

4.8.3.3. Open Area of Well-Screen

For head loss through the well screen to be minimum, Peterson *et al.* (18) have suggested that the value of the parameter $C_c A_p L/D$ should be greater than 0.53. Here, C_c is the coefficient of contraction for the openings, A_p the ratio of the open area to the total surface area of the screen, L the screen length, and D is the diameter of well screen. Theoretical studies conducted at the UP Irrigation Research Institute have shown that the parameter $C_c C_v A_p L/D$ should be greater than 1.77. Here, C_v is the coefficient of velocity. A factor of safety of 2.5 is further recommended by Sharma and Chawla (16). This means that $C_c C_v A_p L/D$ should be greater than 4.42.

4.8.3.4. Diameter of Well Screen

The screen diameter should be such that there is enough open area so that the entrance velocity of water generally does not exceed the design standard of 3 cm/s (6). Table 4.9 gives values of the optimum diameter for different values of the yield and hydraulic conductivity of the aquifer. These values have been worked out considering the cost of screens, cost of boring and the running expenses (16). USBR's recommended values have also been given in Table 4.9.

Table 4.9 Optimum diameter of well screen (16, 19)

Well discharge (m^3/s)	*Optimum diameter of well screen in cm for hydraulic conductivity equal to*			*USBR's recommended value of well screen diameter (cm)*
	0.04 cm/s	*0.09 cm/s*	*0.16 cm/s*	
0.04	15	18	22	25
0.08	20	25	30	30
0.12	23	28	33	35
0.16	25	30	35	40

4.8.3.5. Entrance Velocity

The entrance velocity of water moving into the well screen should be kept below a permissible value which would avoid movement of fine particles from the aquifer and filter pack to the well. The permissible entrance velocity depends on the size distribution and the granular structure of the aquifer material, the chemical properties of the ground water and shape of the screen openings. Its exact evaluation is difficult. The permissible entrance velocity is usually taken as 3 cm/s for the design of the well screen (6, 16).

4.8.3.6. Well Screen Material

Four factors govern the choice of material used for the construction of a well screen. These are: (*i*) water quality, (*ii*) presence of iron, (*iii*) strength requirements of screen, and (*iv*) cost of screen. Quality analysis of ground water usually shows that the water is either corrosive or incrusting. Corrosive water is usually acidic and contains dissolved oxygen and carbon dioxide which accelerate the corrosion. Corrosion may further increase due to higher entrance velocities. Incrustation is caused due to precipitation of iron and manganese hydroxides and other materials from water. It is, therefore, important to use corrosion-resistant materials for the fabrication of a well screen. The following alloys (in decreasing order of their ability to resist corrosion) or their suitable variations are used for the fabrication of well screens (16).

(*i*) Monel alloy (or Monel metal) (70% nickel and 30% copper)

(*ii*) Cupro-nickel (30% nickel and 70% copper)

(*iii*) Everdur A alloy (96% copper, 3% silicon and 1% manganese)

(*iv*) Stainless steel (74% low carbon steel, 18% chromium and 8% nickel)

(*v*) Silicon red brass (83% copper, 1% silicon and 16% zinc)

(*vi*) Anaconda brass (or Gilding metal) (85% copper and 15% zinc)

(*vii*) Common yellow brass (67% copper and 33% zinc)

(*viii*) Armco iron (99.84% pure iron)

(*ix*) Low carbon steel

(*x*) Ordinary cast iron.

4.9. MEHODS OF WELL CONSTRUCTION

The operations involved in well construction are drilling, installing the casing, placing a well screen and filter pack, and developing the well to ensure maximum sand-free water yield. Shallow wells, generally less than about 15 m deep, are constructed by digging, boring, driving or jetting. Deep wells are constructed using drilling methods. Wells used for irrigation purposes are generally deep.

4.9.1. Digging

Wells in shallow and unconsolidated glacial and alluvial aquifers can be dug by hand using a pick and shovel. Loose material is brought to the surface in a container by means of rope and pulleys. The depth of a dug well may vary from about 3 to 15 m depending upon the position of the water table. Dug wells usually have large diameter ranging from about 1 to 5 m. Dug wells must penetrate about 4 to 6 m below the water table. The yield of the dug wells is generally small and is of the order of about 500 litres per minute.

4.9.2. Boring

Hand-operated or power-driven earth augers are used for boring a well in shallow and unconsolidated aquifers. A simple auger has a cutting edge at the bottom of a cylindrical container (or bucket). The auger bores into the ground with rotary motion. When the container is full of excavated material, it is raised and emptied. Hand-bored wells can be up to about 20 cm in diameter and about 15 m deep. Power-driven augers can bore holes up to about 1 m in diameter and 30 m deep (4).

4.9.3. Driving

In this method, a series of connected lengths of pipe are driven by repeated impacts into the ground to below the water table. Water enters the well through a screened cylindrical section which is protected during driving by a steel cone at the bottom. Driven wells can be installed only in an unconsolidated formation relatively free of cobbles or boulders. The diameters of driven wells are in the range of about 3–10 cm. Such wells can be constructed up to about 10 m, if hand driven, and up to about 15 m when heavy hammers of about 300 kg are used. The maximum yield of driven wells is usually around 200 litres per minute. The main advantage of a driven well is that it can be constructed in a short time, at minimum cost, and by one man.

4.9.4. Jetting

The jetting (or jet drilling) method uses a chisel-shaped bit attached to the lower end of a pipe string. Holes on each side of the bit serve as nozzles. Water jets through these nozzles keep the bit clean and help loosen the material being drilled. The fluid circulation system is similar to that of a direct rotary drilling method. With water circulation maintained, the drill rods and the bit are lifted and dropped in manner similar to cable tool drilling but with shorter strokes. Jet drilling is limited to drilling of about 10 cm diameter wells to depths of about 60 m, although larger diameter wells have been drilled up to about 300 m by this method (6). Other drilling methods have replaced jet drilling for deep and larger diameter wells.

4.9.5. Cable Tool Drilling

It is the earliest drilling method developed by the Chinese some 4000 years ago. A cable tool drilling equipment mainly consists of a drill bit, drill stem, drilling jars, swivel socket, and cable (Fig. 4.10). The cable tool drill bit is very heavy (about 1500 kg) and crushes all types of earth materials. The drill stem provides additional weight to the bit and its length helps in maintaining a straight vertical hole while drilling in hard rock. The length of the drill stem varies from about 2 to 10 m and its diameter from 5 to 15 cm. Its weight ranges from 50 to 1500 kg. Drilling jars consist of a pair of linked steel bars and help in loosening the tools when these stick in the hole. Under the normal tension of the drilling line, the jars are fully extended. When tools get stuck, the drilling line is slackened and then lifted upward. This causes an upward blow to the tools which are consequently released. The swivel socket (or rope socket) connects the string of tools to the cable. The wire cable (about 25 mm in diameter) which carries and rotates the drilling tool on each upstroke is called the drill line. The cable tool drilling rig mainly consists of a mast, a multiline hoist, a walking beam, and an engine. Drill cuttings are removed from the well by means of bailers having capacities of about 10 to 350 litres. A bailer is simply a pipe with a valve at the bottom and a ring at the top for attachment to the bailer line. The valve allows the cuttings to enter the bailer but prevents them from escaping. Another type of bailer is called the sand pump or suction bailer which is fitted with a plunger. An upward pull on the plunger produces a vacuum which opens the valve and sucks sand or slurried cuttings into the tubing. Most sand pumps are about 3 m long.

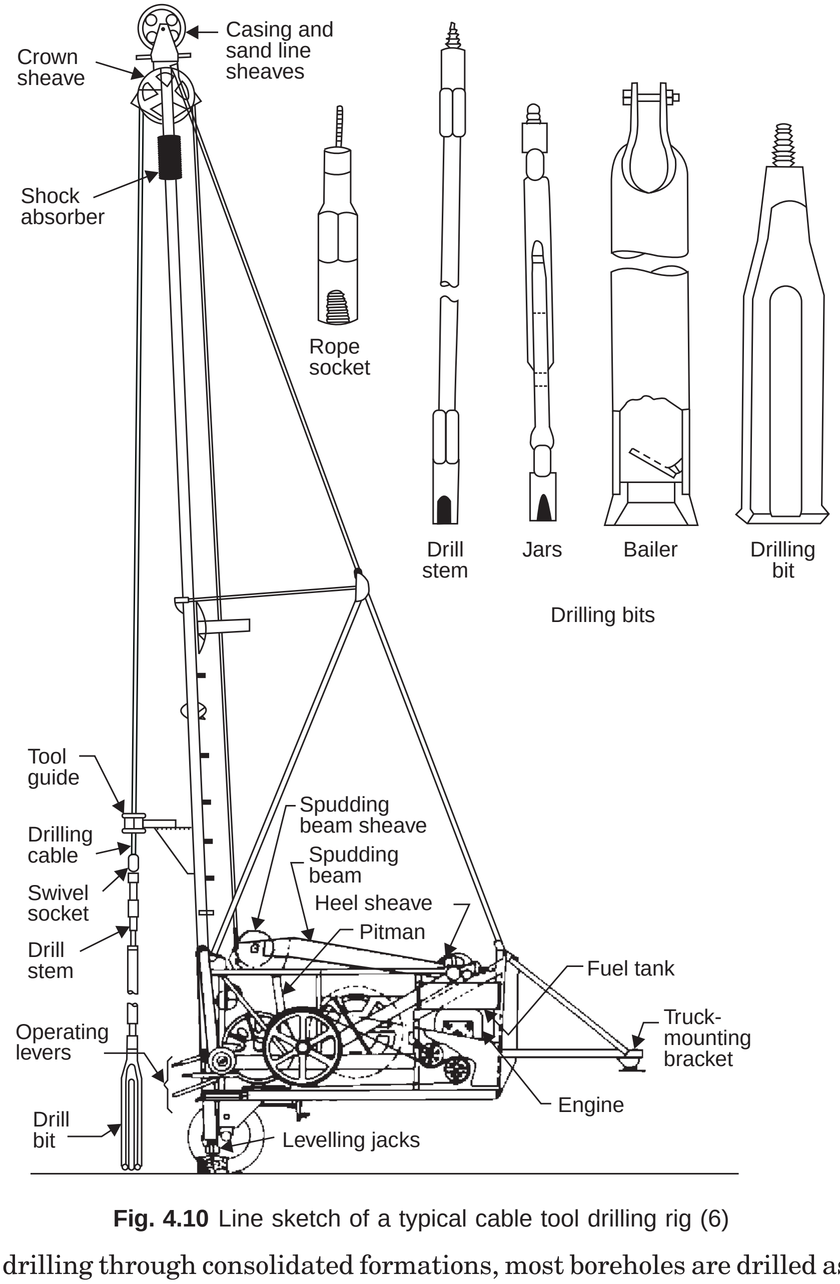

Fig. 4.10 Line sketch of a typical cable tool drilling rig (6)

While drilling through consolidated formations, most boreholes are drilled as "open hole", *i.e.,* no casing is used during the drilling operation. In such conditions the cable tool bit is essentially a crusher. On the other hand, there is a danger of caving in while drilling through unconsolidated formations. For this reason, the casing pipe must follow the drill bit closely to keep the borehole open in unconsolidated formations. Also, in the case of such formations, the drilling action of the bit is primarily a loosening and mixing process. Actual crushing would take place only if a large stone or boulder were encountered.

For the driving operation of the casing pipe, a drive head is fitted to the top of the casing. The drive head serves as an anvil and protects the top of the casing. Similarly, a drive shoe made of hardened and tempered steel is attached to the lower end of the casing pipe. The shoe prevents the damage to the bottom end of the casing pipe when it is being driven. The casing is driven down by means of drive clamps, constructed of heavy steel forgings made in halves, fastened to the top of the drill stem. Drive clamps act as the hammer face and the up-and-down motion of tools provides the weight for striking the top of the casing pipe and thus driving it into the ground.

The procedure for drilling through unconsolidated formation consists of repeated driving, drilling, and bailing operations. The casing pipe is initially driven for about 1 to 3 m in the ground. The material within the casing pipe is then mixed with water by the drill bit to form slurry. The slurry is bailed out and the casing pipe is driven again. Sometimes, the hole is drilled 1 to 2 m below the casing pipe; the casing is then driven down to the undisturbed material and drilling is resumed. The drilling tools make 40 to 60 strokes of about 40 to 100 cm length every minute. The drill line is rotated during drilling so that the resulting borehole is round. The slurry formed by the mixing of cuttings with added water (if not encountered in the ground) reduces the friction on the cutting bit and helps in bailing operations.

If the friction on the outside of the casing pipe increases so much that it cannot be driven any more or if further driving might damage the pipe, a string of smaller casing is inserted inside the first one. Drilling is thus continued. Sometimes, two or three such reductions may be required to reach the desired aquifer. The diameter of the well is reduced. If such a situation is anticipated, the casing in the upper part should be of larger diameter. The drilling process through consolidated formation, not requiring casing, would consist of repeated drilling and bailing operations only.

The cable tool method has survived for thousands of years mainly because of its suitability in a wide variety of geological conditions. It offers the following advantages (6):

(*i*) Cable tool drilling rigs are relatively cheaper.

(*ii*) The rigs are simpler and do not require sophisticated maintenance.

(*iii*) The machines have low power requirements.

(*iv*) The borehole is stable during the entire drilling operation.

(*v*) Recovery of reliable samples is possible at every depth.

(*vi*) Wells can be drilled in water-scarce areas.

(*vii*) Because of their size, the machines can be operated in more rugged, inaccessible terrain or in other areas where limited space is available.

(*viii*) Wells can be drilled in formations where water is likely to be lost.

Slow driling rate, higher cost of casing pipe, and difficulty in pulling back long strings of casing pipes are some of the disadvantages of cable tool drilling.

4.9.6. Direct Rotary Drilling

Direct rotary drilling is the fastest method of drilling deep wells of diameters of up to 45 cm (or more with the use of reamers) through unconsolidated formations. The drilling bit is attached to a heavy drill pipe which is screwed to the end of the kelly which is a drill pipe of square section (Fig. 4.11). The drill collar or stabilizer helps in maintaining, straight hole in soft formations through its large wall contact. The drill pipe is turned by a rotating table which fits closely round the kelly and allows the drill rod to slide downward as the hole deepens. The drilling rig consists of a mast, a rotating table, a pump, a hoist, and an engine. The borehole is drilled by rotating a hollow bit attached to the lower end of a string of a drill pipe. Cuttings are

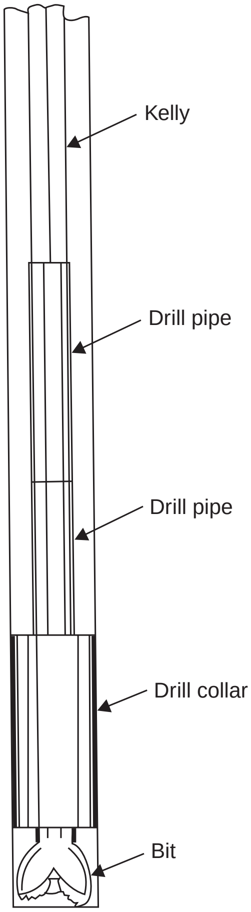

Fig. 4.11 Drill string for rotary drilling

removed continuously by pumping drilling fluid (a mixture of clay and water with some additives to make it viscous) down the drill pipe and through the orifices in the bit. The drilling fluid then flows upward through the annular space between the drill pipe and the borehole, carrying the cuttings in suspension to the surface settling pits where the cuttings settle down in the pits. The clear drilling fluid is pumped back into the borehole. The settling pits can either be portable or excavated for temporary use during drilling and then backfilled after completion of the well. Usually no casing is required during drilling because the drilling mud forms a clay lining on the borehole walls which prevents the formation materials from caving in. After drilling, the casing pipe with perforated sections opposite the aquifers is lowered into the borehole. The drilling rotary method has become the most common due to its following advantages (6):

(*i*) Drilling rates are relatively high.

(*ii*) Minimum casing is required during drilling.

(*iii*) Rig mobilisation and demobilisation are fast.

(*iv*) Well screens can be set easily as part of the casing installation.

Some of the major disadvantages of the direct rotary method are as follows (6):

(*i*) Drilling rigs are expensive.

(*ii*) It is costly to maintain them.

(*iii*) The mobility of the rigs is restricted depending on the slope and wetness of the land surface.

(*iv*) The collection of accurate samples requires special procedure.

(*v*) The drilling fluid may cause the plugging of some aquifer formations.

4.9.7. Reverse Rotary Drilling

The direct rotary drilling method is capable of drilling boreholes with a maximum diameter of about 60 cm. High-capacity wells, particularly those with filter pack, need to be much larger in size. Besides, the drilling rate becomes smaller with increase in borehole diameter in the case of direct rotary drilling. To overcome these limitations, the reverse rotary drilling technique has been developed. This technique is capable of drilling boreholes of about 1.2 m diameter in unconsolidated formation. Recently, the reverse rotary method has been used in soft consolidated rocks such as sandstone, and even in hard rocks using both water and air as the drilling fluid (6).

In reverse rotary drilling, the flow of the drilling fluid is opposite to that in direct rotary drilling. The reverse rotary drilling rig is similar to the direct rotary drilling rig except that it requires larger-capacity centrifugal pumps, a larger diameter drill pipe, and other components also of larger size. The drilling fluid moves down the annular space between the borehole wall and the drill pipe, and picks up the cuttings before entering the drill pipe through the ports of the drill bit. The drilling fluid, along with its cuttings load, moves upwards inside the drill pipe which has been connected to the suction end of the centrifugal pump through the kelly and swivel. The mixture is brought to a settling pit where the cuttings settle at the bottom and the drilling fluid (*i.e.,* muddy water) moves down the borehole again. The drilling fluid is usually water mixed only with fine-grained soil. The hydrostatic pressure and the velocity head of the drilling fluid moving down the borehole supports the borehole wall. To prevent the formation from caving in, the fluid level must always be up to the ground surface even when drilling is suspended temporarily. The advantages of the reverse rotary drilling method are as follows (6):

(*i*) The formation near the borehole is relatively undisturbed compared to other methods.

(*ii*) Large-diameter holes can be drilled rapidly and economically.

(*iii*) No casing is required during the drilling operation.

(*iv*) Well screens can be set easily while installing the casing.

(*v*) The boreholes can be drilled through most geologic formations, except igneous and metamorphic rocks.

(*vi*) Because of the low velocity of the drilling fluid, there is a little possibility of its entering the formation.

The disadvantages of the reverse rotary drilling method are as follows (6):

(*i*) A large quantity of water is needed.

(*ii*) The reverse rotary drilling rig is costlier because of larger size of equipment.

(*iii*) Large mud pits are required.

(*iv*) Some drill sites may be inaccessible because of the larger size of the rig.

4.10. WELL COMPLETION

After drilling a well, the well screen and filter pack (wherever necessary) are to be placed and the casing removed. If the formations are sufficiently strong and stable, ground water may directly enter the uncased well. In unconsolidated formations, however, a casing with perforation (or a well screen) is needed to support the outside material and also to admit water freely into the well.

The installation of the well screen and the removal of the casing is best done by the pull-back method in which the casing is installed to the full depth of the well and the well screen, whose size is smaller than that of casing, is lowered inside the casing. The casing is then pulled back or lifted far enough to expose the screen to the water-bearing formation (Fig. 4.12).

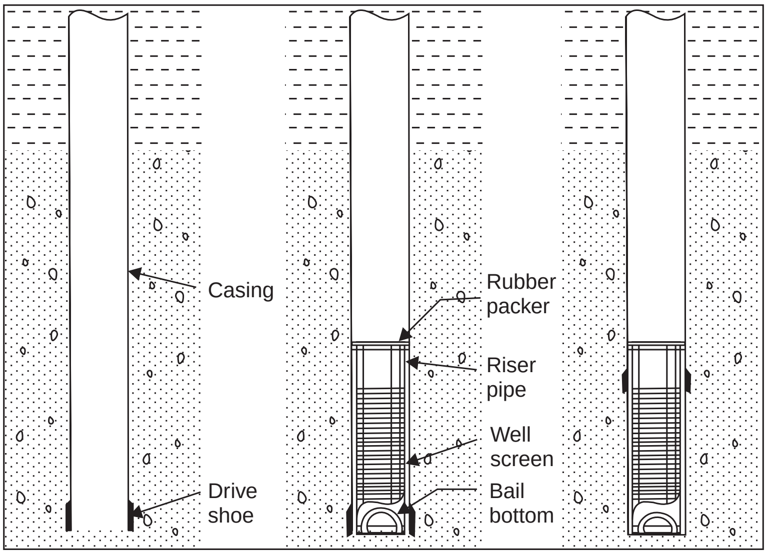

Fig. 4.12 Pull-back method of installing screen

In the case of rotary-drilled wells, setting the casing to the bottom of the hole and then pulling it back may appear to be extra and unnecessary work in view of the drilling fluid supporting the borehole wall. But this extra work prevents serious problems which may arise on account of premature caving in which may occur when the viscosity of the drilling fluid is reduced prior to development. This casing is also useful when there is likely to be a longer period between drilling and screen installation, and during which period a momentary loss of drilling fluid may cause partial collapse of the borehole.

A filter pack is generally placed in large-diameter wells by the reverse circulation of the fluid in the well as the filter pack material is fed into the annular space outside the screen by a continuous-feed hopper. When the filter pack material fills the space around the well screen, the transporting water is drawn upward through the screen openings (Fig. 4.13).

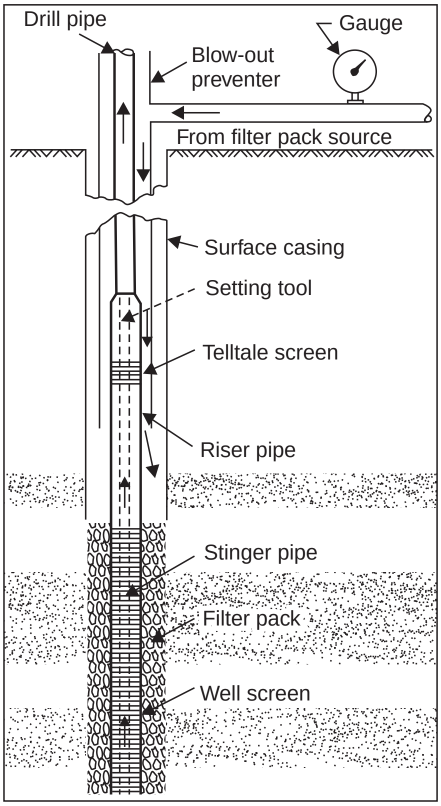

Fig. 4.13 Filter packing of wells

4.11. DEVELOPMENT OF WELLS

Drilling operations for well excavation change the hydraulic characteristics of the formation materials in the vicinity of the borehole. Very often, these changes result in the reduction of the hydraulic conductivity close to the borehole. When a well is drilled with a cable tool rig equipped with a casing driver, the repeated blows on the casing rearrange the grains in the vicinity of the casing. In rotary drilling methods, the drilling fluids containing clay may flow into the aquifer for some distance and thus plug the pore spaces of the permeable formation. Before commissioning the well for use, it is, therefore, necessary to repair the damage done to the aquifer by the drilling operations. Besides, there is also a need to improve the basic physical characteristics of the aquifer in the vicinity of the well screen so that water can flow more freely into the well. A well is, therefore, 'developed' in order to attain these two objectives, and thus, maximise well yield. Well development involves applying some form of energy to the water-bearing formation in the vicinity of the well so as to remove fine materials (including drilling mud) from the aquifer and rearrange formation particles so that the well yields clear sand-free water in maximum quantity with minimum drawdown. Well development serves the following beneficial purposes :

(*i*) It increases the permeability of the aquifer material surrounding the well and filter pack (if present) by:

(*a*) reducing the compaction and intermixing of grains of different sizes during drilling by removing fine grains,

(*b*) removing the filter cake or drilling fluid film that coats the borehole,

(*c*) removing much or all of the drilling fluid which has entered the aquifer,

(*d*) breaking sand-grain bridging across the screen openings, and

(*e*) increasing the natural porosity of the previously undisturbed formation near the borehole by removing the finer fraction of the aquifer material.

(*ii*) It creates a graded zone of aquifer material around the screen in a naturally developed well. This effect stabilises the formation so that the well will yield sand-free water.

(*iii*) It reduces the head loss near the well screen.

(*iv*) It increases the useful life of the well screen.

(*v*) It brings the well to its maximum specific capacity, *i.e.,* the maximum yield at minimum drawdown.

The methods usually adopted for well development are as follows (6):

(*i*) Overpumping,

(*ii*) Backwashing.

(*iii*) Mechanical surging,

(*iv*) Air surging and pumping,

(*v*) High-velocity jetting, and

(*vi*) High-velocity water jetting combined with simultaneous pumping.

There are several variations of most of these methods (16). Only the main features of these methods have been described in the following paragraphs.

4.11.1. Overpumping

In the ovrpumping method, the well is pumped at a discharge rate higher than the discharge rate of the well during its normal operation. The logic of the method is that any well which can be pumped sand-free at a high rate can be pumped sand-free at a lower rate. It is the simplest method of developing wells. However, the development by this method is not effective and the developed well is seldom efficient. The aquifer material is also not fully stabilised. This incomplete development is due to the following reasons:

(*i*) Water flows in only one direction and some sand grains may be left in a bridged condition. The formation is thus partially stabilised.

(*ii*) Most of the development takes place in the most permeable zones of the aquifer which are usually closest to the top of the screen. Therefore, less development takes place in the lower layers of the aquifer.

Besides, this method generally uses the pump intended for regular use during the normal operation of the well. Pumping of silt-laden water at higher rates can reduce efficiency of the pump.

4.11.2. Backwashing

Reversal of flow through the screen openings agitates the aquifer material, removes the finer fraction and rearranges the remaining aquifer particles. These effects usually cause effective development of the well. The “rawhiding” method of backwashing consists of alternately lifting a column of water significantly above the pumping level and then letting the water fall back into the well. To minimise the changes of sand-locking the pump, its discharging rate should be gradually increased to the maximum capacity before stopping the pump. During this process, the well is occasionally pumped to waste to remove the sand brought to the well by the surging action of this method of well development. As in the case of the overpumping method, the surging action may be concentrated only in the upper layers of the aquifer. Besides, the surging effect is not vigorous enough to cause maximum benefits. When compared with other methods of well development, the overall effectiveness of backwashing as well as overpumping methods in case of high-capacity wells is rather limited.

4.11.3. Mechanical Surging

In this method, a close-fitting surge plunger, moving up and down in the well casing, forces water to flow into and out of the well screen. The initial movements of the plunger should be relatively gentle so that the material blocking the screen may go into suspension and then move into the well. To minimise the problem of the fine materials going back to the aquifer from the well, the fine material should be removed from the well as often as possible. The surging method is capable of breaking sand bridges and produces good results. However, it is not very effective in developing filter-packed wells because the water movement is confined only up to the filter pack and the aquifer remains unaffected by the surging action.

4.11.4. Air Surging and Pumping

This method requires two concentric pipes – the inner pipe known as the air line and the outer one known as the pumping pipe (or eductor pipe). The assembly of these pipes is lowered into the well. In air surging, compressed air is injected through air line into the well to force aerated water up through the annular space between the air line and the pumping pipe. As this aerated water reaches the top of the casing, the air supply is stopped so that aerated water column starts falling. Air-lift pump is used to pump the well periodically to remove the sand brought into the well as a result of air surging. The compressed air produces powerful surging action. This method is used to develop wells in consolidated and unconsolidated formations.

4.11.5. High-Velocity Jetting

This method consists of shooting out high-velocity jets of water from a jetting tool to the aquifer through the screen openings. The equipment of this method consists of a jetting tool provided with two or more equally spaced nozzles, high-pressure pump, high-pressure hose and connections, and a water supply source. The forceful action of high-velocity jets loosens the drilling mud and agitates, and rearranges the sand and gravel paritcles around the well. The loosened material is removed by pumping. In this method of development, the entire surface of the screen can be subjected to vigorous jet action by slowly rotating and gradually raising and lowering jetting tool.

This method has the following advantages:

(*i*) The energy is concentrated over a small area with greater effectiveness.

(*ii*) Every part of the screen can be developed selectively.

(*iii*) The method is relatively simple.

The method of jetting is particularly successful in developing highly stratified and unconsolidated water-bearing formations.

4.11.6. High-Velocity Water Jetting Combined with Simultaneous Pumping

The method of high-velocity water jetting results in very effective development of wells. But, maximum development efficiency can be obtained by combining high-velocity water jetting with simultaneous air-lift pumping method (Fig. 4.14). The method requires that the volume of water pumped from a well will always be more than that pumped into it so that the water level in the well is always below the static level and there is a continuous movement of water from the aquifer to the well. This would help remove some of the suspended material loosened by the jetting operation (6).

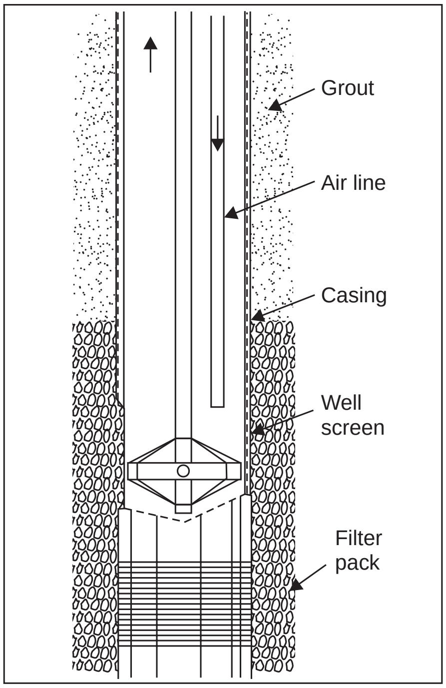

Fig. 4.14 Jetting and air-lift pumping

4.12. PUMPING EQUIPMENT FOR WATER WELLS

In most wells, the static water level is below the ground surface and, hence, flowing wells are rare. The water has to be lifted from inside the well to the ground surface. Rope and bucket with or without windlass have been used and are still used for shallow wells and for low discharges. For deeper wells and high yields of water, pumps have to be used.

The purpose of installing pumps in wells is to lift water from inside the well to the ground surface. Pumps can be broadly classified as shallow well pumps and deep well pumps depending upon the position of the pump and not the depth of the well. A shallow well pump is installed on the ground and lifts water from the well by suction lift. A deep well pump is installed within the well casing and its inlet is submerged below the pumping level. If the pumping level is lower than the limit of a suction lift (about 7.5 m), only the deep well pump should be used.

Pumps are also classified on the basis of their design as positive displacement pumps and variable displacement pumps (6). Positive displacement pumps discharge almost the same volume of water irrespective of the head against which they operate. The input power, however, varies in direct proportion to the head. Such pumps are used extensively in ground water monitoring wells, hand pump-equipped wells, and wind-powered wells. They are rarely used for large-capacity water wells. The most common type of positive displacement pumps is the piston pump.

The variable displacement pumps are used for large-capacity wells. For these pumps, there is an inverse relationship between the discharge and the working head. Maximum input power is required when the pump has to operate at low heads delivering large volumes of water. The major types of variable displacement pumps are as follows (6):

(*i*) Centrifugal pumps:

(*a*) suction lift pump,

(*b*) deep-well turbine pump, and

(*c*) submersible turbine pump.

(*ii*) Jet pumps

(*iii*) Air-lift pumps

4.12.1. Centrifugal Pumps

Centrifugal pumps are the most popular. They are capable of delivering large volumes of water against high as well as low head with good efficiency. Besides, these pumps are relatively simple and compact. The basic principle of centrifugal pumping can be understood by considering the effect of swinging a bucket of water around in a circle at the end of a rope. The centrifugal force causes the water to press against the bottom of the bucket rather than spill out of the bucket. If a hole is cut in the bottom, water would discharge through the opening at a velocity which would depend on the centrifugal force. If an airtight cover were put on the bucket top, a partial vacuum would be created inside the bucket as the water would leave through the opening in the bottom. If a water source is connected to the airtight cover through an intake pipe, the partial vacuum will draw additional water into the bucket as the water is being discharged through the bottom hole. The bucket and cover of this example correspond to the casing of a centrifugal pump; the discharge hole and the intake pipe correspond to the pump outlet and inlet, respectively; the arm that swings the bucket corresponds to the energy source and the rope performs the function of a pump impeller.

Suction-lift pumps create negative pressure at the pump intake. The atmospheric pressure at the free surface of water in the well forces the well water into and up the intake pipe. The maximum suction lift depends on the atmospheric pressure (10.4 m of water head), vapour pressure of water, head loss due to friction, and the head requirements of the pump itself. Under field conditions, the average suction-lift capability of a suction-lift centrifugal pump is about 7.5 m (6).

A deep-well vertical turbine pump consists of one or more impellers housed in a single- or multi-stage unit called a bowl assembly. Each stage gives a certain amount of lift and sufficient number of stages (or bowl assemblies) are assembled to meet the total head requirement of the system (6).

Vertical turbine pumps in high-capacity wells are highly reliable over long periods of time. The motors of these pumps are not susceptible to failure caused by fluctuations in electric supply. Motor repairs can be carried out easily because of their installation on the ground surface. These pumps, however, cannot be used in wells which are out of the alignment. Besides, these pumps require highly skilled personnel for installation and service.

Submersible pumps have bowl assemblies which are the same as those of vertical turbine pumps. But, the motor of the submersible pump is submerged and is directly connected to and located just beneath the bowl assembly. Water enters through an intake screen between the motor (at lower level) and the bowl assembly (at higher level), passes through various stages, and is discharged directly through the pump column to the surface (6).

The motor of a submersible pump is directly coupled to impellers and is easily cooled because of complete submergence. Ground surface noise is also eliminated. The pump can be mounted in casings which are not entirely straight. The pump house is also not necessary. There are, however, electrical problems associated with submerged cables. These pumps cannot tolerate sand pumping and work less efficiently. The motor is less accessible for repairs and cannot tolerate voltage fluctuations.

4.12.2. Jet Pumps

The jet pump is a combination of a centrifugal pump and a nozzle-venturi arrangement as shown in Fig. 4.15. The nozzle causes increased velocity and reduced pressure at point A. The lowered pressure at A draws additional water from the intake pipe and this water is added to the total volume of water flowing beyond A. The venturi tube helps in the recovery of pressure at B with minimum loss of head. Compared to centrifugal pumps, jet pumps are inefficient but have some advantageous features too. These are adaptable to small wells down to a 5 cm inside diameter. All moving parts of the jet pump are accessible at the ground surface. Their design is simple and results in low equipment and maintenance costs.

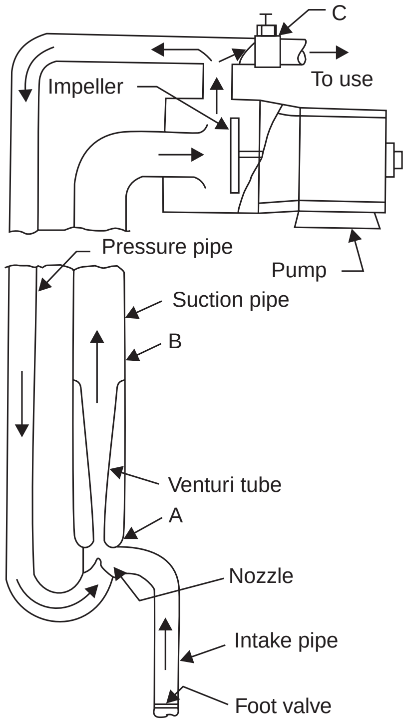

Fig. 4.15 Jet pump (6)

4.12.3. Air-Lift Pumps

Water can also be lifted inside a well by releasing compressed air into an air discharge pipe (air line) lowered into the well. Because of the reduced specific gravity, aerated water is lifted to the ground surface. Air-lift pumping is inefficient and requires cumbersome and expensive equipment and is, therefore, rarely used as a permanent pumping system.

The main factors which must be considered while selecting a pump for water well are the anticipated pumping conditions, specific installation and maintenance conditions, and the basic pump characteristics. As in well construction, the initial cost of a pump and its installation are relatively less important than the performance, reliability, and operating costs during the life span of the pumping equipment.

EXERCISES

4.1 In what respects is ground water better than surface water? Compare well irrigation with canal irrigation.

4.2 What is the meaning of conjunctive use? Why is it useful to consider it in water resources planning?

4.3 Compare surface and subsurface methods of ground water exploration.

4.4 Discuss in brief the steps for designing various components of a water well.

4.5 Describe different methods of well drilling, mentioning their merits and suitability for different field conditions.

4.6 What are the benefits of well development? Describe various methods of well development.

4.7 A constant head permeability test was carried out on a soil sample of diameter 10 cm and length 15 cm. If 100 cc of water was collected in 100 s under a constant head of 50 cm, determine the coefficient of permeability of the soil sample.

4.8 Calculate the amount of water flowing into a coastal aquifer extending to a length of 30 km along the coast for the following data:

Average permeability of the aquifer	= 40 m/day
Average thickness of the aquifer	= 15 m
Piezometric gradient	= 5 m/km

4.9 A 20 cm well completely penetrates an artesian aquifer. The length of the strainer is 16 m. What is the well yield for a drawdown of 3 m when the coefficient of permeability and the radius of influence are 30 m/day and 300 m, respectively.

4.10 A well with a radius of 0.5 m completely penetrates an unconfined aquifer with K = 32 m/day and the height of water table above the bottom of the aquifer being 45 m. The well is pumped so that the water level in the well remains 35 m above the bottom of the well. Assuming that pumping has essentially no effect on the water table at a distance of 300 m from the well, determine the steady-state well discharge.

4.11 A fully penetrating artesian well is pumped at a rate of 1500 m^3/day from an aquifer whose S and T values are 4×10^{-4} and 0.145 m^2/min, respectively. Find the drawdowns at a distance 3 m from the production well after one hour of pumping, and at a distance of 350 m after one day of pumping.

4.12 The values of drawdowns were observed in an observation well located at a distance of 3 m from a fully penetrating artesian well pumping at 2.2 litres per second. The drawdowns were 0.75 and 0.95 m after two and four hours of pumping respectively. Determine the aquifer constants S and T.

4.13 Two tubewells of 200 mm diameter are spaced at 120 m distance and penetrate fully a confined aquifer of 12 m thickness. What will be the percentage decrease in the discharge of each of these wells as a result of pumping both wells simultaneously with a depression head (*i.e.* drawdown) of 3 m in either case. Assume permeability of the aquifer as 40 m/day and the radius of influence as 200 m.

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5

CANAL IRRIGATION

5.1. CANALS

A conveyance subsystem for irrigation includes open channels through earth or rock formation, flumes constructed in partially excavated sections or above ground, pipe lines installed either below or above the ground surface, and tunnels drilled through high topographic obstructions. Irrigation conduits of a typical gravity project are usually open channels through earth or rock formations. These are called canals.

A *canal* is defined as an artificial channel constructed on the ground to carry water from a river or another canal or a reservoir to the fields. Usually, canals have a trapezoidal cross-section. Canals can be classified in many ways.

Based on the nature of source of supply, a canal can be either a permanent or an inundation canal. A *permanent canal* has a continuous source of water supply. Such canals are also called perennial canals. An *inundation canal* draws its supplies from a river only during the high stages of the river. Such canals do not have any headworks for diversion of river water to the canal, but are provided with a canal head regulator.

Depending on their function, canals can also be classified as: (*i*) irrigation, (*ii*) navigation, (*iii*) power, and (*iv*) feeder canals. An *irrigation canal* carries water from its source to agricultural fields. Canals used for transport of goods are known as *navigation canals*. *Power canals* are used to carry water for generation of hydroelectricity. A *feeder canal* feeds two or more canals.

A canal can serve more than one function. The slope of an irrigation canal is generally less than the ground slope in the head reaches of the canal and, hence, vertical falls have often to be constructed. Power houses may be constructed at these falls to generate power and, thus, irrigation canals can be used for power generation also.

Similarly, irrigation canals can also be utilised for the transportation of goods and serve as navigation canals. Inland navigation forms a cheap means of transportation of goods and, hence, must be developed. However, in India, inland navigation has developed only to a limited extent. This is mainly due to the fact that irrigation canals generally take their supplies from alluvial rivers and, as such, must flow with sufficient velocity to prevent siltation of the canal. Such velocities make upstream navigation very difficult. Besides, the canals are generally aligned on the watershed[1] so that water may reach the fields on both sides by flow. This alignment may not be suitable for navigation which requires the canal to pass through the areas in the vicinity of industries.

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[1] **Watershed is the dividing line between the catchment areas of two drains (see Sec. 5.3.2.)**

An irrigation canal system consists of canals of different sizes and capacities (Fig. 5.1). Accordingly, the canals are also classified as: (*i*) main canal, (*ii*) branch canal, (*iii*) major distributary, (*iv*) minor distributary, and (*v*) watercourse.

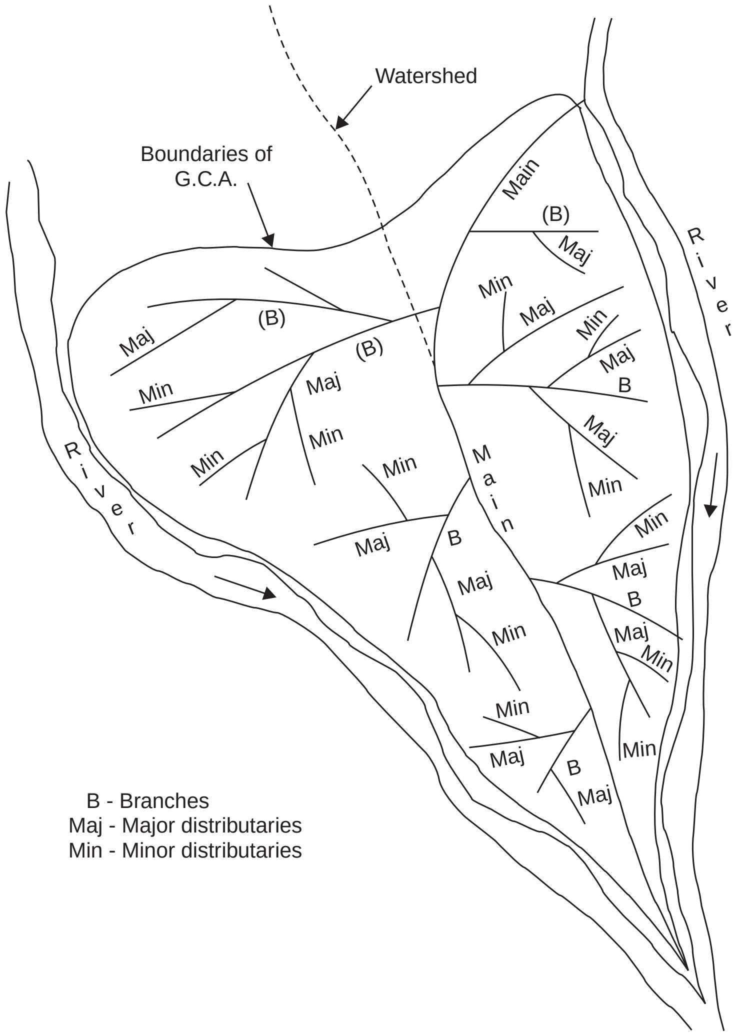

Fig. 5.1 Layout of an irrigation canal network

The *main canal* takes its supplies directly from the river through the head regulator and acts as a feeder canal supplying water to branch canals and major distributaries. Usually, direct irrigation is not carried out from the main canal.

Branch canals (also called 'branches') take their supplies from the main canal. Branch canals generally carry a discharge higher than 5 m^3/s and act as feeder canals for major and minor distributaries. Large branches are rarely used for direct irrigation. However, outlets are provided on smaller branches for direct irrigation.

Major distributaries (also called 'distributaries' or *rajbaha*) carry 0.25 to 5 m^3/s of discharge. These distributaries take their supplies generally from the branch canal and sometimes from the main canal. The distributaries feed either watercourses through outlets or minor distributaries.

Minor distributaries (also called 'minors') are small canals which carry a discharge less than 0.25 m^3/s and feed the watercourses for irrigation. They generally take their supplies from major distributaries or branch canals and rarely from the main canals.

A *watercourse* is a small channel which takes its supplies from an irrigation channel (generally distributaries) through an outlet and carries water to the various parts of the area to be irrigated through the outlet.

5.2. COMMAND AREAS

Gross command area (or GCA) is the total area which can be economically irrigated from an irrigation system without considering the limitation on the quantity of available water. It includes the area which is, otherwise, uncultivable. For example, ponds and residential areas are uncultivable areas of gross command area. An irrigation canal system lies in a *doab* (*i.e.,* the area between two drainages), and can economically irrigate the *doab*. It is, obviously, uneconomical to use the irrigation system to irrigate across the two drainages. Thus, the boundaries of the gross command of an irrigation canal system is fixed by the drainages on either side of the irrigation canal system.

The area of the cultivable land in the gross command of an irrigation system is called *culturable command area* (CCA) and includes all land of the gross command on which cultivation is possible. At any given time, however, all the cultivable land may not be actually under cultivation. Therefore, sometimes the CCA is divided into two categories: cultivated CCA and cultivable but not cultivated CCA.

Intensity of irrigation is defined as the percentage of CCA which is proposed to be annually irrigated. Till recently, no irrigation system was designed to irrigate all of its culturable command every year. This practice reduces the harmful effects of over-irrigation such as waterlogging and malaria. Also, due to the limitations on the quantity of available water, it is preferred to provide protection against famine in large areas rather than to provide intensive irrigation of a smaller area. The intensity of irrigation varied between 40 per cent to 60 per cent till recently. This needs to be raised to the range of 100 per cent to 180 per cent by cultivating parts of CCA for more than one crop in a year and through improved management of the existing system. Future projects should be planned for annual intensities of 100 per cent to 180 per cent depending on the availability of total water resources and land characteristics.

The culturable command area multiplied by the intensity of irrigation (in fraction) gives the actual area to be irrigated. The water requirements of the controlling crops of two crop seasons may be quite different. As such, the area to be irrigated should be calculated for each crop season separately to determine the water requirements.

5.3. PLANNING OF AN IRRIGATION CANAL SYSTEM

Planning of an irrigation canal project includes the determination of: (*i*) canal alignment, and (*ii*) the water demand. The first step in the planning of an irrigation canal project is to carry out a preliminary survey to establish the feasibility or otherwise of a proposal. Once the feasibility of the proposal has been established, a detailed survey of the area is carried out and, thereafter, the alignment of the canal is fixed. The water demand of the canal is, then, worked out.

5.3.1. Preliminary Survey

To determine the feasibility of a proposal of extending canal irrigation to a new area, information on all such factors which influence irrigation development is collected during the preliminary (or reconnaissance) survey. During this survey all these factors are observed or enquired from the local people. Whenever necessary, some quick measurements are also made.

The information on the following features of the area are to be collected:

(*i*) Type of soil,

(*ii*) Topography of the area,

(*iii*) Crops of the area,

(*iv*) Rainfall in the area,

(*v*) Water table elevations in the area,

(*vi*) Existing irrigation facilities, and

(*vii*) General outlook of the cultivators with respect to cultivation and irrigation.

The type of soil is judged by visual observations and by making enquiries from the local people. The influence of the soil properties on the fertility and waterholding capacity has already been discussed in Chapter 3.

For a good layout of the canal system, the command area should be free from too many undulations. This requirement arises from the fact that a canal system is essentially a gravity flow system. However, the land must have sufficient longitudinal and cross slopes for the channels to be silt-free. During the preliminary survey, the topography of the area is judged by visual inspection only.

Water demand after the completion of an irrigation project would depend upon the crops being grown in the area. The cropping pattern would certainly change due to the introduction of irrigation, and the possible cropping patterns should be discussed with the farmers of the area.

The existing records of rain gauge stations of the area would enable the estimation of the normal rainfall in the area as well as the probability of less than normal rainfall in the area. This information is, obviously, useful in determining the desirability of an irrigation project in the area.

Water table elevation can be determined by measuring the depth of water surface in a well from the ground with the help of a measuring tape. Water table elevation fluctuates considerably and information on this should be collected from the residents of the area and checked by measurements. Higher water table elevations in an area generally indicate good rainfall in the area as well as good soil moisture condition. Under such conditions, the demand for irrigation would be less and introduction of canal irrigation may cause the water table to rise up to the root zone of the crops. The land is then said to be waterlogged and the productivity of such land reduces considerably. Waterlogged land increases the incidence of malaria in the affected area. Thus, areas with higher water table elevation are not suitable for canal irrigation.

Because of limited financial and hydrological resources, an irrigation project should be considered only for such areas where maximum need arises. Areas with an extensive network of ponds and well systems for irrigation should be given low priority for the introduction of canal irrigation.

The success or failure of an otherwise good irrigation system would depend upon the attitudes of the farmers of the area. Enlightened and hard-working cultivators would quickly adapt themselves to irrigated cultivation to derive maximum benefits by making use of improved varieties of seeds and cultivation practices. On the other hand, conservative farmers will have to be educated so that they can appreciate and adopt new irrigated cultivation practices.

The information collected during preliminary survey should be carefully examined to determine the feasibility or otherwise of introducing canal irrigation system in the area. If the result of the preliminary survey is favourable, more detailed surveys would be carried out and additional data collected.

5.3.2. Detailed Survey

The preparation of plans for a large canal project is simplified in a developed area because of the availability of settlement maps (also called *shajra* maps having scale of 16 inches to a mile *i.e.,* $1/3960 \cong 1/4000$) and revenue records in respect of each of the villages of the area. The settlement maps show the boundaries and assigned numbers of all the fields of the area, location of residential areas, culturable and barren land, wells, ponds, and other features of the area. Usually for every village there is one settlement (or *shajra*) map which is prepared on a piece of cloth. These maps and the revenue records together give information on total land area, cultivated area, crop-wise cultivated area and the area irrigated by the existing ponds and wells.

With the help of settlement maps of all the villages in a *doab*, a drawing indicating distinguishing features, such as courses of well-defined drainages of the area, is prepared. On this drawing are then marked the contours and other topographical details not available on the settlement maps but required for the planning of a canal irrigation project. Contours are marked after carrying out '*levelling*' survey of the area.

The details obtained from the settlement maps should also be updated in respect of developments such as new roads, additional cultivated area due to dried-up ponds, and so on. In an undeveloped (or unsettled) area, however, the settlement maps may not be available and the plans for the canal irrigation project will be prepared by carrying out engineering survey of the area.

One of the most important details from the point of view of canal irrigation is the watershed which must be marked on the above drawing. *Watershed* is the dividing line between the catchment areas of two drains and is obtained by joining the points of highest elevation on successive cross-sections taken between any two streams or drains. Just as there would be the main watershed between two major streams of an area, there would be subsidiary watersheds between any tributary and the main stream or between any two adjacent tributaries.

5.4. ALIGNMENT OF IRRIGATION CANALS

Desirable locations for irrigation canals on any gravity project, their cross-sectional designs and construction costs are governed mainly by topographic and geologic conditions along different routes of the cultivable lands. Main canals must convey water to the higher elevations of the cultivable area. Branch canals and distributaries convey water to different parts of the irrigable areas.

On projects where land slopes are relatively flat and uniform, it is advantageous to align channels on the watershed of the areas to be irrigated. The natural limits of command of such irrigation channels would be the drainages on either side of the channel. Aligning a canal (main, branch as well as distributary) on the watershed ensures gravity irrigation on both sides of the canal. Besides, the drainage flows away from the watershed and, hence, no drainage can cross a canal aligned on the watershed. Thus, a canal aligned on the watershed saves the cost of construction of cross-drainage structures. However, the main canal has to be taken off from a river which is the lowest point in the cross-section, and this canal must mount the watershed in as short a distance as possible. Ground slope in the head reaches of a canal is much higher than the required canal bed slope and, hence, the canal needs only a short distance to mount the watershed. This can be illustrated by Fig. 5.2 in which the main canal takes off from a river at *P* and mounts the watershed at *Q*. Let the canal bed level at *P* be 400 m and the elevation of the highest point *N* along the section MNP be 410 m. Assuming that the ground slope is 1 m per km, the distance of the point *Q* (395 m) on the watershed from *N* would be 15 km. If the required canal bed slope is 25 cm per km, the length *PQ* of the canal would be 20 km. Between *P* and *Q*, the canal would cross small streams and, hence, construction of cross-drainage structures would be necessary for this length. In fact, the alignment *PQ* is influenced considerably by the need of providing suitable locations for the cross-drainage structures. The exact location of *Q* would be determined by trial so that the alignment *PQ* results in an economic as well as efficient system. Further, on the watershed side of the canal *PQ*, the ground is higher than the ground on the valley side (*i.e.*, the river side). Therefore, this part of the canal can irrigate only on one side (*i.e.*, the river side) of the canal.

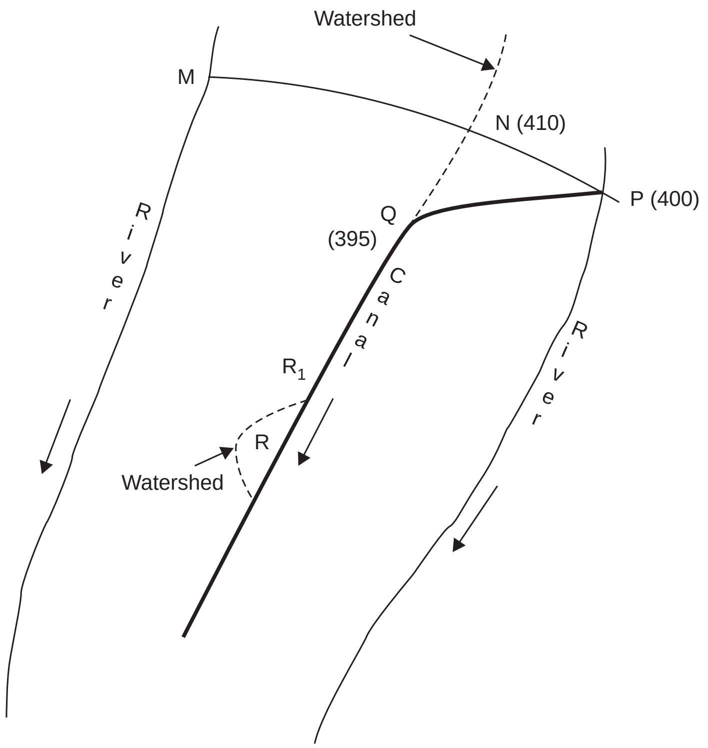

Fig. 5.2 Head reach of a main canal in plains

Once a canal has reached the watershed, it is generally kept on the watershed, except in certain situations, such as the looping watershed at R in Fig. 5.2. In an effort to keep the canal alignment straight, the canal may have to leave the watershed near R. The area between the canal and the watershed in the region R can be irrigated by a distributary which takes off at R_1 and follows the watershed. Also, in the region R, the canal may cross some small streams and, hence, some cross-drainage structures may have to be constructed. If watershed is passing through villages or towns, the canal may have to leave the watershed for some distance.

In hilly areas, the conditions are vastly different compared to those of plains. Rivers flow in valleys well below the watershed or ridge, and it may not be economically feasible to take the channel on the watershed. In such situations, contour channels (Fig. 5.3) are constructed. Contour channels follow a contour while maintaining the required longitudinal slope. It continues like this and as river slopes are much steeper than the required canal bed slope the canal encompasses more and more area between itself and the river. It should be noted that the more fertile areas in the hills are located at lower levels only.

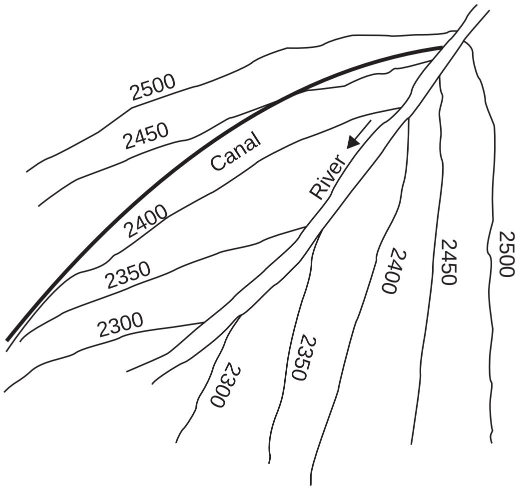

Fig. 5.3 Alignment of main canal in hills

In order to finalise the channel network for a canal irrigation project, trial alignments of channels are marked on the map prepared during the detailed survey. A large-scale map is required to work out the details of individual channels. However, a small-scale map depicting the entire command of the irrigation project is also desirable. The alignments marked on the map are transferred on the field and adjusted wherever necessary. These adjustments are transferred on the map as well. The alignment on the field is marked by small masonry pillars at every 200 metres. The centre line on top of these pillars coincides with the exact alignment. In between the adjacent pillars, a small trench, excavated in the ground, marks the alignment.

5.5. CURVES IN CANALS

Because of economic and other considerations, the canal alignment does not remain straight all through the length of the canal, and curves or bends have to be provided. The curves cause disturbed flow conditions resulting in eddies or cross currents which increase the losses. In a curved channel portion, the water surface is not level in the transverse direction. There is a slight drop in the water surface at the inner edge of the curve and a slight rise at the outer edge of the curve. This results in slight increase in the velocity at the inner edge and slight decrease in the velocity at the outer edge. As a result of this, the low-velocity fluid particles near the bed move to the inner bank and the high-velocity fluid particles near the surface gradually cross to the outer bank. The cross currents tend to cause erosion along the outer bank. The changes in the velocity on account of cross currents depend on the approach flow condition and the characteristics of the curve. When separate curves follow in close succession, either in the same direction or in the reversed direction, the velocity changes become still more complicated.

Therefore, wherever possible, curves in channels excavated through loose soil should be avoided. If it is unavoidable, the curves should have a long radius of curvature. The permissible minimum radius of curvature for a channel curve depends on the type of channel, dimensions of cross-section, velocities during full-capacity operations, earth formation along channel alignment and dangers of erosion along the paths of curved channel. In general, the permissible minimum radius of curvature is shorter for flumes or lined canals than earth canals, shorter for small cross-sections than for large cross-sections, shorter for low velocities than for high velocities, and shorter for tight soils than for loose soils. Table 5.1 indicates the values of minimum radii of channel curves for different channel capacities.

Table 5.1 Radius of curvature for channel curves (1)

Channel capacity (m^3/s)	*Minimum radius of curvature (metres)*
Less than 0.3	100
0.3 to 3.0	150
0.3 to 15.0	300
15.0 to 30.0	600
30.0 to 85.0	900
More than 85	1500

5.6. DUTY OF WATER

For proper planning of a canal system, the designer has to first decide the 'duty of water' in the locality under consideration. Duty is defined as the area irrigated by a unit discharge of water flowing continuously for the duration of the base period of a crop. The *base period* of a crop is the time duration between the first watering at the time of sowing and the last watering before harvesting the crop. Obviously, the base period of a crop is smaller than the crop period. Duty is measured in hectares/m^3/s. The duty of a canal depends on the crop, type of soil, irrigation and cultivation methods, climatic factors, and the channel conditions.

By comparing the duty of a system with that of another system or by comparing it with the corresponding figures of the past on the same system, one can have an idea about the performance of the system. Larger areas can be irrigated if the duty of the irrigation system is improved. Duty can be improved by the following measures:

(*i*) The channel should not be in sandy soil and be as near the area to be irrigated as possible so that the seepage losses are minimum. Wherever justified, the channel may be lined.

(*ii*) The channel should run with full supply discharge as per the scheduled program so that farmers can draw the required amount of water in shorter duration and avoid the tendency of unnecessary over irrigation.

(*iii*) Proper maintenance of watercourses and outlet pipes will also help reduce losses, and thereby improve the duty.

(*iv*) Volumetric assessment of water makes the farmer to use water economically. This is, however, more feasible in well irrigation.

Well irrigation has higher duty than canal irrigation due to the fact that water is used economically according to the needs. Open wells do not supply a fixed discharge and, hence, the average area irrigated from an open well is termed its duty.

Between the head of the main canal and the outlet in the distributary, there are losses due to evaporation and percolation. As such, duty is different at different points of the canal system. The duty at the head of a canal system is less than that at an outlet or in the tail end region of the canal. Duty is usually calculated for the head discharge of the canal. Duty calculated on the basis of outlet discharge is called '*outlet discharge factor*' or simply '*outlet factor*' which excludes all losses in the canal system.

5.7. CANAL LOSSES

When water comes in contact with an earthen surface, whether artificial or natural, the surface absorbs water. This absorbed water percolates deep into the ground and is the main cause of the loss of water carried by a canal. In addition, some canal water is also lost due to evaporation. The loss due to evaporation is about 10 per cent of the quantity lost due to seepage. The seepage loss varies with the type of the material through which the canal runs. Obviously, the loss is greater in coarse sand and gravel, less in loam, and still less in clay soil. If the canal carries silt-laden water, the pores of the soil are sealed in course of time and the canal seepage reduces with time. In almost all cases, the seepage loss constitutes an important factor which must be accounted for in determining the water requirements of a canal.

Between the headworks of a canal and the watercourses, the loss of water on account of seepage and evaporation is considerable. This loss may be of the order of 20 to 50 per cent of water diverted at the headworks depending upon the type of soil through which canal runs and the climatic conditions of the region.

For the purpose of estimating the water requirements of a canal, the total loss due to evaporation and seepage, also known as conveyance loss, is expressed as m^3/s per million square metres of either wetted perimeter or the exposed water surface area. Conveyance loss can be calculated using the values given in Table 5.2. In UP, the total loss (due to seepage and evaporation) per million square metres of water surface varies from 2.5 m^3/s for ordinary clay loam to 5.0 m^3/s for sandy loam. The following empirical relation has also been found to give comparable results (2).

$$q_l = (1/200)\,(B + h)^{2/3} \tag{5.1}$$

Table 5.2 Conveyance losses in canals (1)

Material	*Loss in m^3/s per million square metres of wetted perimeter (or water surface)*
Impervious clay loam	0.88 to 1.24
Medium clay loam underlaid with hard pan at depth of not over 0.60 to 0.90 m below bed	1.24 to 1.76
Ordinary clay loam, silty soil or lava ash loam	1.76 to 2.65
Gravelly or sandy clay loam, cemented gravel, sand and clay	2.65 to 3.53
Sandy loam	3.53 to 5.29
Loose sand	5.29 to 6.17
Gravel sand	7.06 to 8.82
Porous gravel soil	8.82 to 10.58
Gravels	10.58 to 21.17

In this relation, q_l is the loss expressed in m^3/s per kilometre length of canal and B and h are, respectively, canal bed width and depth of flow in metres.

5.8. ESTIMATION OF DESIGN DISCHARGE OF A CANAL

The amount of water needed for the growth of a crop during its entire crop-growing period is known as the water requirement of the crop, and is measured in terms of depth of water spread over the irrigated area. This requirement varies at different stages of the growth of the plant. The peak requirement must be obtained for the period of the keenest demand. One of the methods to decide the water requirement is on the basis of *kor* watering.

When the plant is only a few centimetres high, it must be given its first watering, called the *kor* watering, in a limited period of time which is known as the *kor* period. If the plants do not receive water during the *kor* period, their growth is retarded and the crop yield reduces considerably. The *kor* watering depth and the *kor* period vary depending upon the crop and the climatic factors of the region. In UP, the *kor* watering depth for wheat is 13.5 cm and the *kor* period varies from 8 weeks in north-east UP (a relatively dry region) to 3 weeks in the hilly region (which is relatively humid). For rice, the *kor* watering depth is 19 cm and the *kor* period varies from 2 to 3 weeks.

If D represents the duty (measured in hectares/m^3/s) then, by definition,

1 m^3/s of water flowing for b (*i.e.*, base period in days) days irrigates D hectares.

$\therefore$ 1 m^3/s of water flowing for 1 day (*i.e.*, 86400 m^3 of water) irrigates D/b hectares

This volume (*i.e.*, 86400 m^3) of water spread over D/b hectares gives the water depth, Δ.

$$\therefore \quad \Delta = \frac{86400}{(D/b)\times 10^4} = 8.64\, b/D \text{ (metres)} \tag{5.2}$$

For the purpose of designing on the basis of the keenest demand (*i.e.*, the *kor* period requirement) the base period b and the water depth Δ are replaced by the *kor* period and *kor* water depth, respectively.

Example 5.1 The culturable command area for a distributary channel is 10,000 hectares. The intensity of irrigation is 30 per cent for wheat and 15 per cent for rice. The *kor* period for wheat is 4 weeks, and for rice 3 weeks. *Kor* watering depths for wheat and rice are 135 mm and 190 mm, respectively. Estimate the outlet discharge.

Solution:

Quantity	*Wheat*	*Rice*
Area to be irrigated (hectares)	$0.30 \times 10{,}000 = 3000$	$0.15 \times 10{,}000 = 1500$
Outer factor $D = 8.64\ b/\Delta$ (in hectares/m^3/s)	$\dfrac{8.64(4 \times 7)}{0.135} = 1792$	$\dfrac{8.64(3 \times 7)}{0.19} = 954.95$
Outlet discharge (m^3/s)	$3000/1792 = 1.674 \approx 1.7$	$1500/954.95 = 1.571 \approx 1.6$

Since the water demands for wheat and rice are at different times, these are not cumulative. Therefore, the distributary channel should be designed for the larger of the two discharges, *viz.,* 1.7 m^3/s. The above calculations exclude channel losses and the water requirement of other major crops during their *kor* period.

The *kor* period for a given crop in a region depends on the duration during which there is likelihood of the rainfall being smaller than the corresponding water requirement. Accordingly, the *kor* period is least in humid regions and more in dryer regions. The *kor* depth requirement must be met within the *kor* period. As such, the channel capacity designed on the basis of *kor* period would be large in humid regions and small in dry regions. Obviously, this method of determining the channel capacity is, therefore, not rational, and is not used in practice.

A more rational method to determine the channel capacity would be to compare evapotranspiration and corresponding effective rainfall for, say, 10-day (or 15-day) periods of the entire year and determine the water requirement for each of these periods. The channel capacity can then be determined on the basis of the peak water requirement of the 10-day (or 15-day) periods. This method has already been explained in Sec. 3.8.

5.9. CANAL OUTLETS

When the canal water has reached near the fields to be irrigated, it has to be transferred to the watercourses. At the junction of the watercourse and the distributary, an outlet is provided. An *outlet* is a masonry structure through which water is admitted from the distributary into a watercourse. It also acts as a discharge measuring device. The discharge though an outlet is usually less than 0.085 m^3/s (3). It plays a vital role in the *warabandi* system (see Sec. 5.11) of distributing water. Thus, an outlet is like a head regulator for the watercourse.

The main objective of providing an outlet is to provide ample supply of water to the fields, whenever needed. If the total available supply is insufficient, the outlets must be such that equitable distribution can be ensured. The efficiency of an irrigation system depends on the proper functioning of canal outlets which should satisfy the following requirements (3):

(*i*) The outlets must be strong and simple with no moving parts which would require periodic attention and maintenance.

(*ii*) The outlets should be tamper-proof and if there is any interference in the functioning of the outlet, it should be easily detectable.

(*iii*) The cost of outlets should be less as a large number of these have to be installed in an irrigation network.

(*iv*) The outlet should be able to draw sediment in proportion to the amount of water withdrawn so that there is no silting or scouring problem in the distributary downstream of the outlet.

(*v*) The outlets should be able to function efficiently even at low heads.

The choice of type of an outlet and its design are governed by factors such as water distribution policy, water distribution method, method of water assessment, sources of supply, and the working of the distributary channel.

Water may be distributed on the basis of either the actual area irrigated in the previous year or the actual culturable command area. The discharge from the outlet should be capable of being varied in the first case, but, can remain fixed in the second. The method of water distribution may be such that each cultivator successively receives water for a duration in proportion to his area. Or, alternatively, all the cultivators share the outlet discharge simultaneously. The first system is better as it results in less loss of water. The outlet capacity is decided keeping in view the method of water distribution.

If the assessment is by volume, the outlet discharge should remain constant and not change with variation in the water levels of the distributary and the watercourse. On the other hand, if water charges are decided on the basis of area, the variation in the outlet capacity with water levels of the distributary and watercourse is relatively immaterial.

With a reservoir as supply source, the cultivators can be provided water whenever needed and, hence, the outlets should be capable of being opened or closed. The outlets generally remain open if the supply source is a canal without storage so that water is diverted to the field when the canal is running.

At times, the amount of water in the main canal may not be sufficient to feed all the channels simultaneously to their full capacity. As such, either all the channels may run with low discharge or groups of channels may be supplied their full capacity by rotation. In the first case, the outlets must be able to take their proportionate share even with large variations in the discharge of the distributary channel. In the second case, the outlets must be such that the required amount of water is available for all the channels being fed with their full capacity.

It should be noted that whereas the cultivator prefers to have outlets capable of supplying constant discharge, the canal management would prefer that the outlets supply variable discharge depending upon the discharge in the distributary channel so that the tail end of the channel is neither flooded nor dried. Obviously, both these requirements cannot be fulfilled simultaneously.

5.9.1. Types of Outlet

Canal outlets are of the following three types:

(*i*) Non-modular outlets,

(*ii*) Semi-modular outlets, and

(*iii*) Modular outlets.

Non-modular outlets are those whose discharge capacity depends on the difference of water levels in the distributary and the watercourse. The discharge through non-modular outlets fluctuates over a wide range with variations in the water levels of either the distributary or the watercourse. Such an outlet is controlled by a shutter at its upstream end. The loss of head in a non-modular outlet is less than that in a modular outlet. Hence, non-modular outlets are very suitable for low head conditions. However, in these outlets, the discharge may vary even when the water level in the distributary remains constant. Hence, it is very difficult to ensure equitable distribution of water at all outlets at times of keen demand of water.

The discharge through a *semi-modular outlet* (or semi-module or flexible outlet) depends only on the water level in the distributary and is unaffected by the water level in the watercourse provided that a minimum working head required for its working is available. A semi-module is more suitable for achieving equitable distribution of water at all outlets of a distributary. The only disadvantage of a semi-modular outlet is that it involves comparatively greater loss of head.

Modular outlets are those whose discharge is independent of the water levels in the distributary and watercourse, within reasonable working limits. These outlets may or may not have moving parts. In the latter case, these are called rigid modules. Modular outlets with moving parts are not simple to design and construct and are, hence, expensive.

A modular outlet supplies fixed discharge and, therefore, enables the farmer to plan his irrigation accordingly. However, in case of excess or deficient supplies in the distributary, the tail-end reach of the distributary may either get flooded or be deprived of water. This is due to the reason that the modular outlet would not adjust its discharge corresponding to the water level in the distributary. But, if an outlet is to be provided in a branch canal which is likely to run with large fluctuations in discharge, a modular outlet would be an ideal choice. The outlet would be set at a level low enough to permit it to draw its due share when the branch is running with low supplies. When the branch has to carry excess supplies to meet the demands of the distributaries, the discharge through the modular outlet would not be affected and the excess supplies would reach up to the desired distributaries. Similarly, if an outlet is desired to be located upstream of a regulator or a raised crest fall, a modular outlet would be a suitable choice.

5.9.2. Parameters for Studying the Behaviour of Outlets

5.9.2.1. Flexibility

The ratio of the rate of change of discharge of an outlet (dQ_0/Q_0) to the rate of change of discharge of the distributary channel (dQ/Q) (on account of change in water level) is termed the flexibility which is designated as F. Thus,

$$F = (dQ_0/Q_0)/(dQ/Q) \tag{5.3}$$

Here, Q and Q_0 are the flow rates in the distributary channel and the watercourse, respectively. Expressing discharge Q in the distributary channel in terms of depth of flow h in the channel as

$$Q = C_1 h^n$$

one can obtain

$$\frac{dQ}{Q} = n\frac{dh}{h}$$

Similarly, the discharge Q_0 through the outlet can be expressed in terms of the head H on the outlet as

$$Q_0 = C_2 H^m$$

which gives

$$\frac{dQ_0}{Q_0} = m\frac{dH}{H}$$

Here, m and n are suitable indices and C_1 and C_2 are constants. Thus,

$$F = \frac{m}{n} \times \frac{h}{H} \times \frac{dH}{dh} \tag{5.4}$$

For semi-modular outlets, the change in the head dH at an outlet would be equal to the change in the depth of flow dh in the distributary. Therefore,

$$F = \frac{m}{n} \times \frac{h}{H} \tag{5.5}$$

If the value of F is unity, the rate of change of outlet discharge equals that of the distributary discharge. For a modular outlet, the flexibility is equal to zero. Depending upon the value of F, the outlets can be classified as: (*i*) proportional outlets (F = 1), (*ii*) hyper-proportional outlets ($F > 1$), and (*iii*) subproportional outlets ($F < 1$). When a certain change in the distributary discharge causes a proportionate change in the outlet discharge, the outlet (or semi-module) is said to be proportional. A proportional semi-module ensures proportionate distribution of water when the distributary discharge cannot be kept constant. For a proportional semi-modular oulet (F = 1),

$$\frac{H}{h} = \frac{m}{n} \tag{5.6}$$

The ratio (H/h) is a measure of the location of the outlet and is termed *setting*. Every semi-module can work as a proportional semi-module if its sill is fixed at a particular level with respect to the bed level of the distributary. A semi-module set to behave as a proportional outlet may not remain proportional at all distributary discharges. Due to silting in the head reach of a distributary, the water level in the distributary would rise and the outlet located in the head reach would draw more discharge although the distributary discharge has not changed. Semi-modules of low flexibility are least affected by channel discharge and channel regime and should, therefore, be used whenever the modular outlet is unsuitable for given site conditions.

The setting for a proportional outlet is equal to the ratio of the outlet and the channel indices. For hyper-proportional and sub-proportional outlets the setting must be, respectively, less and more than m/n. For a wide trapezoidal (or rectangular) channel, n can be approximately taken as 5/3 and for an orifice type outlet, m can be taken as 1/2. Thus, an orifice-type module will be proportional if the setting (H/h) is equal to (1/2)/(5/3), *i.e.*, 0.3. The module will be hyper-proportional if the setting is less than 0.3 and sub-proportional if the setting is greater than 0.3. Similarly, a free flow weir type outlet (m = 3/2) would be proportional when the setting equals 0.9 which means that the outlet is fixed at 0.9 h below the water surface in the distributary.

5.9.2.2. Sensitivity

The ratio of the rate of change of discharge (dQ_0/Q_0) of an outlet to the rate of change in the water surface level of the distributary channel with respect to the depth of flow in the channel is called the '*sensitivity*' of the outlet. Thus,

$$S = \frac{(dQ_0/Q_0)}{(dG/h)} \tag{5.7}$$

Here, S is the sensitivity and G is the gauge reading of a gauge which is so set that $G = 0$ corresponds to the condition of no discharge through the outlet (*i.e.*, $Q_0 = 0$). Obviously, $dG = dh$. Thus, sensitivity can also be defined as the ratio of the rate of change of discharge of an outlet to the rate of change of depth of flow in the distributary channel. Therefore,

$$S = (dQ_0/Q_0)/(dh/h)$$

Also,

$$F = (dQ_0/Q_0)/(dQ/Q)$$

$$= (dQ_0/Q_0)/n\left(\frac{dh}{h}\right)$$

$$= \frac{1}{n}S$$

$\therefore$

$$S = nF \tag{5.8}$$

Thus, the sensitivity of an outlet for a wide trapezoidal (or rectangular) distributary channel ($n = 5/3$) is equal to $(5/3)F$. The sensitivity of a modular outlet is, obviously, zero.

The 'minimum modular head' is the minimum head required for the proper functioning of the outlet as per its design. The *modular limits* are the extreme values of any parameter (or quantity) beyond which an outlet is incapable of functioning according to its design. The *modular range* is the range (between modular limits) of values of a quantity within which the outlet works as per its design. The *efficiency* of any outlet is equal to the ratio of the head recovered (or the residual head after the losses in the outlet) to the input head of the water flowing through the outlet.

5.9.3. Non-Modular Outlets

The non-modular outlet is usually in the form of a submerged pipe outlet or a masonry sluice which is fixed in the canal bank at right angles to the direction of flow in the distributary. The diameter of the pipe varies from 10 to 30 cm. The pipe is laid on a light concrete foundation to avoid uneven settlement of the pipe and consequent leakage problems. The pipe inlet is generally kept about 25 cm below the water level in the distributary. When considerable fluctuation in the distributary water level is anticipated, the inlet is so fixed that it is below the minimum water level in the distributary. Figure 5.4 shows a pipe outlet. If H is the difference in water levels of the distributary and the watercourse then the discharge Q through the outlet can be obtained from the equation,

$$H = \frac{V^2}{2g}\left[0.5 + \frac{fL}{d} + 1\right] \tag{5.9}$$

or

$$H = \frac{V^2}{2g}\left[1.5 + f\frac{L}{d}\right] \tag{5.10}$$

where

$$V = \frac{Q}{(\pi/4)d^2} = \sqrt{2gH}\left(\frac{d}{1.5d + fL}\right)^{1/2} \tag{5.11}$$

d = diameter of pipe outlet

L = length of pipe outlet

and f = friction factor for pipe.

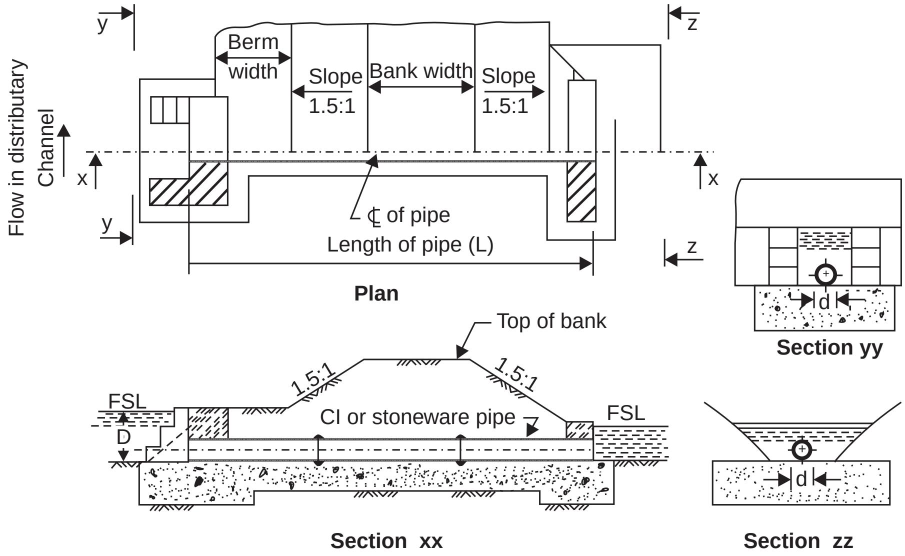

Fig. 5.4 Pipe outlet (3)

Alternatively, the discharge Q can be expressed as

$$Q = AV$$

$$= \left(\frac{\pi}{4} d^2\right) \sqrt{2gH} \left(\frac{d}{1.5d + fL}\right)^{1/2}$$

or $$Q = CA \sqrt{2gH} \tag{5.12}$$

in which $$C = \left(\frac{d}{1.5d + fL}\right)^{1/2}$$

Because of the disturbance at the entrance, the outlet generally carries its due share of sediment. In order to further increase the amount of sediment drawn by the outlet, the inlet end of the outlet is lowered. It is common practice to place the pipe at the bed of the distributary to enable the outlets to draw a fair share of sediment (3). The outlet pipe thus slopes upward. This arrangement increases the amount of sediment withdrawn by the outlet without affecting the discharge through the outlet.

Obviously, the discharge through non-modular outlets varies with water levels in the distributary and watercourse. In the case of fields located at high elevations, the watercourse level is high and, hence, the discharge is relatively small. But, for fields located at low elevations, the discharge is relatively large due to lower watercourse levels. Further, depending upon the amount of withdrawal of water in the head reaches, the tail reach may be completely dry or get flooded. Thus, discharge through pipe outlets can be increased by deepening the watercourse and thereby lowering the water level in it. The discharge varies from outlet to outlet because of flow conditions, and also at different times on the same outlet due to sediment discharge in the distributary channel. For these reasons, proper and equitable distribution of water is very difficult. These are the serious drawbacks of pipe outlets. The non-modular outlets can, however, work well for low heads too and this is their chief merit. Pipe outlets are adopted in the initial stages of distributions or for additional irrigation in a season when excess supply is available.

5.9.4. Semi-Modular Outlets (Semi-Modules or Flexible Outlets)

The simplest type of semi-modular outlet is a pipe outlet discharging freely into the atmosphere. The pipe outlet, described as the non-modular outlet, works as semi-module when it discharges freely into the watercourse. The exit end of the pipe is placed higher than the water level in the watercourse. In this case, the working head H is the difference between the water level in the distributary and the centre of the pipe outlet. The efficiency of the pipe outlet is high and its sediment conduction is also good. The discharge through the pipe outlet cannot be increased by the cultivator by digging the watercourse and thus lowering the water level of the watercourse. Usually, a pipe outlet is set so that it behaves as subproportional outlet, *i.e.*, its setting is kept less than 0.3. Other types of flexible outlets include Kennedy's gauge outlet, open flume outlet, and orifice semi-modules.

5.9.4.1. Kennedy's Gauge Outlet

This outlet was developed by RG Kennedy in 1906. It mainly consists of an orifice with bellmouth entry, a long expanding delivery pipe, and an intervening vertical air column above the throat (Fig. 5.5). The air vent pipe permits free circulation of air around the jet. This arrangement makes the discharge through the outlet independent of the water level in the watercourse. The water jet enters the cast iron expanding pipe which is about 3 m long and at the end of which a cement concrete pipe extension is generally provided. Water is then discharged into the watercourse. This outlet can be easily tampered with by the cultivator who blocks the air vent pipe to increase the discharge through the outlet. Because of this drawback and its high cost, Kennedy's gauge outlet is generally not used.

5.9.4.2. Open Flume Outlet

An open flume outlet is a weir with a sufficiently constricted throat to ensure supercritical flow and long enough to ensure that the controlling section remains within the throat at all discharges up to the maximum. A gradual expansion is provided downstream of the throat. The entire structure is built in brick masonry but the controlling section is generally provided with cast iron or steel bed and check plates. This arrangement ensures the formation of hydraulic jump and, hence, the outlet discharge remains independent of the water level in the watercourse. Figure 5.6 shows the type of open flume outlet commonly used in Punjab. The discharge through the outlet is proportional to $H^{3/2}$. The efficiency of the outlet varies between 80 and 90 per cent.

The throat width of the outlet should not be less than 60 mm as a narrower throat may easily get blocked by the floating material. For the range of outlet discharges normally used, the outlet is either deep and narrow, or shallow and wide. While a narrow outlet gets easily blocked, a shallow outlet is not able to draw its fair share of sediment.

5.9.4.3. Orifice Semi-Modules

An orifice semi-module consists of an orifice followed by a gradually expanding flume on the downstream side (Fig. 5.7). Supercritical flow through the orifice causes the formation of hydraulic jump in the expanding flume and, hence, the outlet discharge remains independent of the water level in the watercourse. The roof block is suitably shaped to ensure converging streamlines so that the discharge coefficient does not vary much. The roof block is fixed in its place by means of two bolts embedded in a masonry key. For adjustment, this masonry can be dismantled and the roof block is suitably adjusted. After this, the masonry key is rebuilt. Thus, the adjustment can be made at a small cost. Tampering with the outlet by the cultivators would be easily noticed through the damage to the masonry key. This is the chief merit of this outlet.

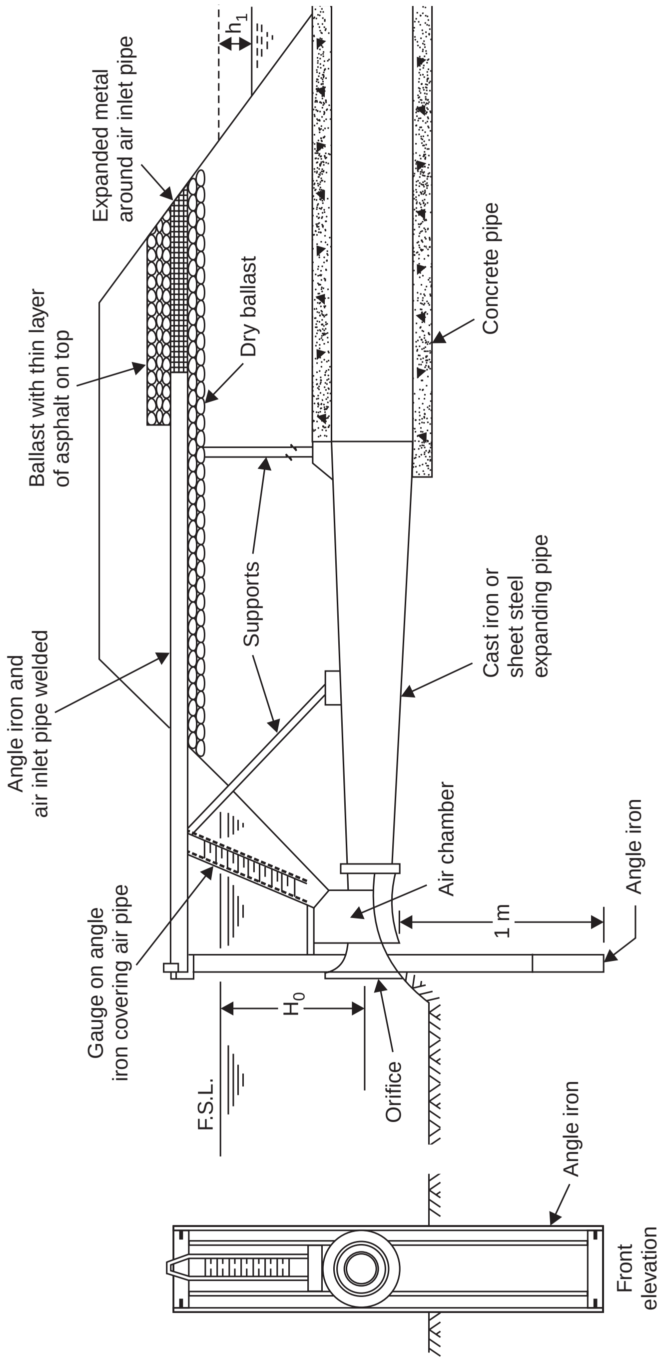

Fig. 5.5 Kennedy's gauge outlet

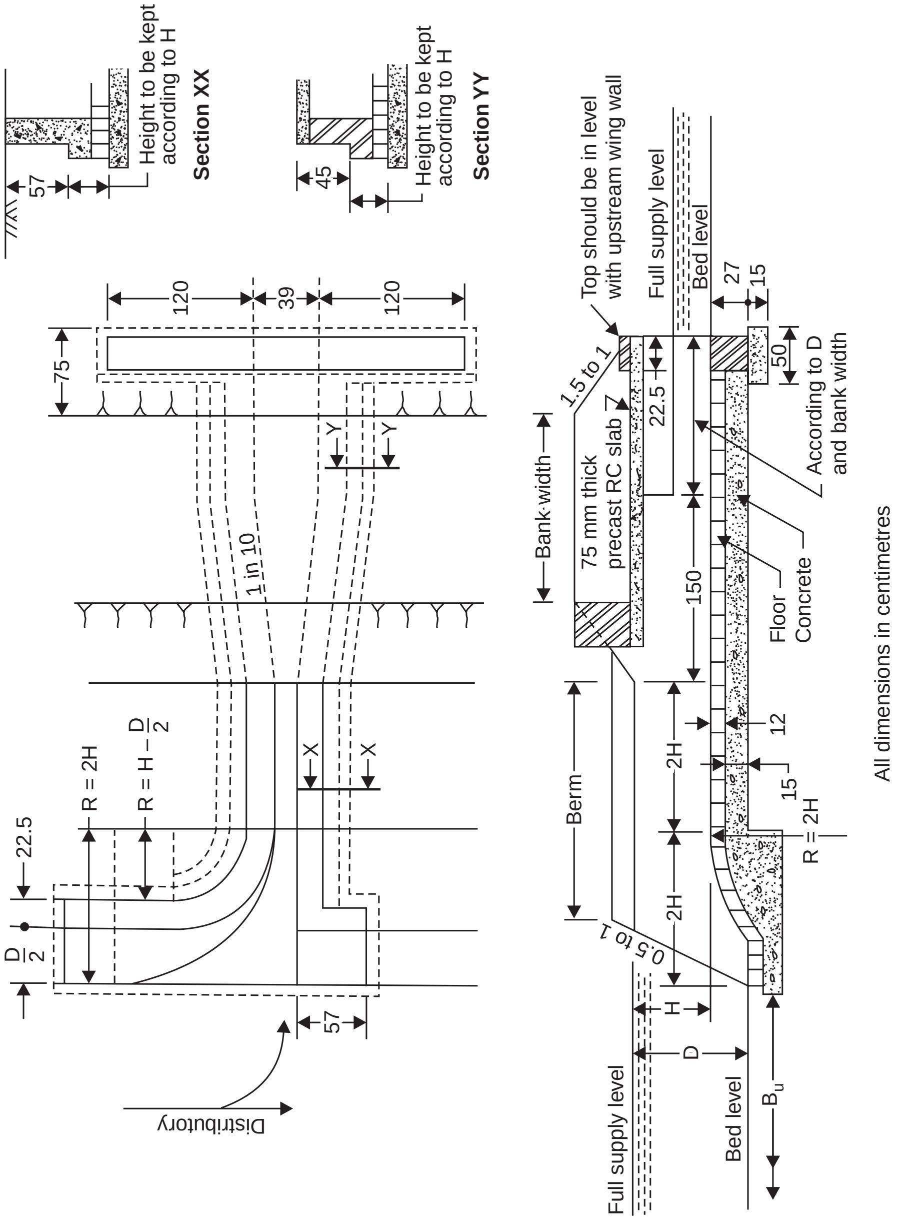

Fig. 5.6 Plan of open flume outlet for distributary above 0.6 m depth and H less than full supply depth (3)

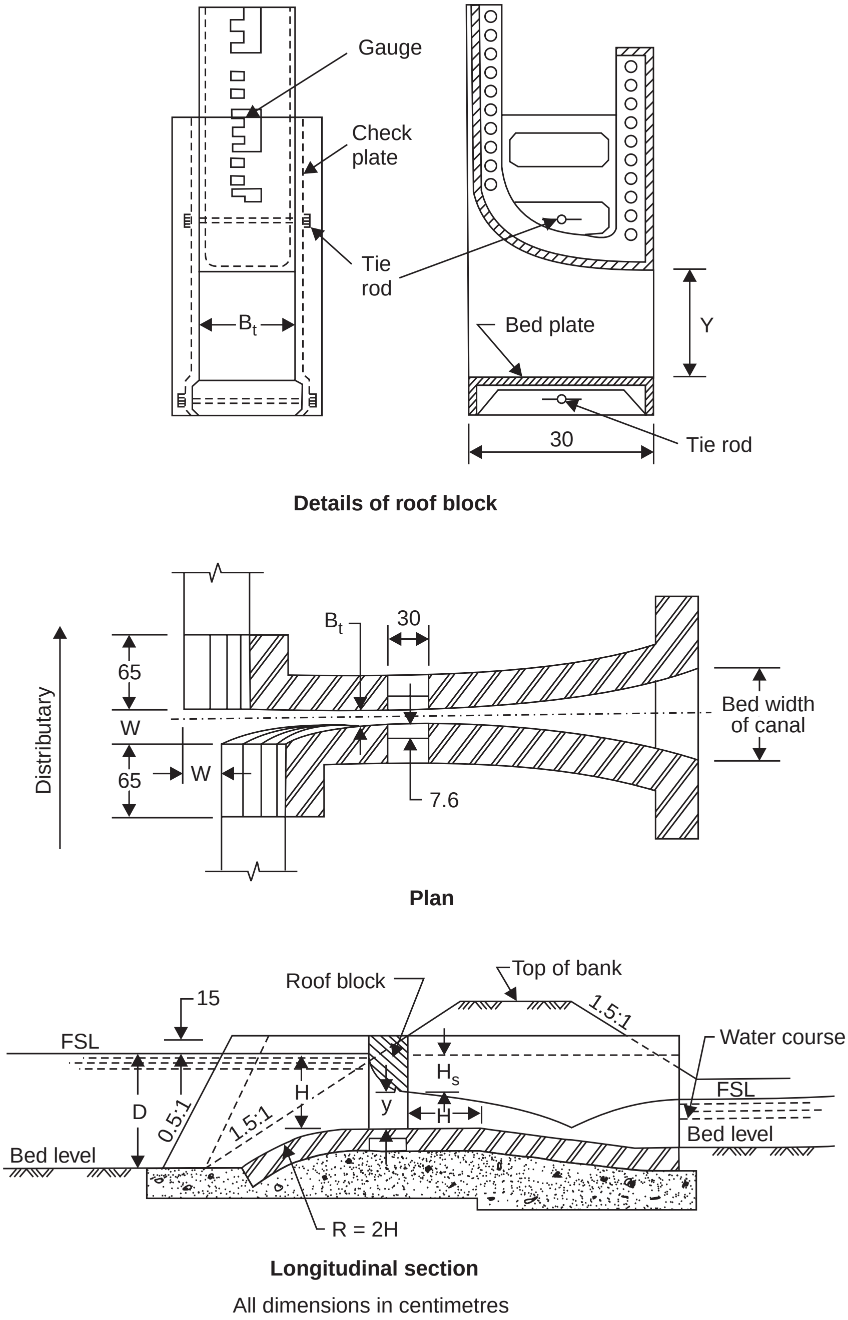

Fig. 5.7 Crump's adjustable proportional module (3)

The base plates and roof blocks are manufactured in standard sizes, such as B_t = 6.1, 7.6, 9.9, 12.2, 15.4, 19.5, 24.4, and 30.5 cm. B_t is the throat width. The base plates and roof blocks of these standard sizes with required opening of the orifice are used to obtain desired supply through the outlet.

The waterway in this type of outlet is either deep and narrow, that can easily get blocked, or shallow and wide in which case it does not draw its fair share of sediment. The discharge in this type of outlet is given by the formula (3):

$$Q = 4.03\, B_t\, Y \sqrt{H} \tag{5.13}$$

The ratio H_s/D should be between 0.375 and 0.48 for proportionate distribution of sediment and should be 0.8 or less for modular working (3). Here, H_s, D (= h), B_t, Y, and H are as shown in Fig. 5.7.

5.9.5. Modular Outlets

Most of the modular outlets have moving parts which make them costly to install as well as maintain. The following two types of modular outlets (also known as rigid modules), however, do not have any moving part:

(*i*) Gibb's rigid module, and

(*ii*) Khanna's rigid module.

5.9.5.1. Gibb's Rigid Module

This module has an inlet pipe under the distributary bank. This pipe takes water from distributary to a rising spiral pipe which joins the eddy chamber (Fig. 5.8). This arrangement results in free vortex motion. Due to this free vortex motion, there is heading up of water (owing to smaller velocity at larger radius–a characteristic of vortex motion) near the outer wall of the rising pipe. The water surface thus slopes towards the inner wall. A number of baffle plates of suitable size are suspended from the roof of the eddy chamber such that the lower ends of these plates slope against the flow direction. With the increase in head, the water banks up at the outer wall of the eddy chamber and impinges against the baffles and spins round in the compartment between two successive baffle plates. This causes dissipation of excess energy and results in constant discharge. The outlet is relatively more costly and its sediment withdrawal is also not good.

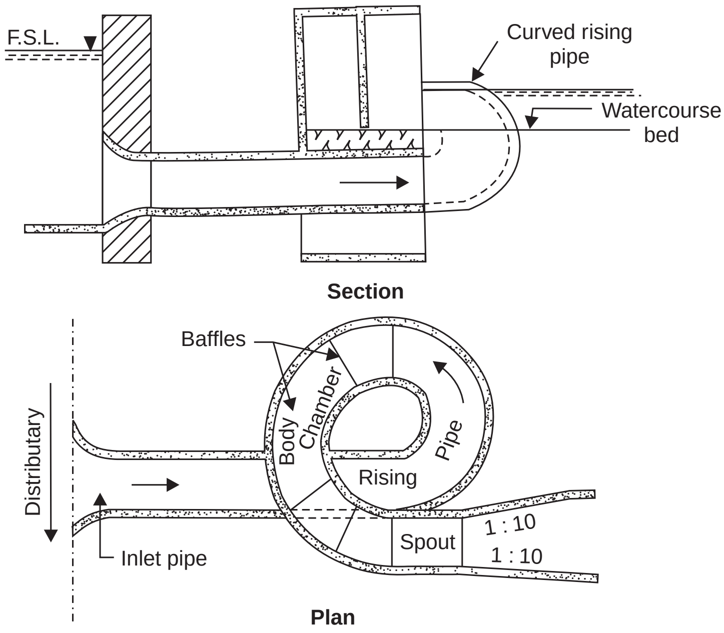

Fig. 5.8 Gibb's module

5.9.5.2. Khanna's Rigid Orifice Module

This outlet is similar to an orifice semi-module. But, in addition, it has sloping shoots fixed in the roof block (Fig. 5.9). These shoots cause back flow and thus keep the outlet discharge constant. If the water level in the distributary is at or below its normal level, the outlet behaves like an orifice semi-module. But, when the water level in the distributary channel is above its normal level, the water level rises in chamber A, and enters the first sloping shoot. This causes back flow and dissipates additional energy. This maintains a constant discharge. The number of sloping shoots and their height above the normal level can vary to suit local requirements. The shoots are housed in a chamber to prevent them from being tampered with. If the shoots are blocked, the outlet continues to function as a semi-module.

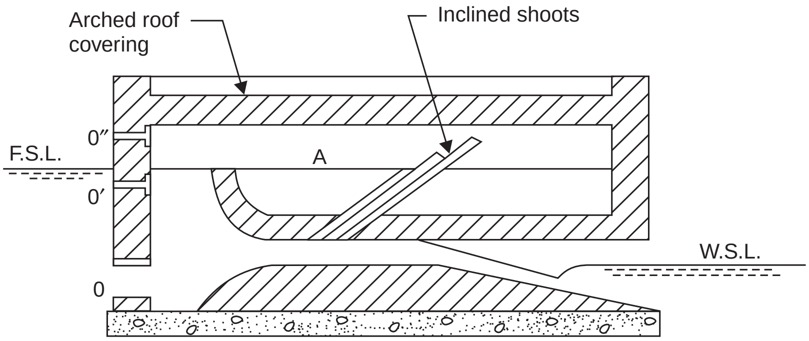

Fig. 5.9 Khanna's module

Example 5.2 A semi-modular pipe outlet of diameter 15 cm is to be installed on a distributary with its bed level and full supply level at 100.3 and 101.5 m, respectively. The maximum water level in the watercourse is at 101.15 m. Set the outlet for maximum discharge and calculate the same. The coefficient C in the discharge equation, Eq. (5.12), may be taken as 0.62. Is the setting proportional, subproportional or hyper-proportional?

Solution: For maximum discharge, the pipe outlet must be at the maximum water level in the watercourse.

Therefore,

$$H = (101.5 - 101.15) - \frac{0.15}{2} = 0.275 \text{ m}$$

and

$$Q = CA\sqrt{2\,gH}$$

$$= 0.62 \times \frac{3.14 \times (0.15)^2}{4}\sqrt{2 \times 9.81 \times 0.275}$$

$$= 0.0254 \text{ m}^3\text{/s}$$

Flexibility

$$F = \frac{m}{n} \times \frac{h}{H}$$

and

$$\frac{m}{n} = \frac{1}{2} \times \frac{3}{5} = 0.3$$

Therefore,

$$F = \frac{0.3 \times 1.2}{0.275} = 1.309$$

Therefore, the setting is hyper-proportional.

Example 5.3 A distributary channel having bed width 5.00 m and full supply depth of 1.20 m carries 3.0 m^3/s of discharge. A semi-modular pipe outlet in this channel has a command area of 15 ha growing rice with a *kor* depth 20 cm and *kor* period of three weeks. Determine the size of the outlet and set it for sub-proportionality with a flexibility of 0.9. Assume the length of the pipe as 3.0 m and friction factor as 0.03. The available diameters of the pipe are 150, 125, 100, and 75 mm.

How does this outlet behave if the distributary runs below FSL at 1.0 m depth ?

Solution:

Outlet discharge factor, $D = \dfrac{8.64 \times 3 \times 7}{0.2} = 907.2$ ha/m^3/s

Therefore, outlet discharge = 15/907.2 = 0.0165 m^3/s

From Eq. (5.5), $F = \dfrac{m}{n} \times \dfrac{h}{H}$

$$\therefore \quad H = \frac{(1/2)}{(5/3)} \times \frac{1.2}{0.9} = 0.4 \text{ m}$$

From Eq. (5.12),

$$Q = \left[\frac{d(2\,gH)}{1.5\,d + fL}\right]^{1/2} \frac{\pi}{4} d^2$$

$$= \left[\left(\frac{\pi}{4}\right)^2 2\,gH.\frac{d^5}{1.5\,d + fL}\right]^{1/2}$$

$$= \left[\left(\frac{\pi}{4}\right)^2 2g\,(0.4)\,\frac{d^5}{1.5\,d + (0.03)\,(3)}\right]^{1/2}$$

$$\therefore \quad Q = \left[\frac{4.84\,d^5}{1.5\,d + 0.09}\right]^{1/2}$$

Assuming d = 0.1 m, Q = 0.142 m^3/s which is less than required.

For d = 0.125 m, Q = 0.023 m^3/s which is okay.

Therefore, recommended diameter of the outlet is 125 mm.

When the distributary is running at 1.0 m depth (*i.e.*, 0.2 m below FSL) then

H = 0.4 – 0.2 = 0.2 m and h = 1.0 m.

$$\therefore \quad F = \left(\frac{1/2}{5/3}\right)\frac{1.0}{0.2} = 1.5$$

Therefore, the outlet behaves as hyper-proportional outlet.

5.10. CANAL REGULATION

The amount of water which can be directed from a river into the main canal depends on: (*i*) the water available in the river, (*ii*) the canal capacity, and (*iii*) the share of other canals taking off from the river. The flow in the main canal is diverted to various branches and distributaries.

The distribution of flow, obviously, depends on the water demand of various channels. The method of distribution of available supplies is termed *canal regulation*.

When there exists a significant demand for water anywhere in the command area of a canal, the canal has to be kept flowing. The canal can, however, be closed if the water demand falls below a specified quantity. It is reopened when the water demand exceeds the specified minimum quantity. Normally, there always exists a demand in some part of the command area of any major canal. Such major canals can, therefore, be closed only for a very small period (say, three to four weeks in a year). These canals run almost continuously and carry discharges much less than their full capacity, either when there is less demand or when the available supplies are insufficient.

If the demand is less, only the distributaries which need water are kept running and the others (including those which have very little demand) are closed. In case of keen demand, but insufficient supplies, either all smaller channels run simultaneously and continuously with reduced supplies, or some channels are closed turn by turn and the remaining ones run with their full or near-full capacities. The first alternative causes channel silting, weed growth, increased seepage, waterlogging, and low heads on outlets. The second alternative does not have these disadvantages and allows sufficient time for inspection and repair of the channels.

A roster is usually prepared for indicating the allotted supplies to different channels and schedule of closure and running of these channels. It is advantageous to have flexible regulation so that the supplies can be allocated in accordance with the anticipated demand. The allocation of supplies is decided on the basis of the information provided by the canal revenue staff who keep a close watch on the crop condition and irrigation water demand.

The discharge in canal is usually regulated at the head regulator which is usually designed as a meter. When the head regulator cannot be used as discharge meter, a depth gauge is provided at about 200 m downstream of the head regulator. The gauge reading is suitably related to the discharge. By manipulating the head regulator gates, the desired gauge reading (and, hence, the discharge) can be obtained.

5.11. DELIVERY OF WATER TO FARMS

Once water has been brought to the watercourse, the problem of its equitable distribution amongst the farms located along the watercourse arises. There are the following two possible alternatives (4), each with its own merit, for achieving this objective:

(*i*) Restrict the canal irrigation to such limited areas as can be fully supported with the lowest available supply. This does not lead to the total utilisation of available water. Agricultural production and protection against famine would also not be optimum. The production would be maximum per unit of land covered though not per unit of water available. It would, however, not require a precise or sophisticated method for the distribution of irrigation water. The delivery system for this alternative can be either continuous or demand-based, depending upon the availability of water. A continuous delivery system can be effectively used for large farms and continuously terraced rice fields. Though ideal, a demand-based delivery system is not practical on large irrigation systems.

(*ii*) Extend irrigation to a much larger area than could be supported by the lowest available supply. This creates perpetual scarcity of irrigation water but ensures that a comparatively less quantity of water remains unutilised. Agricultural production and the protection against famine would be at the optimum levels. The production would be maximum per unit of available water though not per unit of land covered. This method would have greater social appeal, and requires precise and sophisticated methods for equitable distribution of irrigation water.

Irrigation water from a distributary can possibly be delivered to farmers in the following four different ways:

(*i*) Continuous delivery system

(*ii*) Free demand system

(*iii*) Rotation delivery system

(*iv*) Controlled demand (or modified rotation) system.

In the continuous delivery system water is supplied continuously to the farm at a predetermined rate. This system is easy to operate, but would generally result in excessive water applications. This delivery system can be efficient only if the farms are so large that the farmer can redistribute his supply at the farm to different pockets of his farm in accordance with crop and soil conditions. This may necessitate regulatory storage at the farm for efficient utilization of the water delivered to the farm. Large corporation farms or state-owned farms can be served efficiently by this system.

In a free demand system, farmers take intermittent delivery at will, depending upon the needs of their crops, from the constantly available supply in such a manner that their instantaneous withdrawal rate does not exceed that for which they subscribe and which also corresponds, in some way to the installed capacity. Obviously, this system provides maximum flexibility but requires that the farmer is closely aware of crop irrigation requirement and does not have tendency to overirrigate when water is not sold by volume. This system also leads to uncontrolled peak demands during daylight hours and excessive operational losses during night. The day-time peak demands may require large delivery capabilities. Free demand system is well adapted to well irrigation rather than canal irrigation.

In the rotation delivery system the canal authority assumes responsibility for allocating the continuous flow available in the relevant distributary to each farmer of the area which is served by the distributary. The farmers get water according to a fixed delivery schedule. This method is capable of achieving equitable distribution to a large number of farmers with relatively lesser water supplies. Hence, this method is generally adopted for canal irrigation supplies in India and is known by the name of '*warabandi*' in India and has been described in greater detail in the next article.

Obligatory use of water supplied to the farm may cause considerable wastage of water and result in waterlogging without increasing the production. On the other hand, failure to adjust rotation schedules to crop irrigation requirements may result in inefficient irrigation during crucial growth periods and thus affect adversely the crop production. To overcome the deficiency of the rotation delivery method, water may, alternately, be delivered according to some kind of controlled demand system which is a sort of compromise between free demand and rotation delivery systems. In this modified system of modified rotation, priority for delivery is on a rotation basis, but actual delivery may deviate depending upon the actual demand. Obviously, much better coordination between farmers and the authorities would be required for this system to work efficiently.

5.11.1. Warabandi

Warabandi is an integrated management system from source (river or reservoir) down to the farm gate, *i.e.*, *nakka*. In the *Warabandi* system (Fig. 5.10), the water from the source is carried by the main canal which feeds two or more branch canals (which operate by rotation) and may not carry the total required supply. This is the primary distribution system which runs throughout the irrigation season with varying supply. The secondary distribution system consists of a larger number of distributaries which too run by rotation but carry full supply. They are fed by the branch canals of the primary distribution system. The distributaries supply water to the watercourse through outlets. These watercourses run full supply when the supplying distributary is running.

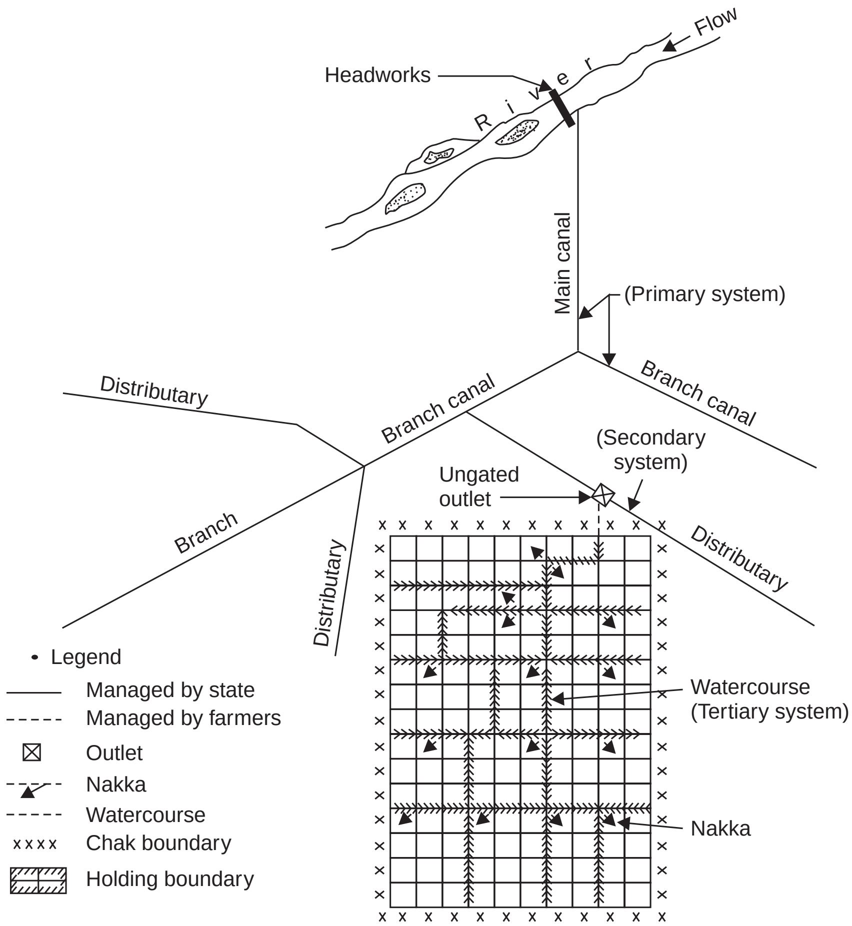

Fig. 5.10 Typical *warabandi* distribution system (4)

Water is then allocated to various fields (or farms) situated along the watercourse by a time roster. This is the tertiary distribution system.

Warabandi is a distribution system whose main objective is to attain high efficiency of water use by imposing water scarcity on every user. The system ensures equitable distribution and safeguards the interest of the farmer whose field is located at the tail end of the conveyance system. Such a system is a classic example of the joint state-farmer management of the irrigation system. The segment upstream of the outlet is managed by the state whereas the farmers manage the segment downstream of the outlet.

In the *warabandi* system, each unit of culturable command area is allocated a certain rate of flow of water, termed water allowance, whose value is generally a compromise between demand and supply. The carrying capacity of distributaries and watercourses is designed on the basis of water allowance. Whenever distributaries run, they are expected to carry their full supply. The outlets to watercourses are so planned and constructed that all the watercourses on a distributary withdraw their authorised share of water simultaneously. For the Bhakra project covering Punjab, Haryana, and Rajasthan, the value of water allowance at the head of the watercourse is 0.017 m^3/s per 100 hectares of culturable command area.

To check the dangers of waterlogging and salinity, no distributary is allowed to operate all the days during any crop season. The ratio of the operating period of a distributary and the crop period is called the *capacity factor* of the distributary. For the Bhakra project, the capacity factors of Kharif and Rabi are, respectively, 0.8 and 0.72 which means that each distributary would receive its full supply for a period of about 144 and 129 days, respectively, in a crop season of 180 days.

Because of the limits on the supply of irrigation water as well as other factors, it is generally not possible to irrigate all culturable command area. The ratio of the resultant irrigated area to the culturable command area is termed intensity of irrigation. Its value is 62 per cent for the Bhakra project. The intensity of irrigation is an index of the actual performance of the irrigation system.

In the *warabandi* system, the design of distributaries and watercourses is related to the culturable command area (which is fixed) rather than the variable cropping pattern. The total amount of water available at the source has its own limitations and it may not always be possible to expand or augment the supply to keep pace with the ever-increasing demand of the cropping pattern. Therefore, there is an obvious advantage of relating the design to the culturable command area rather than to the needs of the cropping pattern.

5.11.2. Management of *Warabandi* System

The distribution of water in the *warabandi* system is a two-tier operation and each is managed by a separate agency. The state manages the supply in distributaries and watercourses which, when running, always carry their full supply discharge. This reduces their running time and, hence, the conveyance losses in the distributaries and the watercourses. The distributaries are generally operated in eight-day periods. The number of these periods would depend on the availability of water and crop requirements. In a normal year, it is possible to run the distributaries of the Bhakra project for 18 periods during Kharif and 16 periods during Rabi.

The second stage of managing the distribution of water coming out of an outlet and flowing into a watercourse is the responsibility of the cultivators themselves. The distribution is done on seven-day rotation basis with the help of an agreed roster (or roster of turns) which divides 168 hours of seven days in the ratio of the holdings. The eight-day period of distributary running ensures a minimum of seven days running for each watercourse including those which are at the tail end of the distributary.

Each cultivator's right to share water in a watercourse is guaranteed by law and the Canal Act empowers canal officers to ensure this right for everyone.

Whenever a distributary is running, watercourse receives its share of water at a constant rate round the clock and water distribution proceeds from head to tail. Each cultivator on the watercourse is entitled to receiving the entire water in a watercourse only on a specific weekday and at a specific time (during day and/or night). There is no provision in this system to compensate a defaulting farmer who has failed to utilise his turn for any reason.

5.11.3. Roster of Turn

The cycle of turns on a watercourse or its branch starts from its head, proceeds downstream, and ends at its tail. Before a farmer receives his share of water, some time is spent in filling up the empty watercourse between the point of taking over and the beginning of his holding. This time is called *bharai* (*i.e.*, filling time) which is debited to a common pool time of 168 hours and credited to the account of the concerned farmer.

The supply in the watercourse has to be stopped when the tail-end farmer is having his turn. The water filled in the watercourse during the common pool time (*bharai*) can be discharged only into the field of the tail-end farmer and, hence, normally the total time spent on the filling of the watercourse should be recovered from him in lieu of this. But he does not receive this water at a constant rate. Such a supply, beyond a limit, is not efficient from the point of view of field application. The tail-end farmer is, therefore, compensated for it and is allowed a certain discount on the recovery of the *bharai* time. This value of *bharai* is termed *jharai* (*i.e.*, emptying time). Obviously, correct determination of *jharai* time is an unresolved problem and its present values are favourable to the tail-end farmer.

After allowing for *bharai* and *jharai*, the flow time (FT) for a unit area and for an individual farmer are given as follows (4):

$$\text{FT for unit area} = \frac{168 - (\text{Total } bharai - \text{Total } jharai)}{\text{Total area}}$$

$$\text{FT for farmer} = (\text{FT for unit area} \times \text{area of farmer's field}) + (\text{his } bharai - \text{his } jharai)$$

Obviously, *bharai* is generally zero in the case of the last farmer and *jharai* is zero for all except the last farmer. It should be noted that the losses in watercourse are not reflected anywhere in the above equations.

5.12. FLOW MEASUREMENT

The importance of accurate flow measurement for proper regulation, distribution, and charging of irrigation water cannot be overemphasised. There are several flow measuring devices available for flow measurement in irrigation systems. Generally weirs and flumes are used for this purpose. Besides, there are some other indirect methods by which velocities are measured and the discharge computed. In these methods, the channel section is divided into a suitable number of compartments and the mean velocity of flow for each of these compartments is measured by using devices such as current meter, surface floats, double floats, velocity rods, and so on. The discharge through any compartment is obtained by multiplying the mean velocity of flow with the area of cross-section of the compartment. The sum of all compartmental discharges gives the channel discharge.

5.12.1. Weirs

Weirs have been in use as discharge measuring devices in open channels for almost two centuries. A *weir* is an obstruction over which flow of a liquid occurs (Fig. 5.11). Head H over the weir is related to the discharge flowing and, hence, the weir forms a useful discharge measuring device. Weirs can be broadly classified as thin-plate (or sharp-crested) and broad-crested weirs.

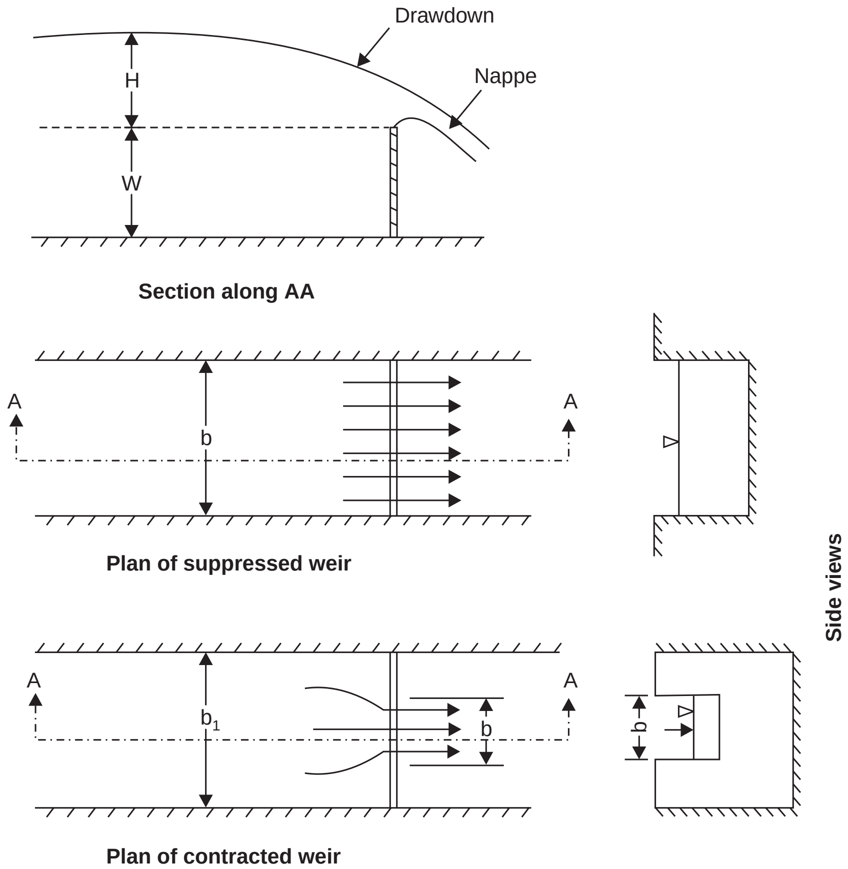

Fig. 5.11 Flow over suppressed and contracted weirs

5.12.1.1. Thin-Plate Weirs

A sharp-crested (or thin-plate) weir is formed in a smooth, plane, and vertical plate and its edges are bevelled on the downstream side to give minimum contact with the liquid. The area of flow is most commonly either triangular or rectangular and, accordingly, the weir is said to be a triangular or rectangular weir. In general, the triangular weir (or simply the V-notch) is used for the measurement of low discharges, and the rectangular weir for the measurement of large discharges.

The pattern of the flow over a thin-plate weir is very complex and cannot be analysed theoretically. This is due to the non-hydrostatic pressure variation (on account of curvature of streamlines), turbulence and frictional effects, and the approach flow conditions. The effects of viscosity and surface tension also become important at low heads. Therefore, the analytical relation (between the rate of flow and the head over the weir), obtained after some simplifying assumptions, are suitably modified by experimentally determined coefficients. Following this approach, Ranga Raju and Asawa (5) obtained the following discharge equations:

For thin-plate triangular weir with notch angle θ

$$Q = k_1 \frac{8}{15} C_d \sqrt{2g}\,(\tan \theta/2)\, H^{5/2} \tag{5.14}$$

For a suppressed thin-plate rectangular weir

$$Q = \left[\frac{2}{3}\left(0.611 + 0.075\,\frac{H}{W}\right) b\,\sqrt{2g} H^{3/2}\right] k_1 \tag{5.15}$$

where, Q = discharge flowing over the weir,

H = head over the weir,

b = width of the weir,

C_d = coefficient of discharge for triangular weir (Fig. 5.12),

A = area of cross-section of the approach flow,

k_1 = correction factor to account for the effects of viscosity and surface tension (Fig. 5.13),

$R_e = g^{1/2} H^{3/2}/\nu$ (typical Reynolds number),

ν = kinematic viscosity of the flowing liquid,

$W_1 = \rho g H^2/\sigma$ (typical Weber number),

σ = surface tension of the flowing liquid,

ρ = mass density of the flowing liquid, and

g = acceleration due to gravity.

It should be noted that $k_1 = 1.0$ for $R_e^{0.2}\, W_1^{0.6}$ greater than 900. This limit corresponds to a head of 11.0 cm for water at 20°C. The mean line drawn in Fig. 5.13 can be used to find the value of k_1. The scatter of data (not shown in the figure) was generally less than 5 per cent implying maximum error of ± 5 per cent in the prediction of discharge.

Equation (5.15) along with Fig. 5.13, and Eq. (5.14) along with Figs. 5.12 and 5.13 enable computations of discharge over a suppressed thin-plate rectangular weir and a thin-plate 90°-triangular weir, respectively. A weir is termed suppressed when its width equals the channel width and in such cases the ventilation of nappe becomes essential.

5.12.1.2. Broad-Crested Weirs

Broad-crested weirs are generally used as diversion and metering structures in irrigation systems in India. The weir (Fig. 5.14) has a broad horizontal crest raised sufficiently above the bed so that the cross-sectional area of the approaching flow is much larger than the cross-sectional area of flow over the top of the weir. The upstream edge of the weir is well rounded to avoid undue eddy formation and consequent loss of energy. The derivation of the discharge equation for flow over a broad-crested weir is based on the concept of critical flow. Ranga Raju and Asawa (6) proposed the following discharge equation for a broad-crested weir with well-rounded upstream edge and vertical upstream and downstream faces:

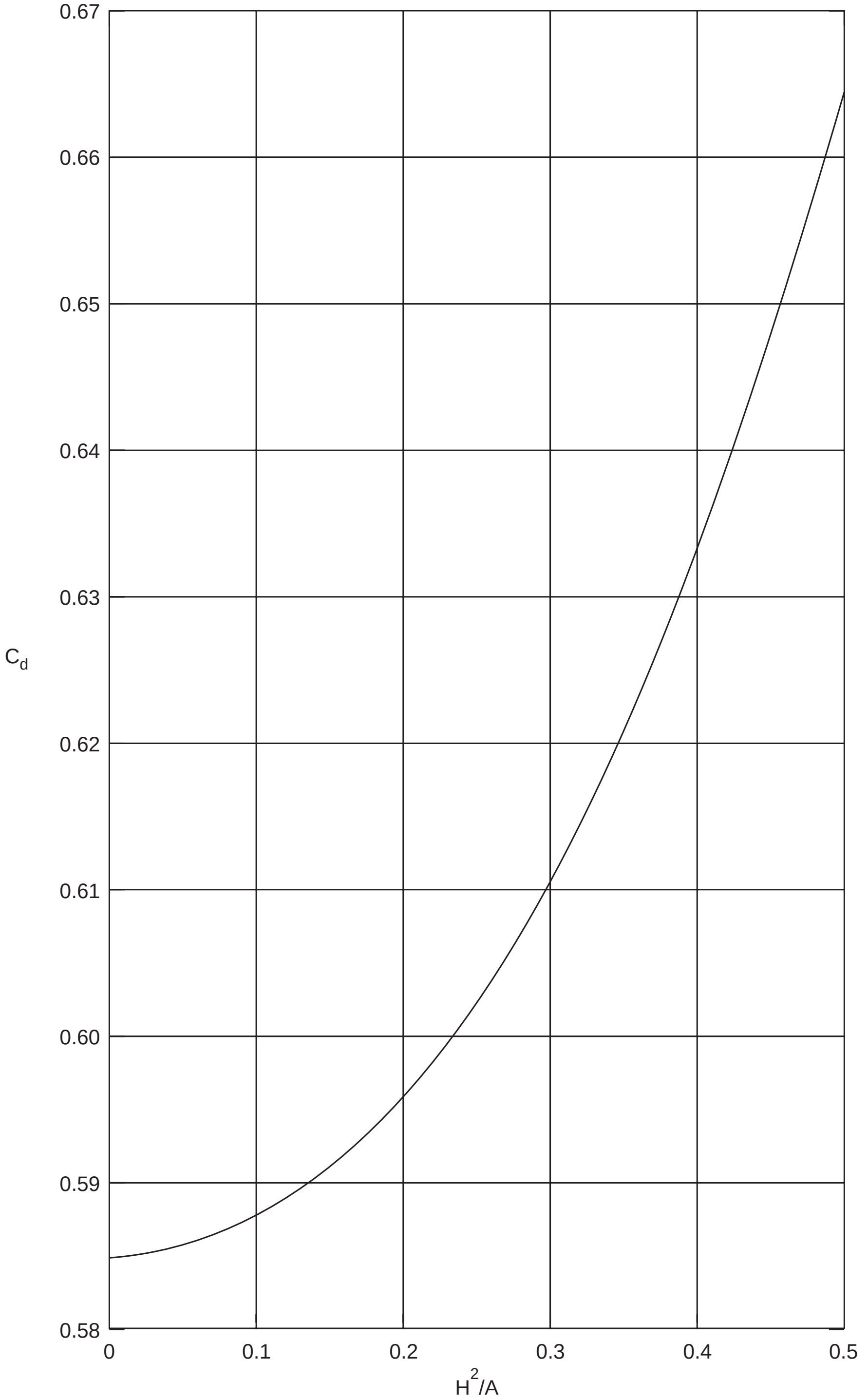

Fig. 5.12 Variation of C_d with H^2/A for 90°-triangular weir (5)

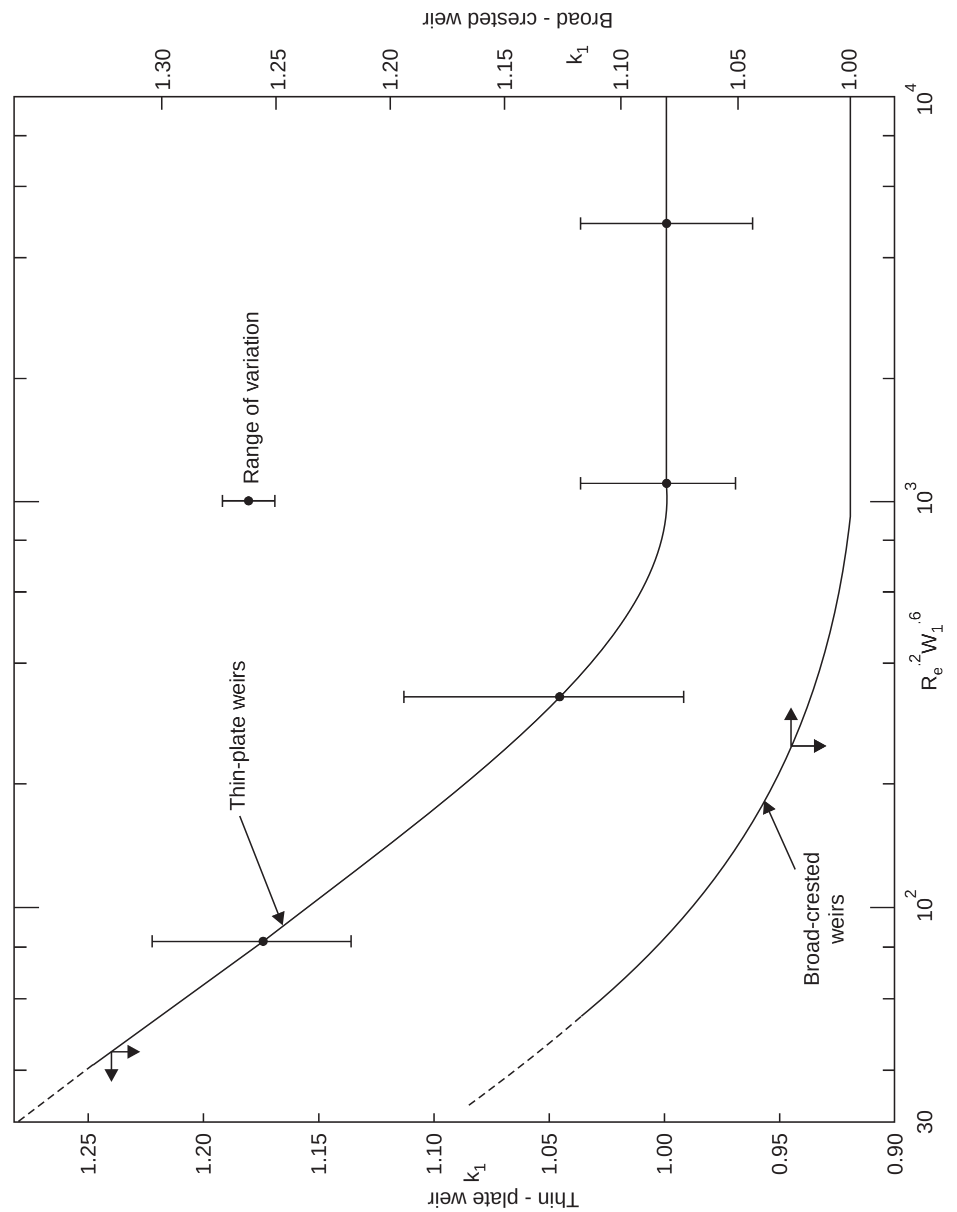

Fig. 5.13 Correction factor k_1 for influence of viscosity and surface tension (5, 6)

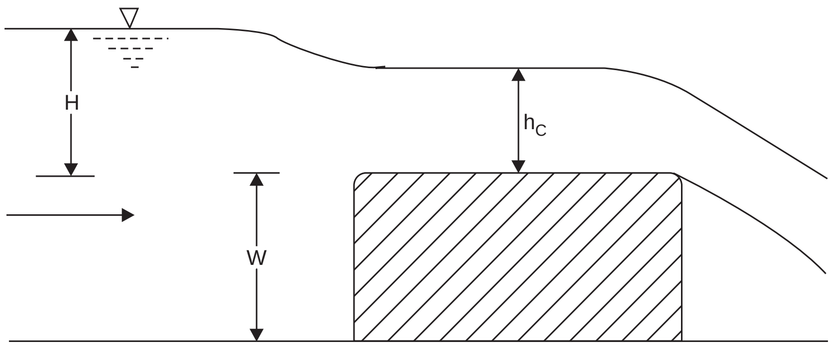

Fig. 5.14 Flow over a broad-crested weir

$$Q = k_1\, k_2\, Cb\, \sqrt{g}\, H^{3/2} \tag{5.16}$$

Here, k_2 is the correction for the effect of curvature of flow over the weir crest (Fig. 5.15), and L is the length of the weir along the flow direction. C is obtained from Fig. 5.16 which is based on the following equation:

$$\frac{H}{H+W} = \frac{\sqrt{3}\,[C^{2/3} - (2/3)]^{1/2}}{(b/b_1)\,C} \tag{5.17}$$

For suppressed broad-crested weirs $b = b_1$. Here, b_1 is the width of the channel. Thus, one uses the curve for $b/b_1 = 1.0$ in Fig. 5.16 to find C. k_1 is obtained from Fig. 5.13. For broad-crested weirs with sloping upstream and downstream faces one can use Eq. (5.16) with different functional relations for k_1 and k_2 shown in Figs. 5.17 and 5.18, respectively.

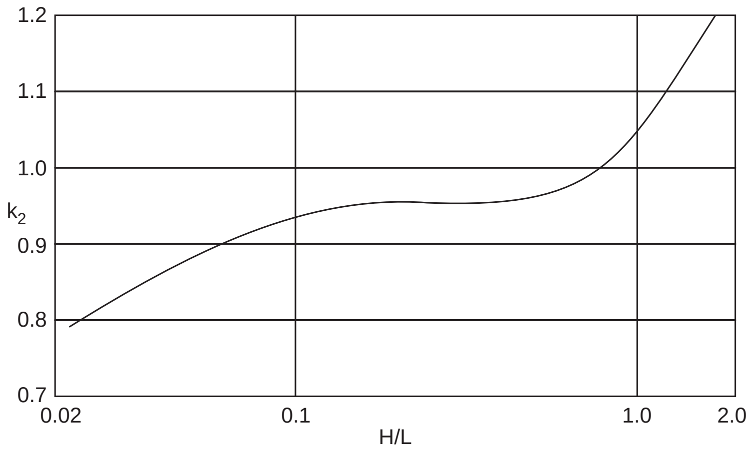

Fig. 5.15 Variation of k_2 with H/L for vertical-faced weir with rounded corner

For a submerged broad-crested weir, the discharge equation is written as (7, 8)

$$Q = Cb\,\sqrt{g}\; H^{3/2}\, k_1\, k_2\, k_4 \tag{5.18}$$

Assuming that C, k_1, and k_2 remain unaffected due to submergence, relationship for k_4 is as shown in Fig. 5.19. It should be noted that the discharge on broad-crested weirs remains unaffected up to submergence (H_2/H_1) as high as 75 per cent. Figure 5.20 compares the discharge characteristics of submerged broad-crested weirs with those of sharp-crested weirs.

Weirs of widths smaller than that of the approach channel are termed contracted weirs. The above-mentioned relationships, Eq. (5.15) for sharp-crested rectangular weirs and Eq. (5.16) for broad-crested weirs, require some modifications for contracted weirs. Ranga Raju and Asawa (6) suggested the following equation for the actual discharge over a contracted sharp-crested rectangular weir:

$$Q = k_1\, k_3\, \frac{2}{3}\, [0.611 + C_1\, (H/W)]\, b\, \sqrt{2g}\, H^{3/2} \tag{5.19}$$

in which k_3 is the correction factor for lateral contraction and C_1 is a function of b/b_1 as shown in Fig. 5.21. k_3 should logically be a function of H/b. On analysing the experimental data, the average value of k_3 was found to be 0.95 for H/b ranging from 0.1 to 1.0 (6).

Similarly, the actual discharge over a contracted broad-crested weir may be written as (6)

$$Q = k_1\, k_2\, k_3\, Cb\, \sqrt{g}\, H^{3/2} \tag{5.20}$$

in which k_3 is a correction factor for contraction effects and the value of C for various values of b/b_1, as obtained by solving Eq. (5.17), can be read from Fig. 5.16. Logically, k_3 should be a function of H/b. Based on the analysis of experimental data, the value of k_3 can be taken as unity for H/b ranging from 0.1 to 1.0 (6).

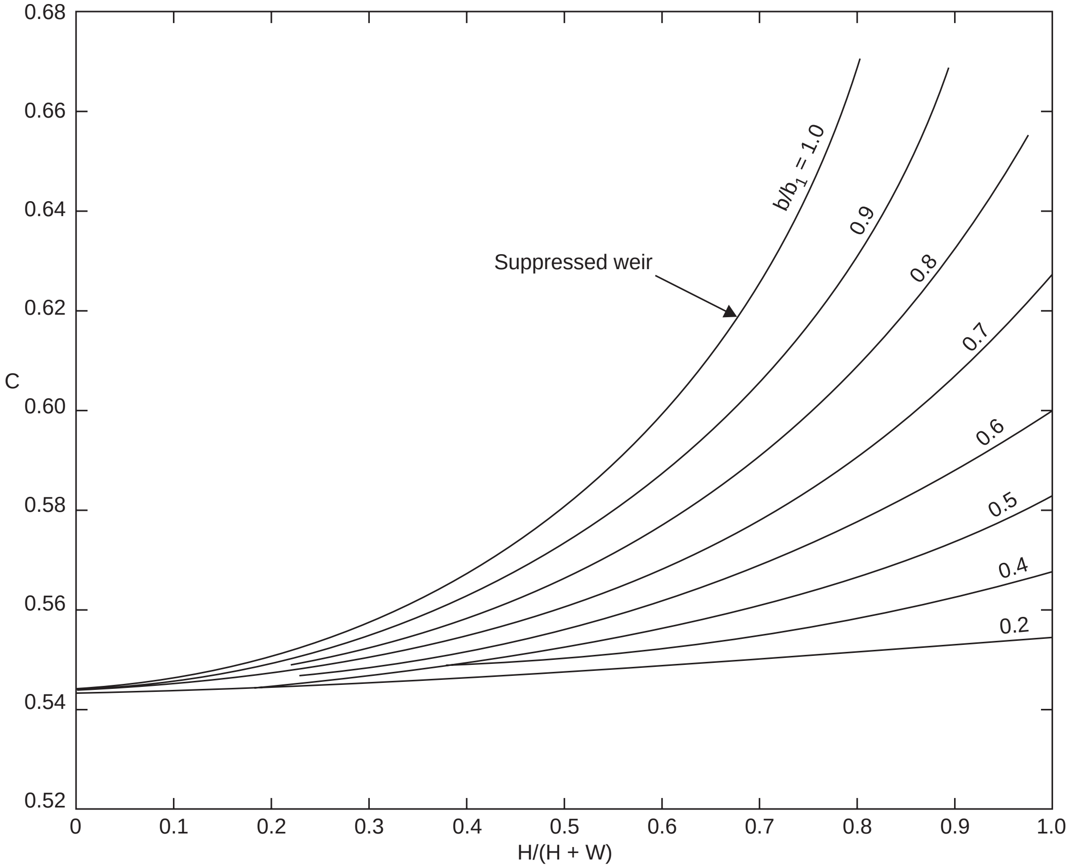

Fig. 5.16 Variation of C with $H/(H + W)$ and b/b_1 for broad-crested weirs (6)

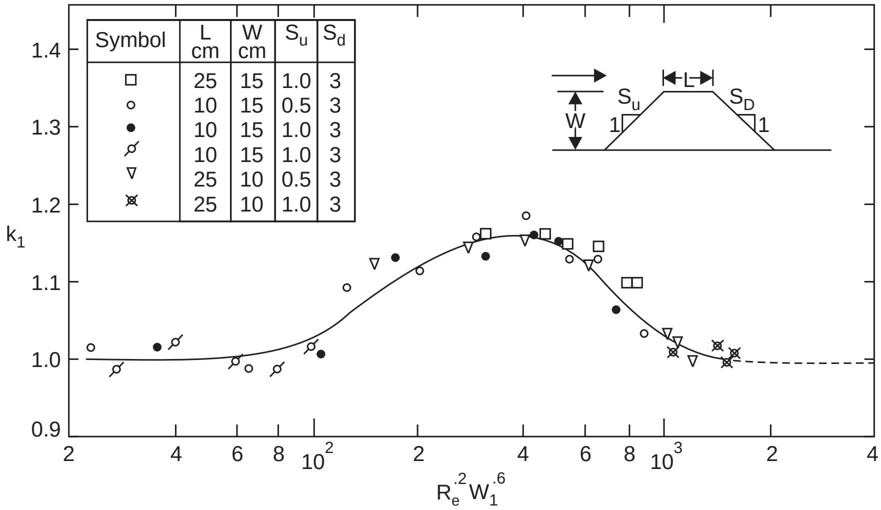

Fig. 5.17 Variation of k_1 for broad-crested weirs with sloping faces (7)

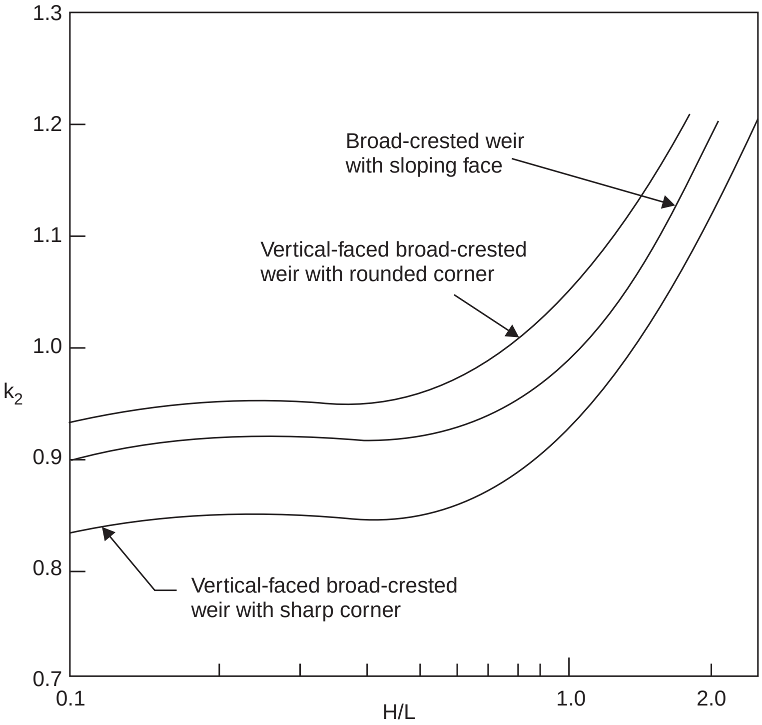

Fig. 5.18 Variation of k_2 with H/L for broad-crested weirs with sloping faces (8)

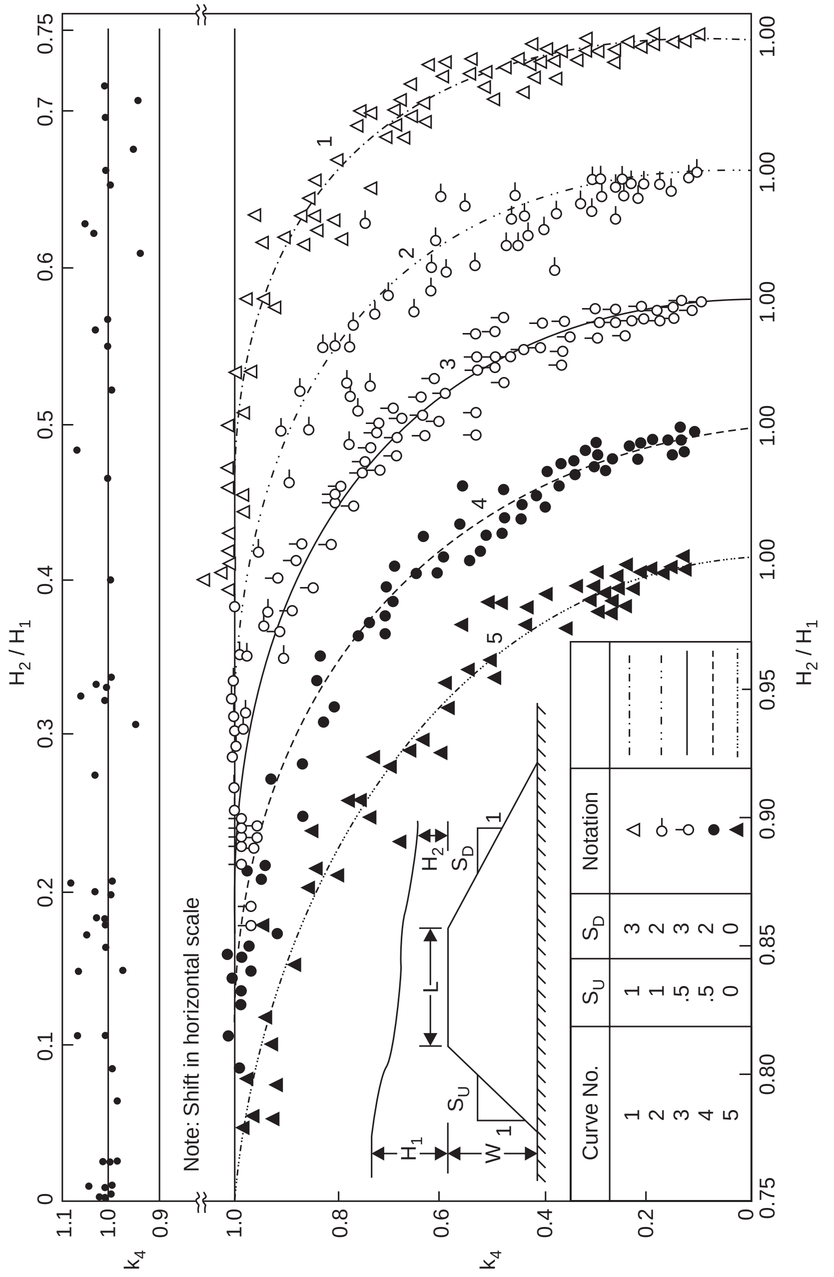

Fig. 5.19 Variation of k_4 with H_2/H_1 and weir geometry (8)

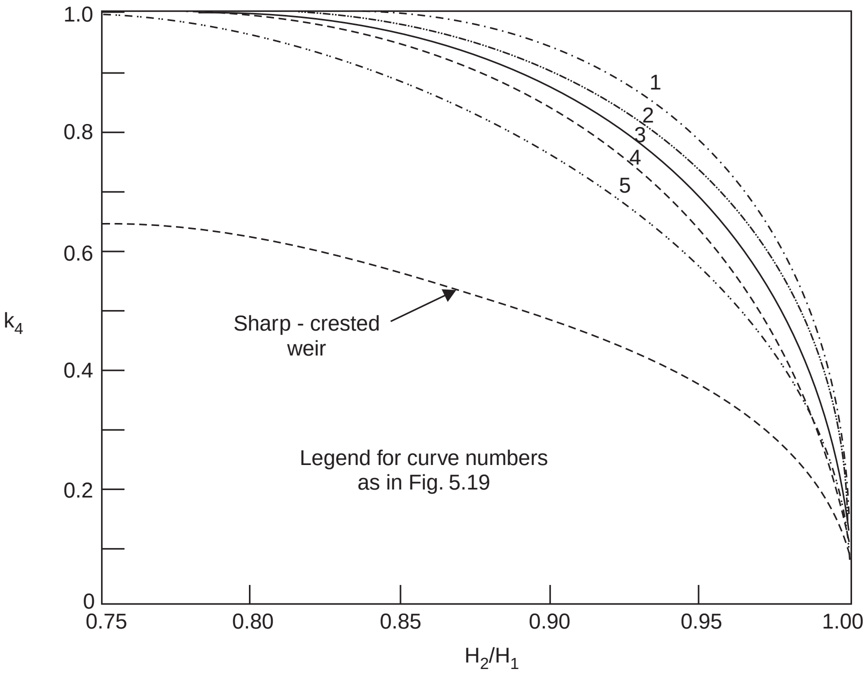

Fig. 5.20 Comparison of discharge characteristics of different weirs under submerged conditions (8)

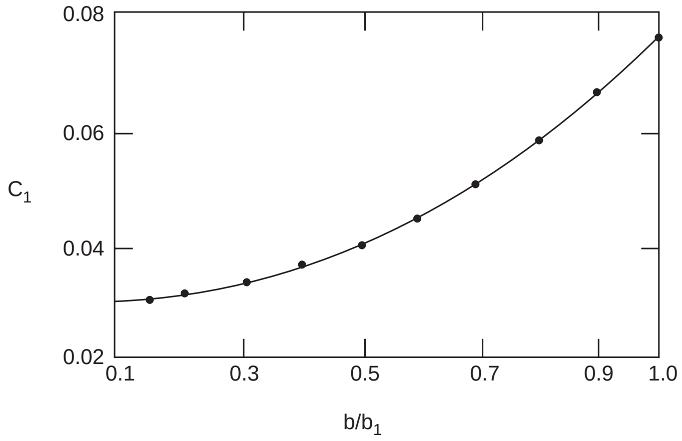

Fig. 5.21 Variation of C_1 with b/b_1 for contracted thin-plate weirs

The advantages of weirs for discharge measurement are as follows:

(*i*) Simplicity and ease in construction,

(*ii*) Durability, and

(*iii*) Accuracy.

However, the main requirement of considerable fall of water surface makes their use in areas of level ground impracticable. Besides, deposition of sand, gravel, and silt upstream of the weir prevents accurate measurements.

5.12.2. Flumes

A *flume* is a flow measuring device formed by a constriction in an open channel. The constriction can be either a narrowing of the channel or a narrowing in combination with a hump in the invert. By providing sufficient amount of constriction, it is possible to produce critical flow conditions there. When this happens, there exists a unique stage-discharge relationship independent of the downstream conditions. The use of critical-depth flumes for discharge measurement is based on this principle.

The main advantage of a critical-depth flume over a weir is in situations when material (sediment or sewage) is being transported by the flow. This material gets deposited upstream of the weir and affects the discharge relation and results in a foul-smelling site in case of sewage flow. The critical-depth flumes consisting only of horizontal contraction would easily carry the material through the flume. Critical-depth fumes can be grouped into two main categories.

5.12.2.1. Long-Throated Flumes

The constriction of these flumes (Fig. 5.22) is sufficiently long (the length of the throat should be at least twice the maximum head of water that will occur upstream of the flume) so that it produces small curvature in the water surface and the flow in the throat is virtually parallel to the invert of the flume.

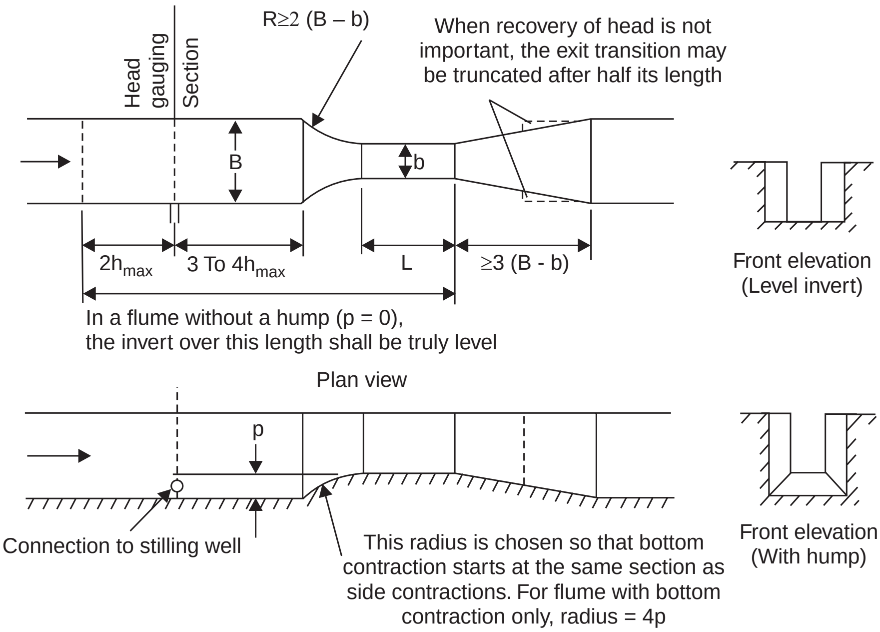

Fig. 5.22 Geometry of rectangular long-throated flume

This condition results in nearly hydrostatic pressure distribution at the control section (where critical depth occurs) which, in turn, allows analytical derivation of the stage-discharge relation. This gives the designer the freedom to vary the dimensions of the flume in order to meet specific requirements. Such flumes are usually of rectangular, trapezoidal, triangular or U-shaped cross-section. For a rectangular flume, the discharge of an ideal fluid is expressed as

$$Q = \left(\frac{2}{3}\right)^{3/2} \sqrt{g}\, bH^{3/2} \tag{5.21}$$

Here, H represents the upstream energy and b is the typical width dimension for the particular cross-sectional shape of the flume. By introducing suitable coefficients this equation can be generalised in the following form so that it applies to any cross-sectional shape (9):

$$Q = \left(\frac{2}{3}\right)^{3/2} \sqrt{g}\, C_v\, C_s\, C_d\, bh^{3/2} \tag{5.22}$$

where, C_v = coefficient to take into account the velocity head in the approach channel,

C_s = coefficient to take account of the cross-sectional shape of the flume,

C_d = coefficient for energy loss,

and h = depth of water, upstream of the flume, measured relative to the invert level of the throat (*i.e.*, gauged head).

5.12.2.2. Short-Throated Flumes

In these flumes, the curvature of the water surface is large and the flow in the throat is not parallel to the invert of the flume. The principle of operation of these flumes is the same as that of long-throated flumes, *viz.* the creation of critical conditions at the throat. However, non-hydrostatic pressure distribution (due to large curvature of flow) does not permit analytical derivation of the discharge equation. Further, energy loss also cannot be assessed. Therefore, it becomes necessary to rely on direct calibration either in the field or in the laboratory for the determination of the discharge equation. The designer does not have complete freedom in choosing the dimensions of the flume but has to select the closest standard design to meet his requirements. Such flumes, however, require lesser length and, hence, are more economical than long-throated flumes. One of the most commonly used short-throated flumes is the Parshall flume which has been described here.

Parshall (10, 11 and 12) designed a short-throated flume with a depressed bottom (Fig. 5.23) which is now known as the Parshall flume. This was first developed in the 1920's in the USA and has given satisfactory service at water treatment plants and irrigation projects. It consists of 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.

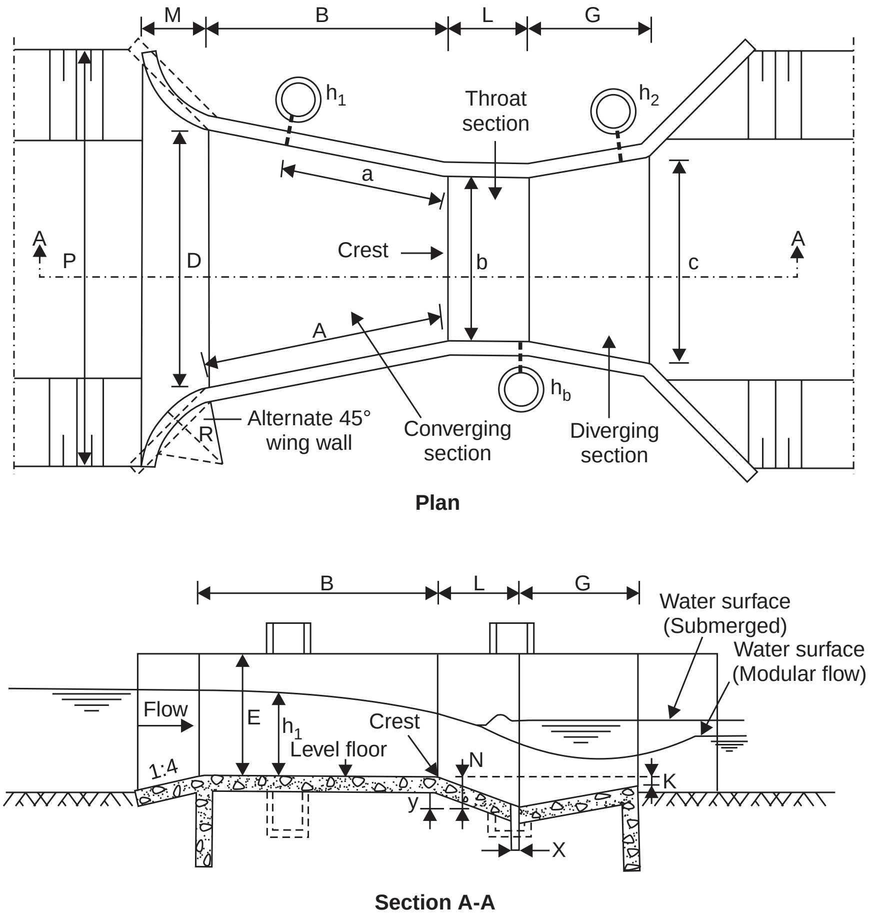

Fig. 5.23 Parshall flume

There are 22 standard designs covering a wide range of discharge from 0.1 litre per second to 93 m^3/s. The main dimensions of the Parshall flume are given in Table 5.3 and Fig. 5.23. The discharge characteristic of these flumes are given in Table 5.4.

5.12.3. Current Meter

The current meter is a widely used mechanical device for the measurement of flow velocity and, hence, the discharge in an open channel flow. It consists of a small wheel with cups at the periphery or propeller blades rotated by the force of the flowing water, and a tail or fins to keep the instrument aligned in the direction of flow. The cup-type current meter has a vertical axis, and is a more rugged instrument which can be handled by relatively unskilled technicians. The propeller-type current meter has been used for relatively higher velocities (up to 6 to 9 m/s as against 3 to 5 m/s for the cup-type current meter). The small size of the propeller-type current meter is advantageous when the measurements have to be taken close to the wall. The propeller-type meter is less likely to be affected by floating weeds and debris.

Table 5.3 Parshall flume dimensions (mm) (9)

b	b (mm)	A	a	B	C	D	E	L	G	H	K	M	N	P	R	X	Y	Z
1 in	25.4	363	242	356	93	167	229	76	203	203	19	–	29	–	–	8	13	3
2 in	50.8	414	276	406	135	214	254	114	254	257	22	–	43	–	–	16	25	6
3 in	76.2	467	311	457	178	259	457	152	305	309	25	–	57	–	–	25	38	13
6 in	152.4	621	414	610	394	397	610	305	610	–	76	305	114	902	406	5	76	–
9 in	228.6	879	587	864	381	575	762	305	610	–	76	305	114	1080	406	51	76	–
1 ft	304.8	1372	914	1343	610	845	914	610	914	–	76	381	229	1492	508	51	76	–
1 ft 6 in	457.2	1448	965	1419	762	1026	914	610	914	–	76	381	229	1676	508	51	76	–
2 ft	609.6	1524	1016	1495	914	1206	914	610	914	–	76	381	229	1854	508	51	76	–
3 ft	914.4	1676	1118	1645	1219	1572	914	610	914	–	76	381	229	2222	508	51	76	–
4 ft	1219.2	1829	1219	1794	1524	1937	914	610	914	–	76	457	229	2411	610	51	76	–
5 ft	1524.0	1981	1321	1943	1829	2302	914	610	914	–	76	457	229	3080	610	51	76	–
6 ft	1828.8	2134	1422	2092	2134	2667	914	610	914	–	76	457	229	3442	610	51	76	–
7 ft	2133.6	2286	1524	2242	2438	3032	914	610	914	–	76	457	229	3810	610	51	76	–
8 ft	2438.4	2438	1626	2391	2743	3397	914	610	914	–	76	457	229	4172	610	51	76	–
10 ft	3048	–	1829	4267	3658	4756	1219	914	1829	–	152	–	343	–	–	305	229	–
12 ft	3658	–	2032	4877	4470	5607	1524	914	2438	–	152	–	343	–	–	305	229	–
15 ft	4572	–	2337	7620	5588	7620	1829	1219	3048	–	229	–	457	–	–	305	229	–
20 ft	6096	–	2845	7620	7315	9144	2134	1829	3658	–	305	–	686	–	–	305	229	–
25 ft	7620	–	3353	7620	8941	10668	2134	1829	3962	–	305	–	686	–	–	305	229	–
30 ft	9144	–	3861	7925	10566	12313	2134	1829	4267	–	305	–	686	–	–	305	229	–
40 ft	12192	–	4877	8230	13818	15481	2134	1829	4877	–	305	–	686	–	–	305	229	–
50 ft	15240	–	5893	8230	17272	18529	2134	1829	6096	–	305	–	686	–	–	305	229	–

Table 5.4 Discharge characteristics of parshall flumes $Q = kh_1^n$ (9)

Throat width b	*Discharge range*		*k*	*n*	*Head range, m*		*Modular limit*
	minimum	*maximum* (*l/s*)			*minimum*	*maximum*	h_2/h_1
1 in	0.09	5.4	0.0604	1.55	0.015	0.21	0.50
2 in	0.18	13.2	0.1207	1.55	0.015	0.24	0.50
3 in	0.77	32.1	0.1771	1.55	0.03	0.33	0.50
6 in	1.50	111	0.3812	1.58	0.03	0.45	0.60
9 in	2.50	251	0.5354	1.53	0.03	0.61	0.60
1 ft	3.32	457	0.6909	1.52	0.03	0.76	0.70
1 ft 6 in	4.80	695	1.056	1.538	0.03	0.76	0.70
2 ft	12.1	937	1.428	1.550	0.046	0.76	0.70
3 ft	17.6	1427	2.184	1.566	0.046	0.76	0.70
4 ft	35.8	1923	2.953	1.578	0.06	0.76	0.70
5 ft	44.1	2424	3.732	1.587	0.06	0.76	0.70
6 ft	74.1	2929	4.519	1.595	0.076	0.76	0.70
7 ft	85.8	3438	5.312	1.601	0.076	0.76	0.70
8 ft	97.2	3949	6.112	1.607	0.076	0.76	0.70
	m^3/s						
10 ft	0.16	8.28	7.463	1.60	0.09	1.07	0.80
12 ft	0.19	14.68	8.859	1.60	0.09	1.37	0.80
15 ft	0.23	25.04	10.96	1.60	0.09	1.67	0.80
20 ft	0.31	37.97	14.45	1.60	0.09	1.83	0.80
25 ft	0.38	47.14	17.94	1.60	0.09	1.83	0.80
30 ft	0.46	56.33	21.44	1.60	0.09	1.83	0.80
40 ft	0.60	74.70	28.43	1.60	0.09	1.83	0.80
50 ft	0.75	93.04	35.41	1.60	0.09	1.83	0.80

For measurements, the current meter is mounted on a rod and moved vertically to measure the velocity at different points. The speed of rotation of cups or blades depends on the velocity of flow. The instrument has an automatic counter with which the number of rotations in a given duration is determined.

The current meter is calibrated by moving it with a known speed in still water and noting the number of revolutions per unit of time. During measurement, the current meter is held stationary in running water. Using the appropriate calibration (supplied by the manufacturer) the velocity can be predicted. By this method one can obtain velocity distribution and, hence, the discharge. Or, alternatively, one can measure the velocity at 0.2 h and 0.8 h (here, h is the depth of flow) below the free surface and the mean of the two values gives the average velocity of flow. Sometimes, velocity at 0.6 h is taken as the average velocity of flow.

5.12.4. Other Methods

Mean velocities in open channels can, alternatively, be determined by measuring surface velocities using surface floats. The surface float is an easily visible object lighter than water, but sufficiently heavy not to be affected by wind. The surface velocity is measured by noting down the time the surface float takes in covering a specified distance which is generally not less than 30 metres and 15 metres for large and small channels, respectively. The surface velocity is multiplied by a suitable coefficient (less than unity) to get the average velocity of flow.

A double float consists of a surface float to which is attached a hollow metallic sphere heavier than water. Obviously, the observed velocity of the double float would be the mean of the surface velocity and the velocity at the level of the metallic sphere. By adjusting the metallic sphere at a depth nearly equal to $0.2h$ above the bed, the observed velocity will be approximately equal to the mean velocity of flow.

Alternatively, velocity rods can be used for the measurement of average velocity of flow. Velocity rods are straight wooden rods or hollow tin tubes of 25 mm to 50 mm diameter and weighted down at the bottom so that these remain vertical and fully immersed except for a small portion at the top while moving in running water. These rods are either telescopic-type or are available in varying lengths so that they can be used for different depths of flow. As the rod floats vertically from the surface to very near the bed, its observed velocity equals the mean velocity of flow in that vertical plane.

For measuring discharge in a pipeline, one may employ either orifice meter or venturi meter or bend meter or any other suitable method.

5.13. ASSESSMENT OF CHARGES FOR IRRIGATION WATER

Irrigation projects involve huge expenditure for their construction. The operation and maintenance of these projects also require finances. With the introduction of irrigation facilities in an area, farmers of the area are immensely benefited. Hence, it is only appropriate that they are suitably charged for the irrigation water supplied to them.

The assessment of irrigation water charges can be done in one of the following ways:

(*i*) Assessment on area basis,

(*ii*) Volumetric assessment,

(*iii*) Assessment based on outlet capacity,

(*iv*) Permanent assessment, and

(*v*) Consolidated assessment.

In the area basis method of assessment, water charges are fixed per unit area of land irrigated for each of the crops grown. The rates of water charges depend on the cash value of crop, water requirement of crop, and the time of water demand with respect to the available supplies in the source. Since the water charges are not related to the actual quantity of water used, the farmers (particularly those whose holdings are in the head reaches of the canal) tend to overirrigate their land. This results in uneconomical use of available irrigation water besides depriving the cultivators in the tail reaches of the canal of their due share of irrigation water. However, this method of assessment, being simple and convenient, is generally used for almost all irrigation projects in India.

In volumetric assessment, the charges are in proportion to the actual amount of water received by the cultivator. This method, therefore, requires installation of water meters at all the outlets of the irrigation system. Alternatively, modular outlets may be provided to supply a specified discharge of water. This method results in economical use of irrigation water and is, therefore, an ideal method of assessment. However, it has several drawbacks. This method requires the installation and maintenance of suitable devices for measurement of water supplied. These devices require adequate head at the outlet. Further, there is a possibility of water theft by cutting of banks or siphoning over the bank through a flexible hose pipe. Also, the distribution of charges among the farmers, whose holdings are served by a common outlet, may be difficult. Because of these drawbacks, this method has not been adopted in India.

The assessment of canal water charges based on outlet capacity is a simple method and is workable if the outlets are rigid or semi-modular and the channel may run within their modular range.

In some regions, artificial irrigation, though not essential, has been provided to meet the water demand only in drought years. Every farmer of such a region has to pay a fixed amount. The farmers have to pay these charges even for the years for which they do not take any water. A farmer has also to pay a tax on the land owned by him. In the consolidated assessment method, both the land revenue and the water charges are combined and the cultivators are accordingly charged.

5.14. WATERLOGGING

In all surface water irrigation schemes, supplying the full water requirements of a crop, more water is added to the soil than is actually required to make up the deficit in the soil resulting from continuous evapotranspiration by crops. This excess water and the water that seeps into the ground from reservoirs, canals, and watercourses percolate deep into the ground to join the water table and, thus, raise the water table of the area. When the rising water table reaches the root zone, the pore spaces of the root-zone soil get saturated.

A land is said to be *waterlogged* when the pores of soil within the root zone of a plant gets saturated and the normal growth of the plant is adversely affected due to insufficient air circulation. The depth of the water table at which it starts affecting the plant would depend on plant and soil characteristics. A land would become waterlogged sooner for deep-rooted plants than for shallow-rooted plants. Impermeable soils generally have higher capillary rise and, hence, are waterlogged more easily than permeable soils. A land is generally waterlogged when the ground water table is within 1.5 to 2.0 m below the ground surface. Water table depth is good if the water table is below 2 m and rises to 1.8 m for a period not exceeding 30 days in a year (13). If the water table is at about 1.8 m and rises to about 1.2 m for a period not exceeding 30 days in a year, the condition is considered as fair. If the water table depth is between 1.2 to 1.8 m which may rise to 0.9 m for a period not exceeding 30 days in a year, the condition of water table depth is rather poor. In a poor condition of water table depth, the water level is less than 1.2 m from the surface and is generally rising.

A high water table increases the moisture content of the unsaturated surface soil and thus increases the permeability. There may be advantages of having water table close to the surface as it may result in higher crop yield due to favourable moisture supply. This may, however, be true only for few years after water table has risen from great depths. The favourable condition may be followed by serious decrease in the crop yield in areas where alkali salts are present. With slight increase in inflow to the ground, the high water table may become too close to the ground surface and when this happens the land gets waterlogged and becomes unsuitable for cultivation.

The problem of waterlogging is a world-wide phenomenon which occurs mainly due to the rise of the ground water table beyond permissible limits on account of the change in ground water balance brought about by the percolation of irrigation water. It has become a problem of great importance on account of the introduction of big irrigation projects. The land subjected to waterlogging results in reduction of agricultural production. The problem of waterlogging has already affected about 5 million hectares of culturable area in India (see Table 6.2 for more details).

5.14.1. Causes of Waterlogging

Ground water reservoirs receive their supplies through percolation of water from the ground surface. This water may be from rainfall, from lakes or water applied to the fields for irrigation. This water percolates down to the water table and, thus, raises its position. Depending upon the elevation and the gradient of the water table, the flow may either be from surface to the ground (*i.e.*, inflow) or ground to the surface (*i.e.*, outflow). Outflow from a ground water reservoir includes water withdrawn through wells and water used as consumptive use. An overall balance between the inflow and outflow of a ground water reservoir will keep the water table at almost fixed level. This balance is greatly disturbed by the introduction of a canal system or a well system for irrigation. While the former tends to raise the water table, the latter tends to lower it.

Waterlogging in any particular area is the result of several contributing factors. The main causes of waterlogging can be grouped into two categories: (*i*) natural, and (*ii*) artificial.

5.14.1.1. Natural Causes of Waterlogging

Topography, geological features, and rainfall characteristics of an area can be the natural causes of waterlogging.

In steep terrain, the water is drained out quickly and, hence, chances of waterlogging are relatively low. But in flat topography, the disposal of excess water is delayed and this water stands on the ground for a longer duration. This increases the percolation of water into the ground and the chances of waterlogging. The geological features of subsoil have considerable influence on waterlogging. If the top layer of the soil is underlain by an impervious stratum, the tendency of the area getting waterlogged increases.

Rainfall is the major contributing factor to the natural causes of waterlogging. Low-lying basins receiving excessive rainfall have a tendency to retain water for a longer period of time and, thus get, waterlogged. Submergence of lands during floods encourages the growth of weeds and marshy grasses which obstruct the drainage of water. This, again, increases the amount of percolation of water into the ground and the chances of waterlogging.

5.14.1.2. Artificial Causes of Waterlogging

There exists a natural balance between the inflow and outflow of a ground water reservoir. This balance is greatly disturbed due to the introduction of artificial irrigation facilities. The surface reservoir water and the canal water seeping into the ground increase the inflow to the ground water reservoir. This raises the water table and the area may become waterlogged. Besides, defective method of cultivation, defective irrigation practices, and blocking of natural drainage further add to the problem of waterlogging.

5.14.2. Effects of Waterlogging

The crop yield is considerably reduced in a waterlogged area due to the following adverse effects of waterlogging:

(*i*) Absence of soil aeration,

(*ii*) Difficulty in cultivation operations,

(*iii*) Weed growth, and

(*iv*) Accumulation of salts.

In addition, the increased dampness of the waterlogged area adversely affects the health of the persons living in that area.

5.14.2.1. Absence of Soil Aeration

In waterlogged lands, the soil pores within the root zone of crops are saturated and air circulation is cut off. Waterlogging, therefore, prevents free circulation of air in the root zone. Thus, waterlogging adversely affects the chemical processes and the bacterial activities which are essential for the proper growth of a plant. As a result, the yield of the crop is reduced considerably.

5.14.2.2. Difficulty in Cultivation

For optimum results in crop production, the land has to be prepared. The preparation of land (*i.e.*, carrying out operations such as tillage, *etc.*) in wet condition is difficult and expensive. As a result, cultivation may be delayed and the crop yield adversely affected. The delayed arrival of the crop in the market brings less returns to the farmer.

5.14.2.3. Weed Growth

There are certain types of plants and grasses which grow rapidly in marshy lands. In waterlogged lands, these plants compete with the desired useful crop. Thus, the yield of the desired useful crop is adversely affected.

5.14.2.4. Accumulation of Salts

As a result of the high water table in waterlogged areas, there is an upward capillary flow of water to the land surface where water gets evaporated. The water moving upward brings with it soluble salts from salty soil layers well below the surface. These soluble salts carried by the upward moving water are left behind in the root zone when this water evaporates. The accumulation of these salts in the root zone of the soil may affect the crop yield considerably.

5.14.3. Remedial Measures for Waterlogging

The main cause of waterlogging in an area is the introduction of canal irrigation there. It is, therefore, better to plan the irrigation scheme in such a way that the land is prevented from getting waterlogged. Measures, such as controlling the intensity of irrigation, provision of intercepting drains, keeping the full supply level of channels as low as possible, encouraging economical use of water, removing obstructions in natural drainage, rotation of crops, running of canals by rotation, *etc.*, can help considerably in preventing the area from getting waterlogged.

In areas where the water table is relatively high, canal irrigation schemes should be planned for relatively low intensity of irrigation. In such areas canal irrigation should be allowed in the Kharif season only. Rabi irrigation should be carried out using ground water. Intercepting drains provided along canals with high embankments collect the canal water seeping through the embankments and, thus, prevent the seeping water from entering the ground. The full supply level in the channels may be kept as low as possible to reduce the seepage losses. The level should, however, be high enough to permit flow irrigation for most of the command area of the channel. For every crop there is an optimum water requirement for the maximum yield. The farmers must be made aware that the excessive use of water would harm the crop rather than benefit it. The levelling of farm land for irrigation, and a more efficient irrigation system decrease percolation to the ground and reduce the chances of waterlogging. The improvement in the existing natural drainage would reduce the amount of surface water percolating into the ground. A judicious rotation of crops can also help in reducing the chances of waterlogging. Running of canals by rotation means that the canals are run for few days and then kept dry for some days. This means that there would not be seepage for those days when the canal is dry. This, of course, is feasible only in case of distributaries and watercourses.

The combined use of surface and subsurface water resources of a given area in a judicious manner to derive maximum benefits is called *conjunctive* use of water. During dry periods, the use of ground water is increased, and this results in lowering of the water table. The use of surface water is increased during the wet season. Because of the lowered water table, the ground water reservoir receives rainfall supplies through increased percolation. The utilisation of water resources in this manner results neither in excessive lowering of the water table nor in its excessive rising. The conjunctive use of surface and subsurface water serves as a precautionary measure against waterlogging. It helps in greater water conservation and lower evapotranspiration losses, and brings larger areas under irrigation.

The most effective method of preventing waterlogging in a canal irrigated area, however, is to eliminate or reduce the seepage of canal water into the ground. This can be achieved by the lining of irrigation channels (including watercourses, if feasible). In areas which have already become waterlogged, curative methods such as surface and subsurface drainage and pumping of ground water are useful.

5.14.4. Lining of Irrigation Channels

Most of the irrigation channels in India are earthen channels. The major advantage of an earth channel is its low initial cost. The disadvantages of an earth channel are: (*i*) the low velocity of flow maintained to prevent erosion necessitates larger cross-section of channels, (*ii*) excessive seepage loss which may result in waterlogging and related problems such as salinity of soils, expensive road maintenance, drainage activities, safety of foundation structures, etc., (*iii*) favourable conditions for weed growth which further retards the velocity, and (*iv*) the breaching of banks due to erosion and burrowing of animals. These problems of earth channels can be got rid of by lining the channel.

A lined channel decreases the seepage loss and, thus, reduces the chances of waterlogging. It also saves water which can be utilised for additional irrigation. A lined channel provides safety against breaches and prevents weed growth thereby reducing the annual maintenance cost of the channel. Because of relatively smooth surface of lining, a lined channel requires a flatter slope. This results in an increase in the command area. The increase in the useful head is advantageous in case of power channels also. The lining of watercourses in areas irrigated by tubewells assume special significance as the pumped water supply is more costly.

As far as practicable, lining should, however, be avoided on expansive clays (14). But, if the canal has to traverse a reach of expansive clay, the layer of expansive clay should be removed and replaced with a suitable non-expansive soil and compacted suitably. If the layer of expansive clay is too thick to be completely excavated then the expansive clay bed is removed to a depth of about 60 cm and filled to the grade of the underside of lining with good draining material. The excavated surface of expansive clay is given a coat of asphalt to prevent the entry of water into the clay.

The cost of lining a channel is, however, the only factor against lining. While canal lining provides a cost-effective means of minimising seepage losses, the lining itself may rapidly deteriorate and require recurring maintenance inputs if they are to be effective in controlling seepage loss. A detailed cost analysis is essential for determining the economic feasibility of lining a channel. The true cost of lining is its annual cost rather than the initial cost. The cost of lining is compared with the direct and indirect benefits of lining to determine the economic feasibility of lining a channel. Besides economic factors, there might be intangible factors such as high population density, aesthetics, and so on which may influence the final decision regarding the lining of a channel.

5.14.5. Economics of Canal Lining

The economic viability of lining of a canal is decided on the basis of the ratio of additional benefits derived from the lining to additional cost incurred on account of lining. The ratio is worked out as follows (15):

Let C = cost of lining in Rs/sq. metre including the additional cost of dressing the banks for lining and accounting for the saving, if any, resulting from the smaller cross-sections and, hence, smaller area of land, quantity of earth work, and structures required for the lined sections. This saving will be available on new canals excavated to have lined cross-section right from the beginning, but not on lining of the existing unlined canals.

s and S = seepage losses in unlined and lined canals, respectively, in cubic metres per square metre of wetted surface per day of 24 hrs.

p and P = wetted perimeter in metres of unlined and lined sections, respectively,

T = total perimeter of lining in metres,

d = number of running days of the canal per year,

W = value of water saved in rupees per cubic metre,

L = length of the canal in metres,

y = life of the canal in years,

M = annual saving in rupees in operation and maintenance due to lining, taking into account the maintenance expenses on lining itself,

and B = annual estimated value in rupees of other benefits for the length of canal under consideration. These will include prevention of waterlogging, reduced cost of drainage for adjoining lands, reduced risk of breach, and so on.

The annual value of water lost by seepage from the unlined section

$$= pLsdW \text{ rupees.}$$

The annual saving in value of water otherwise lost by seepage

$$= (pLsdW - PL\,SdW) \text{ rupees}$$

$$= \{LdW\,(ps - PS)\} \text{ rupees}$$

Total annual benefits resulting from the lining of canal

$$B_t = \{LdW\,(ps - PS) + B + M\} \text{ rupees} \tag{5.23}$$

Additional capital expenditure on construction of lined canal

$$= TLC \text{ rupees}$$

If the prevalent rate of interest is x per cent per year, the annual instalment a (rupees) required to be deposited each year (at its beginning) for a number of y years to amount to TLC plus its interest at the end of y years is determined by the following equation:

$$TLC\left(1+\frac{x}{100}\right)^{y} = a\left\{\left(1+\frac{x}{100}\right)^{y} + \left(1+\frac{x}{100}\right)^{y-1} + \left(1+\frac{x}{100}\right)^{y-2} + \ldots + \left(1+\frac{x}{100}\right)^{1}\right\}$$

$$= \frac{a\left(1+\frac{x}{100}\right)\left\{\left(1+\frac{x}{100}\right)^{y} - 1\right\}}{\left\{\left(1+\frac{x}{100}\right) - 1\right\}}$$

$$\therefore \qquad a = \frac{(TLC)\left(\frac{x}{100}\right)\left(\frac{100+x}{100}\right)^{y-1}}{\left(\frac{100+x}{100}\right)^{y} - 1} \qquad (5.24)$$

For lining to be economically feasible, the value of a should be less than the annual benefit B_t *i.e.,* the ratio B_t/a should be greater than unity.

5.14.6. Types of Lining

Types of lining are generally classified according to the materials used for their construction. Concrete, rock masonry, brick masonry, bentonite-earth mixtures, natural clays of low permeability, and different mixtures of rubble, plastic, and asphaltic materials are the commonly used materials for canal lining. The suitability of the lining material is decided by: (*i*) economy, (*ii*) structural stability, (*iii*) durability, (*iv*) reparability, (*v*) impermeability, (*vi*) hydraulic efficiency, and (*vii*) resistance to erosion (15). The principal types of lining are as follows:

(*i*) Concrete lining,

(*ii*) Shotcrete lining,

(*iii*) Precast concrete lining,

(*iv*) Lime concrete lining,

(*v*) Stone masonry lining,

(*vi*) Brick lining,

(*vii*) Boulder lining,

(*viii*) Asphaltic lining, and

(*ix*) Earth lining.

5.14.6.1. Concrete Lining

Concrete lining is probably the best type of lining. It fulfils practically all the requirements of lining. It is durable, impervious, and requires least maintenance. The smooth surface of the concrete lining increases the conveyance of the channel. Properly constructed concrete lining can easily last about 40 years. Concrete linings are suitable for all sizes of channels and for both high and low velocities. The lining cost is, however, high and can be reduced by using mechanised methods.

The thickness of concrete depends on canal size, bank stability, amount of reinforcement, and climatic conditions. Small channels in warm climates require relatively thin linings.

Channel banks are kept at self-supporting slope (1.5H: 1V to 1.25H: 1V) so that the lining is not required to bear earth pressures and its thickness does not increase. Concrete linings are laid without form work and, hence, the workability of concrete should be good. Also, experienced workmen are required for laying concrete linings.

Reinforcement in concrete linings usually varies from 0.1 to 0.4% of the area in the longitudinal direction and 0.1 to 0.2% of the area in the transverse direction. The reinforcement in concrete linings prevents serious cracking of concrete to reduce leakage, and ties adjacent sections of the lining together to provide increased strength against settlement damage due to unstable subgrade soils or other factors. The reinforcement in concrete linings does not prevent the development of small shrinkage which tend to close when canals are operated and linings are watersoaked. The damage due to shrinkage and temperature changes is avoided or reduced by the use of special construction joints. Reinforced concrete linings may result in increased watertightness of the lining. However, well-constructed unreinforced concrete linings may be almost equally watertight.

The earlier practice of using reinforced concrete linings is now being replaced by the employment of well-constructed unreinforced concrete linings. However, reinforcement must be provided in: (*a*) large canals which are to be operated throughout the year, (*b*) sections where the unreinforced lining may not be safe, and (*c*) canals in which flow velocities are likely to be very high.

Proper preparation of subgrade is essential for the success of the concrete lining which may, otherwise, develop cracks due to settlement. Natural earth is generally satisfactory for this purpose and, hence, subgrade preparation is the least for channels in excavation. Thorough compaction of subgrade for channels in filling is essential for avoiding cracks in lining due to settlement.

Some cracks usually develop in concrete linings. These can be sealed with asphaltic compounds. The lining may be damaged when flow in the canal is suddenly stopped and the surrounding water table is higher than the canal bed. This damage occurs in excavated channels and can be prevented by providing weep holes in the lining or installing drains with outlets in the canal section.

Values of minimum thickness of concrete lining based on canal capacity have been specified as given in Table 5.5.

Table 5.5 Thickness of concrete lining (14)

Canal capacity (m^3/s)	*Thickness of M-150 concrete (cm)*		*Thickness of M-100 concrete (cm)*	
	Controlled	*Ordinary*	*Controlled*	*Ordinary*
0 to less than 5	5.0	6.5	7.5	7.5
5 to less than 15	6.5	6.5	7.5	7.5
15 to less than 50	8.0	9.0	10.0	10.0
50 to less than 100	9.0	10.0	12.5	12.5
100 and above	10.0	10.0	12.5	15.0

Concrete linings have been used in the Nangal Hydel canal. Amaravathi project, the Krishnagiri Reservoir project, and several other projects. The use of concrete lining in India is, however, limited because of the low cost of water and high cost of lining. The Bureau of Indian Standards does not specify use of reinforcement for cement concrete lining (14).

5.14.6.2. Shotcrete Lining

Shotcrete lining is constructed by applying cement mortar pneumatically to the canal surface. Cement mortar does not contain coarse aggregates and, therefore, the proportion of cement is higher in shotcrete mix than in concrete lining. The shotcrete mix is forced under pressure through a nozzle of small diameter and, hence, the size of sand particles in the mix should not exceed 0.5 cm. Equipment needed for laying shotcrete lining is light, portable, and of smaller size compared to the equipment for concrete lining. The thickness of the shotcrete lining may vary from 2.5 to 7.5 cm. The preferred thickness is from 4 to 5 cm.

Shotcrete lining is suitable for: (*a*) lining small sections, (*b*) placing linings on irregular surfaces without any need to prepare the subgrade, (*c*) placing linings around curves or structures, and (*d*) repairing badly cracked and leaky old concrete linings.

Shotcrete linings are subject to cracking and may be reinforced or unreinforced. Earlier, shotcrete linings were usually reinforced. A larger thickness of shotcrete lining was preferred for the convenient placement of reinforcement. The reinforcement was in the form of wire mesh. In order to reduce costs, shotcrete linings are not reinforced these days, particularly on relatively small jobs.

5.14.6.3. Precast Concrete Lining

Precast concrete slabs, laid properly on carefully prepared subgrades and with the joints effectively sealed, constitute a serviceable type of lining. The precast slabs are about 5 to 8 cm thick with suitable width and length to suit channel dimensions and to result in weights which can be conveniently handled. Such slabs may or may not be reinforced. This type of lining is best suited for repair work as it can be placed rapidly without long interruptions in canal operation. The side slopes of the Tungabhadra project canals have been lined with precast concrete slabs.

5.14.6.4. Lime Concrete Lining

The use of this type of lining is limited to small and medium size irrigation channels with capacities of up to 200 m^3/s and in which the velocity of water does not exceed 2 m/s (16). The materials required for this type of lining are lime, sand, coarse aggregate, and water. The lime concrete mix should be such that it has a minimum compressive strength of about 5.00 kN/m^2 after 28 days of moist curing. Usually lime concrete is prepared with 1 : 1.5 : 3 of *kankar* lime : *kankar* grit or sand : *kankar* (or stone or brick ballast) aggregate. The thickness of the lining may vary from 10 to 15 cm for discharge ranges of up to 200 m^3/s. Lime concrete lining has been used in the Bikaner canal taking off from the left bank of the Sutlej.

5.14.6.5. Stone Masonry Lining

Stone masonry linings are laid on the canal surface with cement mortar or lime mortar. The thickness of the stone masonry is about 30 cm. The surface of the stone masonry may be smooth plastered to increase the hydraulic efficiency of the canal. Stone masonry linings are stable, durable, erosion-resistant, and very effective in reducing seepage losses. Such lining is very suitable where only unskilled labour is available and suitable quarried rock is available at low price. This lining has been used in the Tungabhadra project.

5.14.6.8. Brick Lining

Bricks are laid in layers of two with about 1.25 cm of 1 : 3 cement mortar sandwiched in between. Good quality bricks should be used and these should be soaked well in water before being laid on the moistened canal surface.

Brick lining is suitable when concrete is expensive and skilled labour is not available. Brick lining is favoured where conditions of low wages, absence of mechanisations, shortage of cement and inadequate means of transportation exist. Brick linings have been extensively used in north India. The Sarda power channel has been lined with bricks. The thickness of the brick lining remains fixed even if the subgrade is uneven. Brick lining can be easily laid in rounded sections without form work. Rigid control in brick masonry is not necessary. Sometimes reinforced brick linings are also used.

5.14.6.7. Boulder Lining

Boulder lining of canals, if economically feasible, is useful for preventing erosion and where the ground water level is above the bed of the canal and there is a possibility of occurrence of damaging back pressures (17). The stones used for boulder linings should be sound, hard, durable, and capable of sustaining weathering and water action. Rounded or sub-angular river cobbles or blasted rock pieces with sufficient base area are recommended types of stones for boulder lining. Dimensions of stones and thickness of lining are as given in Table 5.6.

Table 5.6 Dimensions of stones and thickness of lining (17)

Canal capacity (m^3/s)	*Thickness of lining (mm)*	*Average dimension along the longest axis (mm)*	*Minimum dimension at any section (mm)*
0 to less than 50	150	150	75
50 to less than 100	225	225	110
100 and above	300	300	150

Wherever required, a 15-cm thick layer of filter material is to be provided. For the laying of boulders, the subgrade (both bed and side slope) of the canal is divided into compartments by stone masonry or concrete ribs. These compartments will not have dimensions more than 15 m along and across the centre line of the canal.

5.14.6.8. Asphaltic Lining

The material used for asphaltic lining is asphalt-based combination of cement and sand mixed in hot condition. The most commonly used asphaltic linings are: (*a*) asphaltic concrete, and (*b*) buried asphaltic membrane. Asphaltic linings are relatively cheaper, flexible, and can be rapidly laid in any time of year. Because of their flexibility, minor movements of the subgrade are not of serious concern. However, asphaltic linings have short life and are unable to permit high velocity of flow. They have low resistance to weed growth and, hence, it is advisable to sterilise the subgrade to prevent weed growth.

Asphaltic concrete is a mixture of asphalt cement, sand, and gravel mixed at a temperature of about 110°C and is placed either manually or with laying equipment. Experienced and trained workmen are required for the purpose. The lining is compacted with heavy iron plates while it is hot.

A properly constructed asphaltic concrete lining is the best of all asphaltic linings. Asphaltic concrete lining is smooth, flexible, and erosion-resistant. Since asphaltic concrete lining becomes distorted at higher temperatures, it is unsuitable for warmer climatic regions. An asphaltic concrete lining is preferred to a concrete lining in situations where the aggregate is likely to react with the alkali constituents of Portland cement.

Buried asphaltic membrane can be of two types:

(*a*) Hot-sprayed asphaltic membrane, and

(*b*) Pre-fabricated asphaltic membrane.

A hot-sprayed asphaltic membrane is constructed by spraying hot asphalt on the subgrade to result in a layer about 6 mm thick. This layer, after cooling, is covered with a layer of earth material about 30 cm thick. The asphalt temperature is around 200°C and the spraying pressure about 3×10^5 N/m^2. For this type of lining, the channel has to be over-excavated. The lining is flexible and easily adopts to the subgrade surface. Skilled workmen are required for the construction of this type of lining.

Pre-fabricated asphaltic membrane is prepared by coating rolls of heavy paper with a 5 mm layer of asphalt or 3 mm of glass fibre-reinforced asphalt. These rolls of pre-fabricated asphaltic membrane are laid on the subgrade and then covered with earth material. These linings can be constructed by commonly available labour.

Materials used for covering the asphaltic membrane determine the permissible velocities which are generally lower than the velocities in unlined canals. Maintenance cost of such linings is high. Cleaning operations should be carried out carefully so as not to damage the membrane.

5.14.6.9. Earth Linings

Different types of earth linings have been used in irrigation canals. They are inexpensive but require high maintenance expenditure. The main types of earth linings are: (*a*) stabilised earth linings, (*b*) loose earth blankets, (*c*) compacted earth linings, (*d*) buried bentonite membranes, and (*e*) soil-cement linings.

Stabilised earth linings: Stabilised earth linings are constructed by stabilizing the subgrade. This can be done either physically or chemically. Physically stabilised linings are constructed by adding corrective materials (such as clay for granular subgrade) to the subgrade, mixing, and then compacting. If corrective materials are not required, the subgrade can be stabilised by scarifying, adding moisture, and then compacting. Chemically stabilised linings use chemicals which may tighten the soil. Such use of chemicals, however, has not developed much.

Loose earth blankets: This type of lining is constructed by dumping fine-grained soils, such as clay, on the subgrade and spreading it so as to form a layer 15 to 30 cm thick. Such linings reduce seepage only temporarily and are soon removed by erosion unless covered with gravel. Better results can be obtained by saturating the clay and then pugging it before dumping on the subgrade. The layer of pugged clay is protected by a cover of about 30 cm silt. This type of lining requires flatter side slopes.

Compacted earth linings: These linings are constructed by placing graded soils on the subgrade and then compacting it. The graded soil should contain about 15% of clay. The compacted earth linings may be either thin-compacted or thick-compacted. In thin-compacted linings, the layer thickness of about 15 to 30 cm along the entire perimeter is used. Thick-compacted linings have a layer about 60 cm thick on the channel bed and 90 cm thick on the sides. If properly constructed, both types are reasonably satisfactory. However, the thick linings are generally preferred.

Compacted-earth linings are feasible when excavated materials are suitable, or when suitable materials are available nearby. Compaction operations along the side slopes are more difficult (particularly in thin-compacted linings) than along the channel bed. The lining material should be tested in the laboratory for density, permeability, and optimum moisture contents. The material must be compacted in the field so as to obtain the desired characteristics.

Buried Bentonite Membranes: Pure bentonite is a hydrous silicate of alumina. Natural deposits of bentonite are special types of clay soil which swell considerably when wetted. The impurities of these soils affect the swelling and, hence, the suitability of these as canal lining material. Buried bentonite linings are constructed by spreading soil-bentonite mixtures over the subgrade and covering it with about 15 to 30 cm of gravel or compacted earth. Sandy soil mixed with about 5 to 25 per cent of fine-grained bentonite and compacted to a thickness of 5 to 7.5 cm results in a membrane which is reasonably tough and suitable for lining.

Soil-cement Linings: These linings are constructed using cement (15 to 20 per cent by volume) and sandy soil (not containing more than about 35 per cent of silt and clay particles). Cement and sandy soil can be mixed in place and compacted at the optimum moisture content. This method of construction is termed the dry-mixed soil-cement method. Alternatively, soil-cement lining can be constructed by machine mixing the cement and soil with water and placing it on the subgrade in a suitable manner. This method is called the plastic soil-cement method and is preferable. In both these methods, the lining should be kept moist for about seven days to permit adequate curing.

The construction cost of soil-cement linings is relatively high. But these resist weed growth and erosion and also permit velocities slightly higher than those permitted by unlined earth channels. The use of soil-cement linings for irrigation canals is restricted to small irrigation canals with capacities of up to 10 m^3/s and in which the velocity of water does not exceed 1 m/s (18).

5.14.7. Failure of Lining

The main causes of failure of lining are the water pressure that builds up behind the lining material due to high water table, saturation of the embankment by canal water, sudden lowering of water levels in the channel, and saturation of the embankment sustained by continuous rainfall. The embankment of a relatively pervious soil does not need drainage measures behind the lining. In all situations requiring drainage measures to relieve pore pressure behind the lining, a series of longitudinal and transverse drains satisfying filter criteria are provided. A typical arrangement of longitudinal filter drain is as shown in Fig. 5.24.

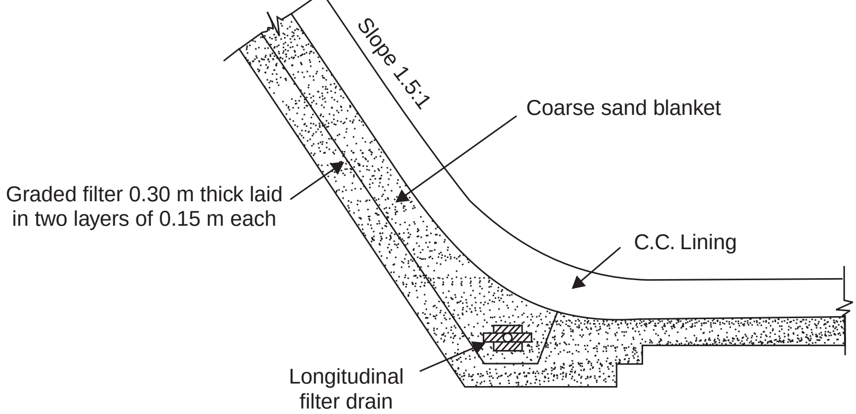

Fig. 5.24 Longitudinal filter drain

The growth of weeds on canal banks and other aquatic plant in channels may not result in failure of the lining but would affect the conveyance of channels which may be lined or unlined. Weeds and aquatic plants consume water for their growth and thus the consumptive use of irrigation water increases. Weed growth increases channel roughness and, hence, reduces the flow velocity thereby increasing evaporation losses. The cleaning of channels having excessive weed growth is, therefore, a vital maintenance problem. Cleaning operations can be carried out manually or by mechanical devices, such as used in dragline excavation and tractor-drawn cranes. Commonly used methods are pasturing, mowing, burning, and applying chemical weed killers.

5.15. DRAINAGE OF IRRIGATED LANDS

Drainage is defined as the removal of excess water and salts from adequately irrigated agricultural lands. The deep percolation losses from properly irrigated lands and seepage from reservoirs, canals, and watercourses make drainage necessary to maintain soil productivity.

Irrigation and drainage are complementary to each other. In humid areas, drainage attains much greater importance than in arid regions. Irrigated lands require adequate drainage to remain capable of producing crops. The adequate drainage of fertile lands requires the lowering of a shallow water table, and this forms the first and basic step in the reclamation of waterlogged, saline, and alkali soils. The drainage of farm lands: (*i*) improves soil structure and increases the soil productivity, (*ii*) facilitates early ploughing and planting, (*iii*) increases the depth of root zone thereby increasing the available soil moisture and plant food, (*iv*) increases soil ventilation, (*v*) increases water infiltration into the ground thereby decreasing soil erosion on the surface, (*vi*) creates favourable conditions for growth of soil bacteria, (*vii*) leaches excess salts from soil, (*viii*) maintains favourable soil temperature, and (*ix*) improves sanitary and health conditions for the residents of the area.

The water table can be lowered by eliminating or controlling sources of excess water. An improvement in the natural drainage system and the provision of an artificial drainage system are of considerable help in the lowering of the water table. A natural drainage system can be properly maintained at low costs and is a feasible method of protecting irrigated lands from excessive percolation. Artificial drainage also aims at lowering the water table and is accomplished by any of the following methods:

(*i*) Open ditch drains

(*ii*) Subsurface drains

(*iii*) Drainage wells

Open ditch drains (or open drains) are suitable and very often economical for surface and subsurface drainage. They permit easy entry of surface flow into the drains.

Open drains are used to convey excess water to distant outlets. These accelerate the removal of storm water and thus reduce the detention time thereby decreasing the percolation of water into the ground. Open drains can be either shallow surface drains or deep open drains. Shallow surface drains do not affect subsurface drainage. Deep open drains act as outlet drains for a closed drain system and collect surface drainage too.

The alignment of open drains follows the paths of natural drainage and low contours. The drains are not aligned across a pond or marshy land. Every drain has an outlet the elevation of which decides the bed and water surface elevations of the drain at maximum flow. The longitudinal slope of drain should be as large as possible and is decided on the basis of non-scouring velocities. The bed slope ranges from 0.0005 to 0.0015. Depths of about 1.5 to 3.5 m are generally adopted for open drains. The side slopes depend largely on the type of embankment soil and may vary from 1/2 H : 1V (in very stiff and compact clays) to 3H : 1V (in loose sandy formations).

The open drains should be designed to carry part of storm runoff also. The cross-section of open drain is decided using the general principles of channel design. The channel will be in cutting and the height of banks will be small. If the drain has to receive both seepage and storm water, it may be desirable to have a small drain in the bed of a large open drain. This will keep the bed of the drain dry for most of the year and maintenance problems will be considerably less. Only the central deeper section will require maintenance.

Open drains have the advantages of: (*a*) low initial cost, (*b*) simple construction, and (*c*) large capacity to handle surface runoff caused by precipitation. However, there are disadvantages too. Besides the cost of land which the open drains occupy and the need of constructing bridges across them, open drains cause: (*a*) difficulty in farming operations, and (*b*) constant maintenance problems resulting from silt accumulation due to rapid weed growth in them.

Flow of clear water at low velocities permits considerable weed growth on the channel surface. The open drains have, therefore, to be cleaned frequently. In addition to manual cleaning, chemical weed killers are also used. But, at times the drain water is being used for cattles and the weed poison may be harmful to the cattles. Aquatic life is also adversely affected by the chemical weed killers.

Subsurface drainage (or underdrainage) involves the creation of permanent drainage system consisting of buried pipes (or channels) which remain out of sight and, therefore, do not interfere with the farming operations. The buried drainage system can remove excess water without occupying the land area. Therefore, there is no loss of farming area. Besides, there is no weed growth and no accumulation of rubbish and, therefore, the underdrainage system can remain effective for long periods with little or no need for maintenance. In some situations, however, siltation and blockage may require costly and troublesome maintenance or even complete replacement.

The materials of the buried pipes include clay pipes and concrete pipes in short lengths (permitting water entry at the joints) or long perforated and flexible plastic pipes. In addition, blankets of gravel laid in the soil, fibrous wood materials buried in the soil or such materials which can be covered by the soil and which will remain porous for long time are used for the construction of underdrainage system. If such drains are to be placed in impervious soil, the drains should be surrounded by a filter of coarser material to increase the permeability and prevent migration of soil particles and blocking of drains.

Mole drains are also included as subsurface drains. The mole drains are unlined and unprotected channels of circular cross-section constructed in the subsoil at a depth of about 0.70 m by pulling a mole plough through the soil without digging a trench, Fig. 5.25. The mole plough is a cylindrical metal object (about 300 to 650 mm long and 50 to 80 mm in diameter) with one of its ends bullet-shaped. The mole is attached to a horizontal beam through a thin blade as shown in Fig. 5.26. A short cylindrical metal core or sphere is attached to the rear of the mole by means of a chain. This expander helps in giving a smooth finish to the channel surface. The basic purpose of all these subsurface drains is to collect the water that flows in the subsurface region and to carry this water into an outlet channel or conveyance structure. The outlets can be either gravity outlets or pump outlets. The depth and spacing of the subsurface drains (and also deep open drains) are usually decided using Hooghoudt's equation described in the following.

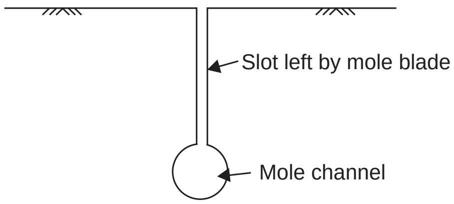

Fig. 5.25. Mole drain

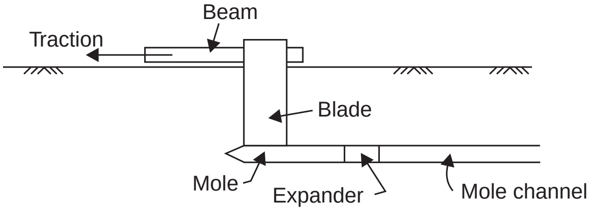

Fig. 5.26 Mole plough

Consider two drains at a spacing of B and the resulting drained water table as shown in Fig. 5.27. An impermeable layer underlies the drain at a depth d. Rainfall intensity (or rate of application of irrigation water) is uniform and is equal to r_a (m/s). Hooghoudt made the following assumptions to obtain a solution of the problem (19):

(*i*) The soil is homogeneous and isotropic,

(*ii*) The hydraulic gradient at any point is equal to the slope of the water table above the point, *i.e.*, dh/dx, and

(*iii*) Darcy's law is valid.

Using Darcy's law one can write,

$$q_x = ky \frac{dy}{dx} \tag{5.25}$$

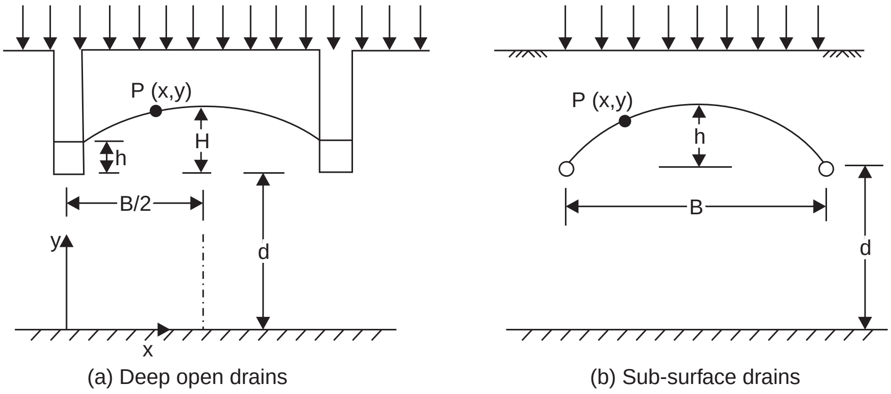

Fig. 5.27 Line sketch of drains

in which q_x is the discharge per unit length of drain at a section x distance away from the drain, and k is the coefficient of permeability of the soil. Also,

$$q_x = \left(\frac{B}{2} - x\right) r_a \tag{5.26}$$

Using Eqs. (5.25) and (5.26), one can write

$$\left(\frac{B}{2} - x\right) r_a = ky \frac{dy}{dx}$$

$$\therefore \qquad \frac{B}{2} r_a \, dx - r_a \, xdx = ky \, dy$$

On integrating,

$$r_a \frac{B}{2} x - r_a \frac{x^2}{2} = k \frac{y^2}{2} + C$$

The constant of integration C can be determined by using the boundary condition: at $x = 0$, $y = h + d$

$$\therefore \qquad C = -\frac{k(h+d)^2}{2}$$

$$r_a \frac{B}{2} x - r_a \frac{x^2}{2} = \frac{k}{2} [y^2 - (h+d)^2]$$

Further, at $x = B/2$, $y = H + d$

$$r_a \frac{B^2}{4} - r_a \frac{B^2}{8} = \frac{k}{2} [(H+d)^2 - (h+d)^2]$$

or

$$r_a \frac{B^2}{4} = k \, [(H + d)^2 - (h + d)^2]$$

$$\therefore \qquad B^2 = \frac{4k}{r_a} \, [(H + d)^2 - (h + d)^2] \tag{5.27}$$

Equation (5.27) is Hooghoudt's equation for either open ditch drains or subsurface drains. If q_d is the discharge per unit length of drain that enters the drain from two sides of the drain, then

$$q_d = r_a B$$

$$\therefore \qquad q_d = \frac{4k}{B} [(H+d)^2 - (h+d)^2] \tag{5.28}$$

In practice, the drain is considered empty (*i.e.*, $h = 0$). Equation (5.28) then reduces to

$$q_d = \frac{4k \, [H+d)^2 - (d)^2]}{B} \tag{5.29}$$

or

$$q_d = \frac{4kH}{B} (H + 2d)$$

i.e.,

$$B^2 = \frac{4kH}{r_a} (H + 2d) \tag{5.30}$$

Thus, knowing q_d, one can determine the spacing B of the drains. The design drain discharge (or the drainage coefficient which is defined as the amount of water that must be removed in a 24-hour period) primarily depends on the rainfall rate, size of the watershed, and the amount of surface drainage water that is admitted to the drainage system and is usually taken as equal to one per cent of average rainfall in one day. Thus,

$$q_d = \frac{0.01 \times r_a \times B}{24 \times 3600} \text{ m}^3\text{/s per metre length of drain.}$$

Here, r_a is the average rainfall intensity in metres, and B is the spacing of the drains in metres. The value of B is generally between 15 and 45 m. The main drawback of the gravity drainage system is that it is not capable of lowering the water table to large depths.

Drainage wells offer a very effective method of draining an irrigated land. The soil permeability and economic considerations decide the feasibility of well drainage. Drainage wells pump water from wells drilled or already existing in the area to be drained. Design of a drainage well system will be based on established principles of well hydraulics which have been discussed in Chapter 4.

The above-mentioned remedial methods can be grouped as structual measures. In addition, the following non-structual measures can also be resorted to for preventing or reducing the menace of waterlogging:

1. Adoption of tolerant crops
2. Restricting canal supplies close to crop-water needs
3. Switch over to drip irrigation
4. Conjunctive use of surface and ground water
5. Rationalization of water and power pricing policies
6. Improvement in canal irrigation management
7. Incentives for reclamation of land

In arid regions, the bio-drainage (plantation of trees having high transpiration rates) would help in controlling the rise of ground water table and soil salinity. In addition, the bio-mass so grown acts as shelter belt in light soil area against shifting sands and dunes such as in Indira Gandhi Nahar Pariyojna (IGNP) command area in which eucalyptus trees and other trees of similar species were planted. The plantation was very effective in lowering of water table (20).

Example 5.4 Determine the location of closed tile drains below ground for the following data:

Root zone depth	= 1.5 m
Capillary rise in soil	= 0.3 m
Coefficient of permeability of soil	= 1.5×10^{-4} m/s
Drainage capacity	= 0.11 $\text{m}^3\text{/s/km}^2$
Spacing of drains	= 200 m
Depth of impervious stratum below ground	= 10.0 m

Solution:

From Eq. (5.29)

$$q_d = \frac{4k\,[(H+d)^2 - d^2]}{B}$$

where $$q_d = 0.11 \times 200 \times 1/10^6 \text{ m}^3\text{/s/m}$$

$$H + d = 10 - (1.5 + 0.3) \text{ m}$$

$$= 8.2 \text{ m}$$

$$B = 200 \text{ m}$$

and $$k = 1.5 \times 10^{-4} \text{ m/s}$$

$$\therefore \quad \frac{0.11 \times 200}{10^6} = \frac{4 \times 1.5 \times 10^{-4} [(8.2)^2 - d^2]}{200}$$

$$d = 7.74 \text{ m}$$

Hence, the drains should be located at 10 – 7.74 = 2.26 m below the ground.

EXERCISES

5.1 Write a brief note on the planning of canal alignments.

5.2 What is meant by the 'duty' of canal water ? Obtain an expression for 'duty' in terms of water depth. Distinguish between 'duty' and 'outlet discharge factor'.

5.3 Describe different types of outlets mentioning their suitability for different sets of field conditions.

5.4 In what possible ways can irrigation water be delivered to various farms once it has been brought up to the watercourse ? Discuss the salient features of these methods.

5.5 How is water distribution managed in the *warabandi* system?

5.6 What are the causes of waterlogging ? How can a waterlogged land be made useful for cultivation?

5.7 An outlet is required to serve 6000 ha of CCA. Determine the discharge for which the outlet should be designed for the following data:

	Wheat	*Rice*
Intensity of irrigation	20%	10%
Kor period	3 weeks	2 weeks
Kor water depth	90 mm	250 mm

5.8 An outlet has a gross command area of 500 ha out of which only 80 per cent is culturable. The intensity of irrigation for the Rabi season is 65 per cent while it is 30 per cent for the Kharif season. Assuming losses in the conveyance system as 6 per cent of the outlet discharge, determine the discharge at the head of the irrigation channel. Assume outlet discharge factor for Rabi season as 1500 ha/m^3/s and for the Kharif season as 800 ha/m^3/s.

5.9 The maximum discharge available at an outlet of an irrigation channel is 1.33 m^3/s. The culturable command area for the outlet is 8000 ha. What percentage of this area can be irrigated for wheat if the *kor* period is 3 weeks and the *kor* water depth is 13.5 cm?

5.10 Closed drains at a spacing of 16 m are located 2 m below the ground surface and the position of the water table is 1.7 m below the ground surface. Find the discharge carried by a drain if the coefficient of permeability of the soil is 2×10^{-2} cm/s and the depth of the pervious stratum is 8 m.

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6

MANAGEMENT OF CANAL IRRIGATION IN INDIA

6.1. NEED FOR CANAL IRRIGATION MANAGEMENT

Because of the limited availability of utilisable water (see Tables 1.4 and 1.5) and the growing need of water for irrigation and other purposes (see Table 1.6), the need for managing the water resources can hardly be overemphasized. Irrigation management assumes special importance in view of the fact that irrigation alone needs about 75 per cent of the total water requirement (see Table 1.6). The objective of irrigation management is to supply and apply the right amount of water at the right place and at the right time. It was estimated that irrigation projects were running at an annual loss of about Rs. 400 crores in the early 1980's. This figure rose to Rs. 800 crores in the mid–1980's. Examination and analysis of the performance of canal irrigation systems in India in the early 1980's revealed many deficiencies in the operation and management of canal irrigation systems, especially those constructed in the previous three decades (1). Table 1.3 indicates the disparity in productivity of canal and tubewell irrigated lands.

In the absence of data on employment and livelihood as a result of introduction of an irrigation system, the performance of irrigation systems is gauged by the following (1):

(*i*) Area irrigated as proportion of the area planned to be irrigated,

(*ii*) Waterlogging,

(*iii*) Tail-end deprivation, and

(*iv*) Yield.

The ratio of the area irrigated to the area planned to be irrigated in some irrigation projects of India are tabulated in Table 6.1. It establishes that the performance in area actually irrigated has fallen far short of that projected.

A land is said to be waterlogged when the pores of soil within the root zone of plants get saturated and the normal growth of the plant is adversely affected due to insufficient air circulation. Poorly-practised canal irrigation is the main cause of waterlogging wherever it occurs. The Irrigation Commission (1972), the National Commission on Agriculture (1976), the Soil and Salinity Research Institute (1981) and the Central Ground Water Board (1982) have separately and at different times assessed the amount of waterlogging areas in the country (Table 6.2).

Table 6.1 Performance of some irrigation projects in terms of area irrigated (1)

Project (Ref.)	*Period*	*Ratio of area irrigated to area planned to be irrigated*
Rajasthan Canal (1)	1980's	0.50
Sarda Sahyak Project (1)	1980–81	0.38
Kosi Canal (2)	1971–76	0.31
Krishna-Cauvery Canal (3)	1980–81	0.88
Tungabhadra High Level Canal (3)	1980–81	0.81
Tungabhadra Low Level Canal (3)	1980–81	0.78
Kaddam (3)	1980–81	0.65
Rajolibanda Diversion Scheme (3)	1980–81	0.60
Nagarjunasagar Left Bank Canal (3)	1980–81	0.42
Nagarjunasager Right Bank Canal (3)	1980–81	0.37
Sri Ram Sagar Project (3)	1980–81	0.17
Perambikulam Aliyar Project (4)	1976–81	0.47

The variation in the assessment of waterlogged area from 3.35 to 6.0 Mha may be due to differences in criteria and methods used. Areas affected by waterlogging and salinity are quite large in the command area of Kosi and Gandak Projects in Bihar, Mahi-Kadana Project in Gujarat, Chambal Project in Rajasthan, Nagarjunasagar Project and Sri Ram Sagar Project in Andhra Pradesh, Tungabhadra Project in Karnataka and Andhra Pradesh, and Sarda Sahayak Project in UP. It has been reported that following the extension of the Sarda Sahayak Project in UP, an expenditure of Rs. 384 crore had added 4 lakh hectares of irrigation area but with a loss of 5 lakh hectares of irrigated area due to waterlogging (1). All these facts indicate that waterlogging is a serious problem in India at levels of irrigation management prevalent in the mid–1980's.

Tail-end deprivation is present in almost every canal irrigation project and is reflected in terms of water supply, irrigation intensity, crops grown, cultivation practices, yields, and incomes. Tail-end deprivation of water supply and irrigation intensity is usually more acute on new projects. For example, on minors in the Hirakud project, 70 per cent of the irrigation water went to the upstream halves with only 30 per cent left for the downstream halves (1). Similarly, in 1981, farmers of the upper reaches of the new Sarda Sahayak command area got five irrigations against only one irrigation received by farmers of the tail-end reaches of the same command area. Some examples of tail-end deprivation of irrigation intensity are given in Table 6.3 for new as well as old irrigation projects. Besides, higher-valued and more water-intensive crops are usually concentrated in head reaches. For example, sugarcane intensity was 44 and 10 per cent, respectively, at the head reach and the tail-end reach of Left Salawa distributary of Upper Ganga canal (6). Crop yields and farm income usually decline from head reach to tail-end reach. However, sometimes they can be higher in the middle reaches (especially if the head reach is waterlogged) or even in the tail-end reaches if ground water is used extensively. Yields and incomes in the head reach and the tail-end reach of a minor of Gambhiri Project are shown in Table 6.4.

Table 6.2 Waterlogged areas in India (5)

S. No.	*State*	*Area under waterlogging (1000 ha) (depth of water within 2m)*			
		Soil and Salinity Research Institute (1981)	*Irrigation Commission (1972)*	*NCA (1976)*	*CGWB (1982)*
1.	Andhra Pradesh	4.0	–	339.0	725.0
2.	Assam	–	–	–	445.0
3.	Arunachal Pradesh	–	–	–	38.0
4.	Bihar	–	–	117.0	178.0
5.	Delhi	1.0	–	1.0	–
6.	Gujarat	484.0	–	484.0	150.6
7.	Haryana	1427.0	651.0	620.0	82.0
		(including Punjab)			
8.	Jammu and Kashmir	10.0	–	10.0	178.0
9.	Karanataka	–	6.6	10.0	
10.	Kerala	–	–	61.0	1.4
11.	Madhya Pradesh	57.0	57.0	57.0	28.8
12.	Maharashtra	111.0	1.6	111.0	273.2
13.	Mizoram	–	–	–	–
14.	Manipur	–	–	–	–
15.	Meghalaya	–	–	–	56.8
16.	Nagaland	–	–	–	–
17.	Orissa	–	–	60.0	372.0
18.	Punjab	(Included in Haryana)	1090.0	1097.0	250.8
19.	Rajasthan	347.0	347.6	348.0	123.8
20.	Tamil Nadu	–	–	18.0	15.0
21.	Uttar Pradesh	686.0	810.0	810.0	351.3
22.	West Bengal	1309.0	388.5	185.0	151.5
	Total	4436.0	3353.0	5986.0	3422.5
		4.44 Mha	3.35 Mha	6.0 Mha	3.42 Mha

Table 6.3 Tail-end deprivation of irrigation intensity (1)

Name of the Channel	*Intensity of irrigation (in per cent)*	
	Head reach	*Tail-end reach*
Ghatampur distributary of Ramganga Project	155	22
Pardhipali Subminor	93	24–46
Old Sarda Canal	42	19
Left Salawa Distributary on the Upper Ganga Canal	119	68

Table 6.4 Yields and income in the head reach and the tail-end reach of a minor of Gambhiri Project (7)

	Yield in kg per hectare	*Net income in Rs. per 100 kg*	*Net income in Rs. per hectare*
Thikaria Minor (Head Reach)	2500	142	881
Rithola Minor (Tail-end)	1400	41	144

Another measure of poor performance of irrigation projects is the average yields which have been usually less than the potential. For example, during 1980–81, wheat yields of Gambhiri Project were only 1850 kg per hectare against 3500 kg per hectare on the experimental farm (7).

From these examples it is obvious that the performance of most of the irrigation projects in India is very poor. However, one should also keep in view the fact that the projections made at the time of planning and design are often unrealistically optimistic. Such projections are made to achieve the desired returns and to pacify political interests demanding water in places which could not be served. Even the so-called physically feasible potentials can never be fully achieved in the actual situation as regards canal systems which have so many constraints. Nevertheless, there is considerable scope to improve the performance of ongoing projects as well as future ones through improved irrigation management.

6.2. INADEQUACIES OF CANAL IRRIGATION MANAGEMENT

From the point of view of performance, the management of the canal irrigation systems in India is far from satisfactory. The major inadequacies are as follows (1, 8 and 9):

(*i*) Insufficient planning and preparation at the stage of execution of the project which results in longer construction time and escalated project cost,

(*ii*) Involvement of more than one ministry/department and poor coordination among them,

(*iii*) Non-responsive, authoritarian, and poor administration resulting in increased malpractices,

(*iv*) Lack of interaction between engineering and agricultural experts,

(*v*) Lag between creation of potential and its utilisation,

(*vi*) Improper assessment of personnel, equipment, and other facilities for proper operation and maintenance of reservoirs and canal systems resulting in erratic (unreliable and insufficient) supplies and inequitable distribution of available water,

(*vii*) Higher conveyance losses,

(*viii*) Absence of conjunctive use of ground water and surface water,

(*ix*) Insufficient drainage, excessive seepage, and waterlogging,

(*x*) Poor on-farm management,

(*xi*) Absence of farmer's participation in the management,

(*xii*) Lack of communication facilities in the command area,

(*xiii*) Poor extension services – lack of pilot projects, demonstration farms, etc., (*xiv*) Problems related to land settlement and rehabilitation of displaced persons, and

(*xv*) Recovery of the project cost.

6.3. OBJECTIVES AND CRITERIA OF GOOD CANAL IRRIGATION MANAGEMENT

There are several conventional measures to improve the performance of canal irrigation systems. Some of these measures are lining of canals and field channels, on-farm development, farmers' organisation, *warabandi* system of water distribution, charging farmers volumetrically for water, and educating farmers in water use management. However, before seeking a solution to improve the irrigation management, it is worthwhile to consider the objectives of irrigation and the criteria for judging the performance of an irrigation project.

The effects or impacts of irrigation can be best phrased as "optimising human well-being" (1). The term "well-being" includes food security, incomes, nutritional status, health, education, amenity, social harmony, and self-respect.

The criteria for judging the performance of canal irrigation systems can be vastly different for different groups of people depending upon their concerns (Table 6.5). However, the most common criteria generally accepted for judging the performance of an irrigation system are productivity, equity, and stability which together contribute to the objective of well-being (1).

Table 6.5 Criteria of good irrigation system performance (1)

Type of person	*Possible first criterion of good system performance*
Landless Labourer	Increased labour demand, days of working, and wages
Farmer	Delivery to his or her farm of an adequate, convenient, predictable, and timely water supply for preferred farming practices
Irrigation engineer	Efficient delivery of water from headworks to outlet
Agricultural engineer	Efficient delivery and field application of irrigation water from the outlet to the root zone of the crop
Agronomist	Creation and maintenance of the optimum moisture regime and plant growth and, in particular, maximising production of that part of the plant which is the harvestable product
Agricultural economist	High and stable farm production and incomes
General economist	A high internal rate of return
Political economist	Equitable distribution of benefits, especially to disadvantaged groups
Sociologist	Participation of irrigators in management

6.3.1. Productivity

Productivity is defined as the ratio of output and input. The output can be water delivered, area irrigated, yield, or income, and the input can be water in the root zone, at the farm gate, at the outlet or at upstream points in the system including the point of diversion or storage (10). Fig. 6.1 shows typical points of input and output measurements for different professionals (1). Improved water supply influences the adoption of high-yielding agricultural practices by farmers which justifies the productivity criterion of performance.

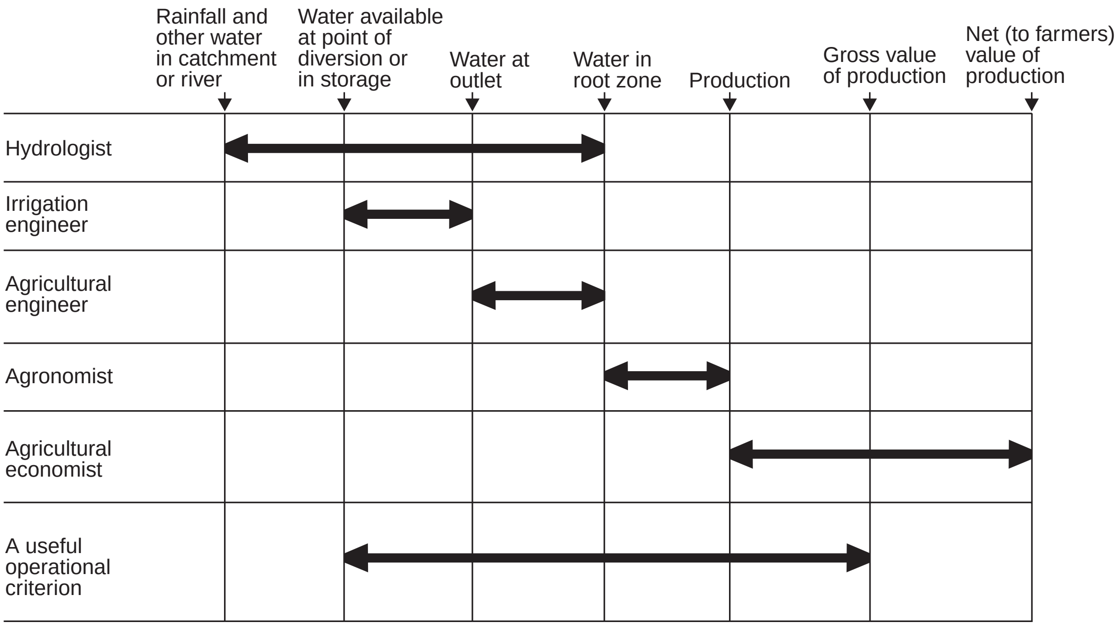

Fig. 6.1 Typical points of input and output measurements of productivity for different professions (1)

6.3.2. Equity

Equity in canal irrigation systems implies equality, fairness, and even-handed dealing in matters of allocation and appropriation of irrigation water (1). There can be several ways to decide the equality of supplies to different farmers. Two of them, practised throughout the world, are the methods of prior appropriation and of proportionate equality. In the method of prior appropriation, whoever first exploits a resource establishes a right to continue to do so. Thus, head reach farmers and early comers to an irrigation project establish their right over irrigation water, even if it means less water or no water to tail-end farmers or late comers. In the second method of proportionate equality, the supply of water is in proportion to the size of the land-holding as in the *warabandi* system of north-west India (11). In India, on most canal irrigation systems, water distribution is far from meeting even the criterion of water proportional to land and, hence, both these methods have been criticised for their inequity (1). Attempts to improve equity are usually limited to achieving the supply of water in proportion to the size of the land.

6.3.3. Stability

Stability of productivity as well as equity are important. Stability can either be short-term stability or long-term sustainability. The short-term or interseasonal stability refers to the variations in productivity and equity between irrigation seasons, and is a function of climate, water supply, storage and control, system management, and other factors such as pests, diseases, and availability of labour and other inputs. It can be measured by comparing performance between seasons. The long-term sustainability has been described as "environmental stability" (12) and "durability" (13) and refers to the prevention or minimising of adverse physical changes such as waterlogging, leaching of nutrients from soils, salinity, erosion, silting, the 'mining' of ground water, and infestations with weeds (1). Sustainability can be monitored by measuring ground water levels, salinity, erosion, or silting through inspecting works, and by measuring long-term trends in productivity and equity.

Well-being is a broad objective achieved through productivity, equity, and stability. There are several aspects of well-being which must be borne in mind at all stages of the project with both positive and negative effects. These aspects include health, nutrition, amenity (especially water for washing and bathing, raising ground water for domestic purposes, and so on), and psychological factors such as freedom from domination, feeling of participation, etc.

6.4. METHODS FOR IMPROVING CANAL IRRIGATION MANAGEMENT

Irrigation management is an interdisciplinary system process with a built-in learning mechanism to improve system performance by adjusting physical, technological, and institutional inputs to achieve the desired levels of output (14). Canal irrigation is a complex process involving physical, bio-economic, and human activities which are interrelated and vary widely over space and time. As such, canal irrigation management demands special methods. Every management problem requires to be analysed in detail and then solved accordingly. Nevertheless, there are some aspects which, if considered properly at different stages, can help significantly in the improvement of canal irrigation. These aspects have been briefly dealt with in the following.

6.4.1. Cropping Patern

Cropping pattern is described in terms of the area under various crops at different periods of a year. An optimum cropping pattern for an area can ideally be determined by using systems analysis. If the local preferences and requirements of the area are included in the analysis, and the necessary inputs are made available, the farmers will adopt the cropping pattern arrived at using systems analysis. One such analysis carried out for the Gomti-Kalyani *doab* under Sarda Sahayak command has recommended a cropping pattern which is expected to increase the net annual benefit to Rs. 23.67 crore from the existing benefit of Rs. 11.57 crore (15).

6.4.2. Conjunctive Use

Often in the past, the development of water resources has taken place in such a manner as if surface water and ground water were two separate sources. Successful management of water resources requires adding to the two-dimensional development of surface water the third dimension of depth to include ground water.

Conjunctive use means that water lifted from below the ground is used in conjunction with canal waters. It results in the coordinated, combined, and creative exploitation of ground water and surface water so as to minimise the dislocation caused by nature's inconsistent rainfall pattern (16). Conjunctive use implies use of surface water (from either reservoir storage or diversion works) during periods of above normal precipitation for irrigation and other activities to the extent possible and letting the balance reach the ground water storage (through artificial recharge) which would be utilised for supplementing surface water supplies during years of subnormal precipitation. Such coordinated use of surface and ground waters results in increased amount of available water, smaller surface distribution system, smaller drainage system, reduced canal linings, greater flood control, and smaller evaporation losses. There are, however, some disadvantages too in resorting to conjunctive use. These are lesser hydroelectric power, greater power requirement, need for artificial recharge, and danger of land subsidence. The parameters related to conjunctive use, such as cropping pattern, canal capacities, capacities and spacing of wells, drainage requirements, optimum ground water level, etc. are best determined by systems analysis to derive maximum benefits.

While studying the available water resources and the original plans of Mahi-Kadana project of Gujarat which did not include extensive use of ground water, Sarma *et al.* (17) proposed the conjunctive use of water. Their calculations, based on a culturable command area of 213,000 ha, indicated that the intensity of irrigation could be raised from 55 per cent (achieved in 1980–81) to 180 per cent through conjunctive use. Besides, the rise in the ground water table would also be arrested.

6.4.3. Channel Capacity

The discharge capacity of the channel system should be decided on the concept of evapotranspiration rather than the '*kor*' period. This has already been discussed in Secs. 3.8 and 5.8.

6.4.4. Canal Lining

Lining of canals is a means to reduce the seepage losses from canals. In one typical case, the benefit-cost ratios of lining distributaries only, distributaries and watercourses, and field channels only were found (18) to be, respectively, 0.33, 0.608 and 2.303. As such, the lining of field channels is the most beneficial. Besides, it involves no dislocation in the operation of an existing system. In order to prevent damage to lining, the slope of a lined channel is reduced. This reduces the sediment carrying capacity of an existing channel which is being lined. Therefore, measures for sediment exclusion are to be considered whenever an existing canal is being lined. Alternative to the lining of canals is the conjunctive use of surface and ground water which should be opted for after comparing the unit cost of water saved by lining with the unit cost of pumped water. For a representative case, Chawla (16) has worked out the cost of water saved by lining as Rs. 25 for every 100 m^3 of water against Rs. 7 to Rs. 12 for the same amount of pumped water.

6.4.5. Regulators and Escapes

For ensuring proper distribution of irrigation water according to the adopted management policy, a suitable number of canal regulators and canal escapes must be provided on the channel network in general and on main canals and branches in particular. Canal escapes are needed for the safety as well as for regulating canal supplies in areas which have received excess rainfall.

6.4.6. Canal Outlets

Another important aspect of designing canal irrigation system is the selection of suitable type of outlet which is crucial in controlling the distribution of water and providing a link between the administration and the farmer. From the considerations of equitable distribution of water, a regulated outlet would be an ideal choice provided that it can be operated efficiently and honestly. Unfortunately, the present socio-political conditions prevailing in the country, however, do not permit such operation (8). As such, for the present, regulated outlets are ruled out. The subproportional semi-modules would be the right choice for locations just upstream of falls and regulators and also on branch canals (8). Other outlets may be of non-modular type.

6.4.7. Main System Management

Operational management of the main system refers to management aspects of the future allocation, scheduling, delivery of water on main systems down to and including outlets, and the disposal of water in drains below *chaks* (*i.e.,* the irrigated fields) (1). It includes planning, decision making, the operation of controls, and communications both upwards to managers and downwards to groups of farmers so that equitable supplies can be ensured throughout the command area (1). Main system management (MSM) is capable of reducing gross inequities of water supply to tail-end farmers and increasing the farm yield from the command area. A suitable type of MSM can induce the farmers for active involvement in on-farm development and maintenance and adoption of high-yielding practices. The importance of MSM is more in reservoir-based irrigation systems than in the systems based on diversion schemes because of the available options of storage and release.

According to the present system of operation, the main canal and branches run continuously with either full or reduced supply depending on the availability of water. The distributaries and minors, however, run intermittently in accordance with the keenness of demand as assessed by the concerned field staff. Such assessment tends to be subjective as well as approximate. A more rational method for the running schedule of distributaries and minors can, alternatively, be worked out as follows (8):

(*i*) Obtain the cropping pattern, preferably an optimum one, for irrigation during the ensuing season,

(*ii*) Estimate weekly evaportranspiration and corresponding effective rainfall based on past records,

(*iii*) Determine the irrigation demand,

(*iv*) Decide upon the amount of canal water and ground water to meet the irrigation demand such that the desired intensity of irrigation on the optimum cropping pattern can be obtained along with a stable water table, and

(*v*) Prepare a roster of regulation of distributaries and minors and notify the concerned farmers well in advance to enable them to plan their sowing and irrigation programmes accordingly.

Provision for departure from the notified roster should be available. Such departures should be made only if there is variation in rainfall pattern.

6.4.8. Night Irrigation

Another important issue in the operation of the main system is related to night irrigation. In most of the canal irrigation projects, the canal water continues to flow at nights as well and is either badly used or wasted. Darkness, cold, fear, normal working hours and desire for sleep discourage the irrigation staff, farmers, and labourers to work at night. In India, about 47 per cent of the 24 hours day is the time of darkness (between sunset to sunrise with an allowance of 20 minute twilight period each after the sunset and before the sunrise). According to Chambers' approximate estimate, about 40 per cent of the canal irrigation water on medium and major systems (*i.e.,* with commands over 2000 ha) is either applied in night irrigation or sent into drains at night (1). He further estimates that, except in north-west India where *warabandi* is practised at night, 25 per cent of the canal irrigation water is wasted at night and much of the 15 per cent which is applied at night is used inefficiently. In view of the magnitude of these losses, night irrigation is too important to remain neglected any more. Farmers usually dislike night irrigation for the following reasons (1):

(*i*) Loss of sleep and disruption in the normal sleeping duration,

(*ii*) Discomfort due to cold night and difficulty in moving around in sticky soils and mud, (*iii*) Danger and fear of snakes, scorpions, accidents, violence including murder, and other problems related to law and order,

(*iv*) Inefficient application of water due to darkness, and

(*v*) Higher costs due to higher night wages, non-availability of family labour, especially women, old people and those very young to work at night, and need of firewood, beverages and lighting.

In addition, field conditions, type of crop, and its stage of growth may add to the difficulties of irrigating at night. Sloping lands with difficult soils and standing crops are difficult to be irrigated at night. Paddy, trees, and crops which are low and wide apart or are in early stages of growth can be irrigated with relative ease. High and dense crops, and crops in the later stages of growth are relatively difficult to be irrigated at night. The difficulties in night irrigation may influence a farmer's choice of crop if he has to rely on only water received at night.

However, sometimes night irrigation is preferred by farmers due to the following reasons (1):

(*i*) In warmer regions, farmers find it more comfortable to irrigate at night.

(*ii*) Part-time farmers having other work during the day would prefer night irrigation.

(*iii*) Tail-end farmers may get relatively more adequate and reliable supply during night.

In addition to the above characteristics of night irrigation, the following features must also be noted (1):

(*i*) It is during the night that illicit appropriation of water (breaching of bunds, removing checks, blocking streams, opening pipes, pumping out of channels etc. to secure water for an individual or a group) takes place.

(*ii*) During nights, evaporation losses are much less. This saving is, however, more than offset by wastage.

(*iii*) Most physical damages such as erosion in steep minors, watercourses and field channels due to relatively larger flows on account of lesser withdrawals upstream, and aggravation of waterlogging and associated soil salinity and alkalinity problems seem to occur at night.

It is, therefore, obvious that night irrigation presents many problems. This situation can be improved either by reducing irrigation at night or improving it. Reduction in night irrigation can be either with or without saving water for subsequent use. Measures to reduce night irrigation with water saving involve storage of water which can be achieved in five different ways – in main reservoirs, in canals, in intermediate storage reservoirs, on-farm storage, and in ground water storage. On the other hand, three obvious measures to reduce night irrigation without saving water are: (*i*) stopping river diversion flows, (*ii*) redistributing day water so that even tail-enders get adequate supplies, and (*iii*) passing water to escapes and drains. For obvious reasons, measures with water saving are more desirable but much needs to be done to make them practicable. Night irrigation can be improved in the following ways (1):

(*i*) Making flows predictable and manageable. For this purpose, the *warabandi* system has obvious advantages. Each farmer knows the time of his turn and does not need to recruit labour to capture and guard his supply.

(*ii*) Improving convenience and efficiency by measures such as (*a*) good lighting, (*b*) organisation of groups of farmers for night irrigation for mutual benefits of shared labour and common protection from water raiders, etc., (*c*) installing and maintaining structures, channels, fields and water application methods requiring minimum observation and adjustment, (*d*) shaping fields for easier water application, and (*e*) providing ways and means for easy movement from residences to the fields and within the fields.

(*iii*) Choosing easy crops such as paddy and trees and crops which are low and wide apart.

(*iv*) Zoning for night flows which means using day flows for more difficult crops and soils and night flows for relatively easier crops and soils.

(*v*) Phasing night irrigation for shorter, warmer, and moon-lit nights would, obviously, be more convenient for farmers.

It should, however, be noted that night irrigation had not received any attention till recently. Only a beginning has been made and lot more needs to be done to mitigate the problems related to night irrigation.

6.4.9. Water Delivery System

Water delivery systems can be of three types:

(*i*) Demand-based,

(*ii*) Continuous, and

(*iii*) Rotational (also known as *warabandi*).

Of these three, the *warabandi* system (Sec. 5.11) seems to be the most feasible and offers many advantages. It has been defined (19) as a system of equitable water distribution by turns according to a predetermined schedule specifying the day, time, and duration of supply to each irrigator in proportion to land holdings in the outlet command. Both the Irrigation Commission of 1972 and the National Commission on Agriculture of 1976 saw *warabandi,* with fixed times but taking water throughout the 24 hours, as a means of tackling waste of water at night (1). The procedure for estimating the entitlement of a farmer varies in details. For example, in UP and north-western states, the entitlement is based only on the area of the farm whereas in Maharashtra and other southern states, the farmer has to seek approval for irrigation of specific areas for specific crops (8). Some studies have indicated that the *warabandi* system, although efficient and acceptable to the farmers, does not result in equitable distribution primarily due to the losses in the watercourses. These losses may cause about 25 to 40 per cent reduction in the share of water of the farmer in the tail-end reach. While determining the schedule, some weightage should be given to the time allotment for tail-enders to compensate for the losses.

Another promising system of water delivery is through water cooperatives which purchase water in bulk and then distribute it among their member farmers. One example of this distribution is the Mohini Water Cooperative Society near Surat which is considered the first successful irrigation cooperative in Gujarat (20).

6.4.10. Irrigation Scheduling

For efficient management of an irrigation system, it is necessary that the water be supplied to the plants when they need it and in quantities actually required by the plants. This necessity leads one to irrigation scheduling which means estimating the starting time, stopping time, and the quantity of water for different cycles of irrigation during the crop period. Irrigation scheduling can be determined by using one of three approaches, *viz.*, (*i*) the soil-moisture depletion approach, (*ii*) the climatological approach using evaportranspiration and effective rainfall data, and (*iii*) the farmer's existing schedule approach. A study on optimal irrigation scheduling for wheat crop of Udaipur region (21) has indicated that the soil-moisture depletion approach results in maximum water use efficiency.

6.4.11. Irrigation Methods

Most of the surface irrigation methods (Sec. 3.10) yield reasonably high field application efficiency provided the land has been prepared properly and due care has been taken during irrigation. However, the sprinkler method of irrigation and the drip irrigation method seem to be more promising than others in most of the conditions. The methods, however, require much higher initial investment, energy for generating pressure, and silt-free water. If the cost of land preparation and the percolation losses are high, sprinkler irrigation may result in considerable saving of money as well as water. The average cost of a sprinkler irrigation system may be approximately Rs. 15,000 per hectare and can possibly be recovered in about 2 years' time (22). The drip irrigation method is highly efficient and better suited for fruit crops, vegetables, and cash crops like sugarcane, cotton, groundnut, *etc*. (23). The cost of drip irrigation system may be around Rs. 30,000 per hectare (22). Table 6.6 compares different irrigation methods and clearly shows the superiority of sprinkler and drip irrigation systems over the surface methods (*i.e.,* flooding, check, basin, border strip, and furrow methods).

Table 6.6 Comparison of suitability of irrigation methods (22)

Site factor	*Surface Method*	*Sprinkler Method*	*Drip Method*
Soil	Uniform with moderate to low infiltration	All	All
Topography	Level to moderate slopes	Level to rolling	All
Crops	All	Tall crops limit type of system	High Value
Water supply	Large streams	Small stream nearly continuous	Small stream continuous and clean
Water quality	All but very salty	Salty water may harm plants	All
Efficiency	50-80%	70-80%	80-90%
Labour requirement	High	Low to moderate	Low to high
Capital requirement	Low to high	Moderate to high	High
Energy requirement	Low	Moderate to high	Low to moderate
Management skill	Moderate	Moderate	High
Weather	All	Poor in windy conditions	All

In India, sprinkler as well as drip irrigation have considerable scope because of the need to save water and extend irrigation facility to as large a cropped area as possible for producing food for the growing population. Sprinklers must find useful application in undulating sandy terrains of Rajasthan, Gujarat, Haryana, and Punjab. Drip irrigation is ideal for fruit orchards vegetable crops, and some cash crops. Water saved due to the introduction of these two methods in favourable regions may be enough to increase the irrigation potential for additional 5 million hectares (8).

6.4.12. Use of Waste Water

By AD 2000, the requirement of domestic water needs and thermal power plant needs will be around 3 Mha.m most of which will be used for non-agricultural purposes (8). It would, obviously, be very beneficial even if half of this used water is suitably treated and used for irrigation. Such measures can provide additional irrigation potential for about 1.5 to 2 Mha of cropped land (8).

6.4.13. Conservation of Water on the Field

Rice fields have to be kept flooded for a sufficiently long time, and this results in large percolation losses from 50 to 80 per cent depending upon the type of soil (22). Therefore, rice cultivation should normally be restricted to soils of relatively low permeability. The percolation losses can also be reduced by puddling the soil using improved puddlers and the saving of water can be between 16 and 26 per cent depending upon the type of soil and puddlers used (22).

6.4.14. Waterlogging

Waterlogging (Sec. 5.14) results in lowered yields, loss of lands for useful activities, and health hazards. To eliminate or control waterlogging one or more of the following, remedial measures have usually been used (1, 8):

(*i*) Reducing inflow to the ground through lining of canals,

(*ii*) Removing ground water through pumping,

(*iii*) Removing surface and ground waters through drainage,

(*iv*) Educating farmers in water management, and

(*v*) conjunctive use.

Of all these methods, the conjunctive use of surface and ground water is the most cost-effective means of fighting waterlogging in canal-irrigated lands. This has already been effectively tried in parts of western UP, Haryana, and Punjab (8). Waterlogging can also be reduced by supplying less water during nights (as was done on the head reach of the Morna system in Maharashtra), cutting off water supplies during rains, rotating supplies in distributaries and minors instead of continuous supply, shortening irrigation periods, and zoning for crop type (1).

6.4.15. Soil Reclamation

Saline soils are found in the states of Madhya Pradesh, Rajasthan, Maharashtra, Karnataka, Andhra Pradesh, West Bengal, Tamil Nadu, and Gujarat. Alkaline soils are found in the Indo-Gangetic plains of Punjab, Haryana, UP, and parts of Bihar and Rajasthan. Because of their adverse effects on agricultural production as well as magnitude, saline and alkaline soils need to be reclaimed on a high priority basis and in a planned manner by the joint efforts of agricultural chemists, agronomists, agricultural experts, and irrigation engineers. The role of an irrigation engineer is important in lowering the water table if it is high and also providing irrigation water of good quality for leaching out the salts.

For reclamation works, one needs to know the following before taking any remedial measure (8):

(*i*) Characteristics of the soil and the salts present in it,

(*ii*) Availability and quality of irrigation water,

(*iii*) Level of ground water table, and

(*iv*) Crops which can be grown under given conditions.

The main components of soil reclamation works are as follows (8):

(*i*) Isolation of land areas according to their categorisation and levelling and bunding of the affected land as per the category.

(*ii*) Provision of drainage (surface or subsurface or vertical) network to remove leaching water and to keep the water table to a safer level.

(*iii*) Breaking up of impervious subsoil layer in alkali soils by deep ploughing.

(*iv*) Adding suitable chemicals (such as gypsum, sulphur, *etc*.) depending upon the results of chemical tests of the affected soil.

After application of the chemicals, the leaching is carried out by four to five applications of good quality water up to depth of about 60 cm.

If the reclaimed soil is not suitable for foodgrain production, it can be used to grow certain species of trees such as *safeda* (hybrid eucalyptus), *vilayati babul* (*prospis jiliflora*), and indigenous *babul* (*acacia milotica*). If the soil has been reclaimed to the extent that it can be used for foodgrain production, it is usual to grow paddy as the first crop in Kharif and barley or wheat in the next Rabi and *dhaincha* in summer as a green manure.

6.4.16. Water Charges and Pricing of the Agricultural Output

In India, canal water charges are generally fixed on the basis of area served and the crop irrespective of the volume of water supplied. Such a method of charging, obviously, leads to inefficient use of water. Due to the large number of small farmers, however, it is impractical to meter the supplies. An alternative, in the form of cooperative societies receiving the supplies in bulk and managing internal distribution and collection of charges from its members, may possibly result in efficient use of water. One such society (the Mohini Water Cooperative Society near Surat in Gujarat) has been operating successfully in a sugarcane belt of Gujarat.

Typical values of water rates have been given in Table 6.7. On the basis of a study of pricing policy taking into account the farmer's paying capacity, risk factor and social objective of income distribution, Padhi (24) has recommended water rates given in Table 6.8. The existing water rates have been fixed after allowing considerable amount of subsidy. World Bank and some economists suggest increase in irrigation charges on the principle of cost recoveries from beneficiaries. However, it should be noted that the procurement prices of foodgrains are calculated on the basis of the subsidised inputs. As such, it is the consumer who is ultimately benefited due to the subsidised rates and not the farmer. In fact, the farmer's economic condition seems to have worsened as can be seen by the values of parity indices (with base value 100 for 1954–55) given in Table 6.9.

Table 6.7 Canal water rates (in rupees per hectare) for some major crops in different states of India (8)

Sl. No.	*Name of State*	*Rice*	*Wheat*	*Cotton*	*Sugarcane*	*Garden Orchards*
1.	Andhra Pradesh	74.13	49.42	49.42	74.13	–
2.	Bihar	40.76 to 77.80	33.45 to 44.45	–	122.27 to 211.19	25.92 to 40.76
3.	Gujarat	74.13 to 123.55	74.13 to 103.78	–	585.65 to 850.05	–

(Contd.)...

4.	Haryana	49.42	19.77	29.65	49.42 to 98.84	61.78
5.	Karnataka	49.42 to 74.13	44.48	44.48	197.69 to 296.53	88.96
6.	Madhya Pradesh	39.54 to 59.31	37.06 to 49.42	39.54	98.80 to 148.20	98.84
7.	Maharashtra	50.00 to 100.00	75.00 to 150.00	250.00	750.00	–
8.	Orissa	59.31	22.23	37.06	66.69	44.46
9.	Punjab	48.19 to 48.82	13.59 to 29.13	32.54 to 38.92	65.72 to 81.55	15.86 to 51.40
10.	Rajasthan	34.60 to 61.78	29.65 to 37.06	49.42 to 61.78	74.13 to 86.49	83.93
11.	Tamil Nadu	3.71 to 61.79	9.98 to 61.78	3.71 to 49.42	11.12 to 74.13	7.41 to 74.13
12.	Uttar Pradesh	41.83 to 95.84	44.48 to 99.84	9.88 to 39.54	34.69 to 197.69	14.38 to 99.84

Table 6.8 Canal water rates (in rupees per hectare) as recommended by Padhi (24)

Sl. No.	*Name of Crop*	*Net income from irrigated crop*	*Ability to pay*	*Price to be charged*		
				Large farm (8.9 ha)	*Medium farm*	*Small farm (0.77 ha)*
1.	Wheat	1210	337	368.70	292.30	207.60
2.	Oilseeds	885	147	160.80	136.40	92.40
3.	Pulses	935	108	216.60	183.70	122.50
4.	Potato	1950	610	667.30	529.00	378.00
5.	Gram	1050	290	317.30	251.50	179.90
6.	Sugarcane	1805	419	458.40	363.80	259.70

Table 6.9 Parity indices (25)

Year	*Index of prices*		*Parity index with respect to 1954–55 base of 100*
	Paid by farmers	*Received by farmers*	
1980–81	471.8	459.3	97.3
1981–82	770.0	607.0	78.8
1982–83	846.5	536.2	63.3
1983–84	933.6	677.2	72.5

6.4.17. Command Area Development Programme

As per the recommendations of the Irrigation Commission, Command Area Development Authorities (CADA's) (Sec. 1.8) were set up for the coordinated and expeditious development of command areas under medium and major projects. This programme was to include systematic programming of land consolidation, the scientific land shaping, construction of watercourses and field channels to carry water to individual fields, field drains to carry surplus water away from field, and a system of roads which will enable farmers to carry the produce to the market. Besides the above measures, adequate and timely supply of inputs (fertilisers, seeds, credit and pesticides) was to be ensured and marketing and other infrastructure facilities were to be created so that the farmers would be able to derive optimum benefits from available land and water (26). There is, however, no mention of adequate and timely supply of water which, in fact, has been implicitly assumed to be available (1). Further, in most states, CADA's jurisdiction was confined to the downstream of the outlet. However, better main system management is the precondition for effective functioning of CADA., as is farmer's participation and willingness to undertake on-farm development programmes. A detailed study of conditions downstream of the outlets of the Mahi-Kadana project (27) indicated that, "the wide variations in flow rate at the field level make it essentially impossible for farmers to control the application of water to a graded border. In general, the result was excess irrigation for some fields and deficient irrigation for others due to the combination of intake, slope, and flow rate. Therefore, effective management under these conditions is extremely difficult, if not impossible". All these suggest that the CAD programme and the main system management must work in unison to obtain maximum benefits.

6.4.18. Farmer's Participation

The farmer forms the target group for all irrigation management and, hence, is a good source of first hand information right from the investigation to the operational stage of any irrigation system. It is, therefore, recognised that the participation of the farmer in all stages of the irrigation system would be very beneficial. Yoganarasimhan (28) has suggested farmer's participation in the following three phases:

Phase 1 – Initiation

- — Publicising the proposal, without restriction, as a promotional measure to gain popular support.
- — Extending invitations for information meeting.
- — Scheduling public hearing.
- — Warranting the critical assessment of objections raised as well as of individual alternatives of supplementary proposals.
- — Facilitating discussions and debates on proposal components and details.
- — Arranging for secret ballots to be cast in preparation of final decision.
- — Facilitating the nomination and election of farmer's representatives to join working groups.
- — Drafting the statutes of organisation which may only be adopted by majority vote through secret ballot.
- — Imparting courage, inspiration, and resolution to farmers.
- — Creating an atmosphere of trust and confidence in the feasibility of local participation.

Phase 2 – Continuation

— Establishing formal organisations of farmers.
— Recognising the roles of organisation.
— Forming joint committees charged with proposing and reviewing.
— Conducting training courses.
— Inviting contributions from farmers in the process of implementation, management, operation and maintenance, and in situations where repairs are required.
— Creating an atmosphere of collaboration and cooperation.
— Strengthening the task-oriented leadership based on majority group consensus and support.

Phase 3 – Perpetuation

— Assisting farmers to maintain their level of participation and increase its intensity.
— Relating their duties to improve their socio-economic status.
— Monitoring the performance of programmes to generate feedback into assistance and instruction.
— Inducing farmers to actively participate in the management through unreserved recognition of their achievements.

6.4.19. Irrigation Manager

Another important factor is the set of people who manage canal irrigation. Any improvement in the existing canal irrigation management, *viz.*, scheduling, reducing losses at night, etc. can only be initiated by the canal managers who are mostly engineers. Their incentives include convenience and amenity for good living, career prospects, job status, respectable income in proportion to their calibre, the avoidance of stress due to 'farmers and politicians' complaints and pressures, and professional satisfaction. The motivation of the canal managers is adversely affected by the 'transfer trade' in which operation and maintenance as well as some other postings are sold by politicians (1). The price of such postings may be several times the annual salary of a manager. The manager then raises money from maintenance works and from farmers who seek firm assurance for supply of water. Such corruption on canal irrigation systems has five adverse effects: (*i*) costs to farmers especially the poorer and weaker, (*ii*) bad physical work in maintenance, (*iii*) bad canal management, (*iv*) indiscipline of field staff, and (*v*) managers being demoralised and distracted from their proper work (1). Effective vigilance, political reforms and discipline can possibly improve the situation. Besides, separating the operations and maintenance cadres, inculcating awareness among farmers about their rights, and introducing accountability and incentives for managers may effectively improve the conditions to a great extent.

6.5. OPERATION AND MAINTENANCE OF CANAL IRRIGATION SYSTEMS

Operation and maintenance of an irrigation and drainage (I and D) system implies management of the system. The plan of operation and management (POM) or the management plan includes a set of documents and instructions, organisation charts, work procedures and rules (including coordination with other disciplines), programmes and schedules which aim at achieving efficient and optimal functioning of the irrigation system. POM is not a rule book but a set of guidelines. POM is not static but dynamic and may require updating during project implementation, postimplementation period of the project (in the light of advances in science and technology, management techniques and the experience gained over the years) in the operation and maintenance of the system.

The operation and maintenance (O and M) unit (or division or designated agency for the formulation of POM) must commence very early in the project planning phase (for aspects such as selection of the conveyance system or determination of the agricultural activities in relation to their costs or training of personnel and farmers) and should continue through the planning, design (for aspects such as detailing the scheme of operation whether regulated/unregulated supply remote/on-site control, storage requirement *etc.* ; overall conveyance/delivery system ; the control, monitoring and communication system, specific O and M offices, inspection houses, shops, yards and related features), construction (for installing the O and M organisation in the field, commissioning of project facilities and transferring responsibility from construction to O and M), commissioning, and operation phases.

Performance analysis of major and medium irrigation projects completed in the post-independence period with huge public money has revealed that the contemplated objectives and benefits have not been achieved in several cases. A proper Plan for Operation and Maintenance (29) is, therefore, necessary to

- achieve stipulated levels of project services including maintenance at minimum achievable cost
- achieve optimum use of canal water
- provide detailed O and M guidelines during various anticipated scenarios of water availability, including equitable water distribution upto the tail-end of the system, and
- effect efficient coordination of staff, equipment, physical and financial resources and related disciplines, active involvement of farmers *etc.*

6.5.1. Operation Policy

Operation policy for any irrigation project should be such that the system is operated in the basic interest of beneficiaries *i.e.,* farmers, and policy conforms to all the laws, ordinances of the state.

The project management may frame appropriate rules, proceed with activities for regulation and conservation of water and for the protection of quality of water. Operation within the project will be guided by such rules and operational policies adopted by the project management.

Following groups of activities are included in the formulation of System Operation rules:

(*i*) Detailed operation policy, rules and specifications

(*ii*) Irrigation plan (seasonal and annual operating plan)

(*iii*) Operational Procedures

(*iv*) Emergency Procedures, and

(*v*) Operations beyond farm outlet

6.5.1.1. Detailed Operational Policy, Rules and Specifications

These should include matters such as

- water sources – limits (quantitative and legal/contractual in relation to riparian states or otherwise) of its availability

- priorities for delivery during normal and restricted availability, and
- categories of demands (project-related, power generation, silt extrusion, flood moderation, rural, municipal, industrial environment, recreation, riparian entitlements) to be met.

6.5.1.2. Irrigation Plan-Seasonal and Annual

This should include irrigation plans including mid-plan corrections and operation of canal systems (main, branch, distributaries, minors) and adjustments of discharges. This would require

- estimation of water supply for different periods (rabi, kharif, hot weather *etc*.)
- estimation of water demand by different users (cropping/demand pattern)
- application of appropriate water allocation criteria and procedures (farmers' involvement), and
- matching supply and demand

6.5.1.3. Operational Procedures

A specific set of procedures and instructions for operating features of units and components such as

- water resources and storage (Reservoir)
- distribution of water
- system scheduling
- operation of delivery system, and
- safeguarding canal system

6.5.1.4. Emergency Procedures

An Emergency Preparedness Plan (also referred to as a Disaster Management Plan) should be developed for all such components (such as dams) and facilities failure (or malfunction) of which could cause

- danger to human life
- substantial property damage
- loss of production, and
- disruption to other community activities

The plan should include

- emergency depots with stockpiles of materials for rapid repairs
- procurement of labour and equipment from some agencies, and
- guidelines/instructions to meet situation such as excessive rainfall, blockage or malfunctioning of gates, breaches or overtopping of banks/flood embankments, obstruction of drainage *etc*.

6.5.1.5. Operation below Outlets

Operation downstream of outlets is the responsibility of individual farmers. However, for small farms, it is common practice to deliver water in bulk to water user groups which could be organised in either formal or informal way. The organisation and responsibility of each such groups and the rights and obligations of each member of the group should be clearly stated.

6.5.1.6. Typical Example of Operating Rules

The operating rules to be included, but not limited to, for a canal irrigation system are as follows (29):

1. The control of the system upto outlets lies with the irrigation department.
2. Maximum water permissible, within the allocation, made by the State will be stored and diverted for irrigation.
3. Water will be supplied to all farmers in the command equitably, proportionate to their holding, as per availability.
4. Water will be delivered as per rotational water supply and schedules communicated to all concerned in advance.
5. Water will not be delivered to, nor will be supplied to any land not included in the project, by the project authorities.
6. Unexpected interruptions in service may occur due to breaches in canals *etc.*, for which the users will not have any recourse for compensation.
7. During normal years of water availability, the water will be supplied generally at 14 days frequency to the farmers.
8. During scarcity years, the quantity of water supplied will be proportionate to availability. This will be done either by less number of turns or by reducing quantity supplied per turn or both. However, all beneficiaries in commissioned command area will be supplied with water.
9. The distributaries will run on 7-8 days 'on' and 7-8 days 'off' basis, to ensure that all farmers get at least one turn of irrigation during one 'on' period.
10. The farmers will take the water as per *Warabandi* schedules prepared on outlet basis. The period of water supply below outlet will be in proportion to the area of the holding bears to the total area under the outlet. The effective period of 168 hours will be apportioned to each holding in the same ratio. These timings and the order of getting water will remain fixed during the year and will be changed by 12 hours in the following year, in order to rotate the day and night irrigation supplies among farmers. The irrigation will have to be done round the clock.
11. When the water runs through the outlets, the farmers having their turn on that date and at that time will start irrigation in the sequential order. Otherwise, the defaulting farmer will lose the particular turn.
12. The schedules for water delivery will be prepared taking into account the general cropping pattern, sowing times and crop water requirements, and will not consider the crops grown by individual farmers and the area irrigated.
13. The schedules thus prepared at the beginning of the season will be rigidly followed and nobody from the operating staff, nor the farmers will be authorised to change, modify or alter the same, except that in times of heavy rainfall the supplies may be suitably reduced during the 'on' period for which no additional supply will be made during the following 'off' period and further that in special circumstances with prior consultation of standing sub-committee of the ICC/CADA board, the schedules may have to be changed.
14. In case of long dry spells in the command, efforts will be made by the management to supply more turns, if water is available in the reservoir and these will be communicated to the farmers at least 48 hours in advance.

15. The farmers will be free to grow any crops (except those that are prohibited by the project authorities) and extend the irrigation to part or full area of their land within the water allocated to them.
16. Taking water unauthorizedly, out of turn and more in quantity by heading up the distributary, lateral, *etc*. will be considered as legal offence. No farmer will be authorized to tamper with the flows in distributaries/laterals/field channels.
17. Additional outlets will be permitted only upon the written orders of the Chief Engineer (O and M).
18. Irrigation supplies will generally begin from 1st June and will continue upto 7th March; with *Kharif* season from 1 June to 15 October and *Rabi* season from 6 October to 28/29 February.
19. During normal year, efforts will be made to provide 19 to 21 turns to all the farmers.
20. Every watering will provide 17 mm/ha of water for the CCA of each farm.

6.5.2. Maintenance Aspects

For proper upkeep of any irrigation system maintenance and repair (M & R) works form an essential part of the project management soon after commissioning of the project. Such M & R works could be periodical, special, emergency, and remodelling to either improve or modify functioning of the project.

Proper maintenance of an irrigation system is necessary because of the following (29) :

(*i*) The convergence and delivery system of the irrigation project should be in a good condition for effective water management and to retain system's operation efficiency.

(*ii*) Practical, predictable and equitable deliveries to the outlets will increase crop productivity from the existing irrigation systems and such deliveries can be assured by timely rectification of maintenance deficiencies.

(*iii*) Conservation of precious water resource for irrigation is essential. Economically viable project sites are shrinking and exploitation of sites in different terrain for future development with available technology is capital intensive and requires huge public funds.

The national water policy also stipulates that structures and systems created through massive investments should be properly maintained in good health. Appropriate annual provisions should be made for this purpose in the budget.

6.5.2.1. Types of Maintenance

Maintenance works include operations performed in preserving system and facilities in good or near-original condition. Repairs are part of maintenance. Maintenance can be of the following types (29):

1. Normal — usual; done annually
2. Emergency — done under unusual conditions affecting adversely the safety of the system.
3. Deferred — routine/normal ones deferred because of shortage of funds/machines, affects adversely hydraulic performance.
4. Catch-up — programme that takes care of deferred maintenance

5.	Preventive	—	takes care of the causes creating the maintenance needs when they are only minor
6.	Rehabilitation	—	renovation needed due to accumulation of deferred maintenance ; otherwise, only for ageing structures
7.	Modernisation	—	updating/improving the system to meet enhanced social, technical, economic activities.
8.	'Walk-thru'	—	two or three individuals walking along canals, taking notes on maintenance needs
9.	Special	—	due to unforeseen mishaps such as breaches.

6.5.2.2. Maintenance Policy

The maintenance policy should include directions on

- degree to which preventive maintenance is to be relied upon
- appropriateness of deferring maintenance of components for which plans have been approved for rehabilitation or modernisation
- relationship among maintenance, rehabilitation and modernisation
- using improved technology for maintenance
- transparency of maintenance
- involvement of water users, and
- execution of maintenance

6.5.2.3. Sources of Maintenance Data

The data to be used in preparing maintenance plans may originate from one of the following :

- reports from field personnel
- inspection report from superior officers
- performance measurement data, and
- research reports on maintenance (material, equipment, method)

6.5.2.4. Work Plan Preparation

The following matters are to be usually included in the maintenance work plans :

- contents of work plans
- type of maintenance
- period of plan
- definition and extent of work
- estimates of cost
- schedule
- methods of execution (departmental, contact-labour/material)
- assignment of responsibilities
- principles in light of policy
- maintenance of services
- approval, and
- notification and liaison

6.5.2.5. Typical Example of Maintenance Aspects

Some maintenance aspects of a Head Regulator to be included in the maintenance plan are as follows (29) :

1. **Approach Channel.** For sediment-free maintenance of approach channel specify
   - Methods such as reduction of sediment entry into the channel by judicious operation of river sluices during floods
   - Frequency of silt clearance of approach channel and maintenance thereof to its original design, and
   - Masonry protection works of the channel to be maintained in good condition.
2. **Gauges.** Gauges must be kept clean and painted regularly so as to remain clearly readable without ambiguity. Approach to gauge is to be maintained for all weather use.
3. **Discharge Verification.** The discharge passing through the head regulator is normally worked out from discharge rating curves (tables) prepared by model experiments, both for free-fall and drowned conditions. This needs verification at site by current meter, *etc*. In case of significant variation, rating curves (tables) need to be modified in consultation with the research unit.
4. **Energy dissipating devices downstream of the Head Regulator.** The performance of such devices should be monitored to check conformance with model results. In case of significant non-conformance, research unit should be consulted. Bank and bed protection of exit channel should be carried out regularly. Routine repairs should be done during closures, and emergency repairs may be undertaken as called for.

6.6. EVALUATION OF PERFORMANCE OF CANAL IRRIGATION SYSTEMS

To manage a system properly, the physical effectiveness of past operations must be considered against the original criteria set forth for the project, or as subsequently amended following modification of the facilities. Procedures for acting on the indicators uncovered in evaluation are critical to the financial and operational efficiencies of a system. Priorities for adjustments in the system and scheduling the needed maintenance can best be made by using inputs for timely and proper evaluation reports. Some of the diagnostic analyses that can be considered are (29) :

(*i*) ***Farmers operation performance***
- adequacy of crop production techniques for irrigated farming including adequacy of supply of inputs such credit, certified seeds, fertilisers, pesticides, *etc.*
- adequacy of irrigation methods
- farm management and economic results
- soil management and erosion control, and
- on-farm efficiency of water use

(*ii*) ***Delivery operational performance***
- water use efficiency in distribution
- water losses (physical including evaporation)
- project overall water use efficiency

- deep percolation
- canal seepage
- spillage from canals
- dam and foundation seepage
- water operational losses (such as leakage from gates, *etc*.)
- adequacy of delivery scheduling, and
- energy use

(*iii*) ***Drainage operational performance***

- drainage requirement area-wise
- water table fluctuations by season and years
- water quality changes reach-wise for drain effluents, and
- soil salinity changes area-wise

(*iv*) ***Maintenance of individual components***

- civil works (such as for canals, hydraulic structures, drains, building and roads), and
- equipment (pumps, hoists, earthmovers, trucks, loaders, computers, measuring devices, gates, office and communication equipment) degradation and prediction of replacement schedule :

(*v*) ***Overall project review: efficiency and effectiveness***

The procedures outlined in paragraph (*i*) to (*iv*) above will facilitate evaluation of the relative performance of various project components and activities, and should show whether poor performance is a technical or managerial problem which could be resolved by internal management processes.

It may be necessary, from time to time, to carry out a more wide-ranging evaluation of the total project, for example, if poor performance is a result of inadequate flow of funds for O & M because of inadequate generation of benefits, or from external economic, social or environmental effects.

Some of the matters which should be highlighted in such a review are :

- documentation of project costs and revenues
- adequacy of revenue sources to meet O & M needs
- benefit flows from project farmers, governments, and others
- comparison of benefits generated to revenue required
- relevant agricultural and engineering issues, and
- social and environmental changes and concerns and resulting implications

EXERCISES

6.1 Describe the common criteria for judging the performance of an irrigation system.

6.2 Discuss the inadequacies of present-day canal irrigation management in India and suggest suitable methods of improvement.

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7

HYDRAULICS OF ALLUVIAL CHANNELS

7.1. GENERAL

Civilisation prospered in agricultural lands by the side of rivers. From the beginning of civilisation, mankind has given attention to the problems of rivers. The boundaries of many rivers consist of loose material, which may be carried by the water flowing in these rivers. Depending upon the prevailing conditions, the loose material may either get deposited or scoured. Thus, the boundaries of such a river channel are mobile and not rigid. A change in discharge of water flowing in a rigid boundary channel will cause a change only in the depth of flow. But, in case of mobile (or loose) boundary channels, a change in discharge may cause changes in cross-section, slopes, plan-form of the channel, bed forms and roughness coefficient. The application of the theory of rigid boundary channels to loose boundary channels is, therefore, not correct. Evidently, the problem of mobile boundary channels is more complicated.

The bed of a river channel generally consists of non-cohesive sediment (*i.e.,* silt, sand, and gravel) and such rivers are called alluvial rivers.

Sediment (also known as alluvium) is defined as the loose and noncohesive material through which a river or channel flows. Sediment is also defined as fragmental material transported by, suspended in, or deposited by water or air, or accumulated in the beds by other natural agents. Ice, logs of wood, and organic materials flowing with water are excluded from the definition of sediment.

A channel (or river) flowing through sediment and transporting some of it along with the flowing water is called an alluvial channel (or river). The complex nature of alluvial channel problems stands in the way of obtaining analytical solutions, and experimental methods are generally adopted for obtaining solutions of problems to alluvial channels.

7.2. INCIPIENT MOTION OF SEDIMENT

Consider the case of flow of clear water in an open channel of a given slope with a movable bed of non-cohesive material. At low discharges, the bed material remains stationary and, hence, the channel can be treated as rigid. With the increase in discharge, a stage will come when the shear force exerted by the flowing water on a particle will just exceed the force opposing the movement of the particle. At this stage, a few particles on the bed move intermittently. This condition is called the incipient motion condition or, simply, the critical condition.

A knowledge of flow at the incipient motion condition is useful in fixing slope or depth for clear water flow in an alluvial channel. Knowledge of the incipient motion condition is also required in some methods of calculation of sediment load. Hence, there is a need to understand the phenomenon which initiates motion of sediment particles.

The experimental data on incipient motion condition have been analysed by different investigators using one of the following three approaches (1):

(*i*) Competent velocity approach,

(*ii*) Lift force approach, and

(*iii*) Critical tractive force approach.

Competent velocity is the mean velocity of flow which just causes a particle to move. A relationship among the size of the bed material, its relative density, and the competent velocity is generally developed and used.

Investigators using the lift force approach assume that the incipient motion condition is established when the lift force exerted by the flow on a particle just exceeds submerged weight of the particle.

The critical tractive force approach is based on the premise that it is the drag (and not lift) force exerted by the flowing water on the channel bed which is responsible for the motion of the bed particles.

Of these three approaches, the critical tractive force approach is considered most logical and is most often used by hydraulic engineers. Hence, only this approach has been discussed here.

The critical tractive (or shear) stress is the average shear stress acting on the bed of a channel at which the sediment particles just begin to move. Shields (2) was the first investigator to give a semi-theoretical analysis of the problem of incipient motion. According to him, a particle begins to move when the fluid drag F_1 on the particle overcomes the particle resistance F_2. The fluid drag F_1 is given as

$$F_1 = k_1 \left[C_D \, d^2 \, \frac{1}{2} \rho \, u_d^{\,2} \right]$$

and the particle resistance F_2 is expressed as

$$F_2 = k_2 \, [d^3 \, (\rho_s - \rho) \, g]$$

where,
C_D = the drag coefficient,
d = the size of the particle,
ρ = the mass density of the flowing fluid,
u_d = the velocity of flow at the top of the particle,
ρ_s = the mass density of the particle,
g = acceleration due to gravity,
k_1 = a factor dependent on the shape of the particle, and
k_2 = a factor dependent on the shape of the particle and angle of internal friction.

Using the Karman-Prandtl equation for the velocity distribution, the velocity u_d can be expressed as

$$\frac{u_d}{u_*} = f_1 \left(\frac{u_* \, d}{\nu} \right) = f_1(R^*)$$

Here, ν is the kinematic viscosity of the flowing fluid, u_* the shear velocity equal to $\sqrt{\tau_0 / \rho}$ and τ_0 is the shear stress acting on the boundary of the channel.

Similarly,

$$C_D = f'\left(\frac{u_d\, d}{\nu}\right)$$

or
$$C_D = f_2\left(\frac{u_* d}{\nu}\right) = f_2\,(R^*)$$

Thus,
$$F_1 = k_1 f_2\,(R^*)\, d^2\, \frac{1}{2}\rho u_*^{\,2}\,[f_1\,(R^*)]^2$$

At the incipient motion condition, the two forces F_1 and F_2 will be equal. Hence,

$$k_1 f_2\,(R_c^*)\, d^2\, \frac{1}{2}\rho\, u_{*c}^{\,2}\,[f_1\,(R_c^*)]^2 = k_2\,[d^3\,(\rho_s - \rho)\, g]$$

Here, the subscript c has been used to indicate the critical condition (or the incipient motion condition). The above equation can be rewritten as

$$\frac{\rho\, u_{*c}^2}{(\rho_s - \rho)\, gd} = \frac{2\,k_2}{k_1}\, f\,(R_c^*)$$

Alternatively,

$$\tau_c^* = f\,(R_c^*) \tag{7.1}$$

where,
$$\tau_c^* = \frac{\tau_c}{\Delta\,\rho_s g d}$$

$$\tau_c = \rho\, u_{*c}^2$$

and
$$\Delta\,\rho_s = \rho_s - \rho$$

On plotting the experimental data collected by different investigators, a unique relationship between τ_c^* and R_c^* was obtained by Shields (2) and is as shown in Fig. 7.1. The curve shown in the figure is known as the Shields curve for the incipient condition. The parameter $R_c^* = \left(\frac{u_{*c} d}{\nu}\right)$ is, obviously, the ratio of the particle size d and ν/u_{*c}. The parameter $\frac{\nu}{u_{*c}}$ is a measure of thickness of laminar sublayer, *i.e.,* δ'. Hence, R_c^* can be taken as a measure of the roughness of the boundary surface. The boundary surface is rough at large values of R_c^* and, hence, τ_c^* attains a constant value of 0.06 and becomes independent of R_c^* at $R_c^* \geq 400$. This value of R_c^* (*i.e.,* 400), indicating that the boundary has become rough, is much higher than the value of 70 at which the boundary becomes rough from the established criterion $\frac{d}{\delta'} > 6.0$. Likewise, the constant value of τ_c^* equal to 0.06 is also on the higher side.

Alternatively, one may use the following equation of the Shields' curve for the direct computation of τ_c (3) :

$$\frac{\tau_c}{\Delta\,\rho_s g \left(\dfrac{\rho\,\nu^2}{\Delta\,\rho_s\, g}\right)^{1/3}} = 0.243 + \frac{0.06\, d_*^2}{(3600 + d_*^2)^{1/2}} \tag{7.2}$$

in which,
$$d_* = \frac{d}{(\rho\,\nu^2 / \Delta\,\rho_s g)^{1/3}}$$

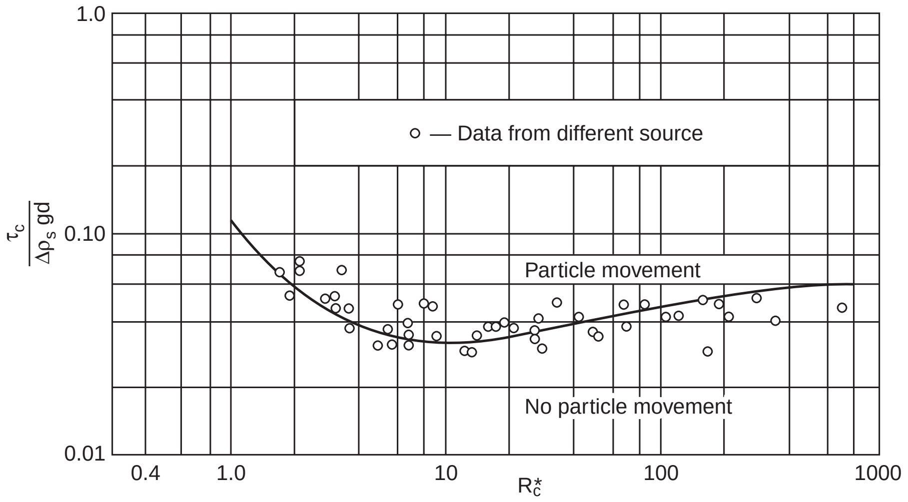

Fig. 7.1 Shields curve for incipient motion condition (2)

For specific case of water (at 20°C) and the sediment of specific gravity 2.65 the above relation for τ_c simply reduces to

$$\tau_c = 0.155 + \frac{0.409\, d^2}{(1 + 0.177\, d^2)^{1/2}} \tag{7.3}$$

in which τ_c is in N/m^2 and d is in mm. Equations (7.2) and (7.3) are expected to give the value of τ_c within about ± 5% of the value obtained from the Shields curve (3).

Yalin and Karahan (4) developed a similar relationship (Fig. 7.2) between τ_c^* and R_c^* using a large amount of experimental data collected in recent years. It is noted that at higher values of R_c^* (> 70) the constant value of τ_c^* is 0.045. This relation (Fig. 7.2) is considered better than the more commonly used Shields' relation (1).

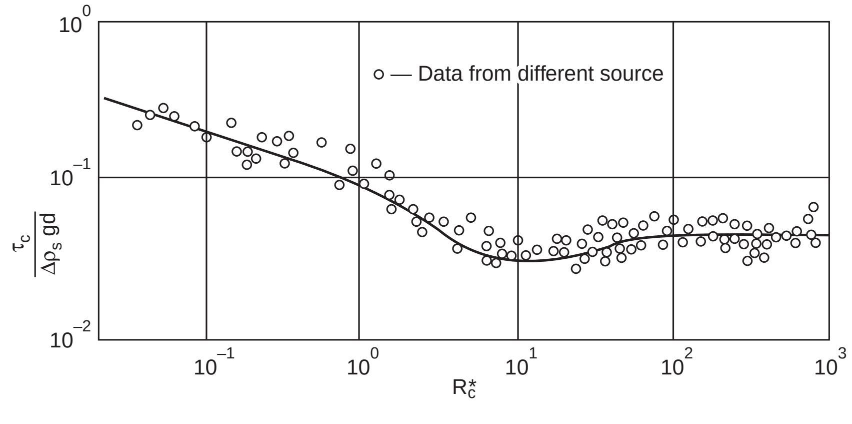

Fig. 7.2 Yalin and Karahan curve for incipient motion condition (4)

For given values of d, ρ_s, ρ, and ν, the value of τ_c can be obtained from Fig. 7.1 or Fig. 7.2 only by trial as τ_c appears in both parameters τ_c^* and R_c^*. However, the ratio of R_c^* and $\sqrt{\tau_c^*}$ yields a parameter R_0^* which does not contain τ_c and is uniquely related to τ_c^*.

$$R_0^* = \frac{R_c^*}{\sqrt{\tau_c^*}} = \frac{u_{*c}\, d}{\nu}\left(\frac{\Delta\,\rho_s\, gd}{\tau_c}\right)^{1/2} = \left(\frac{\Delta\,\rho_s\, gd^3}{\rho\nu^2}\right)^{1/2} \tag{7.4}$$

Since R_c^* is uniquely related to τ_c^* (Fig. 7.2), another relationship between R_0^* and τ_c^* can be obtained using Fig. 7.2 and Eq. (7.4).

The relationship between R_0^* and τ_c^* is as shown in Fig. 7.3 and can be used to obtain direct solution for τ_c for given values of d, ρ_s, ρ and ν.

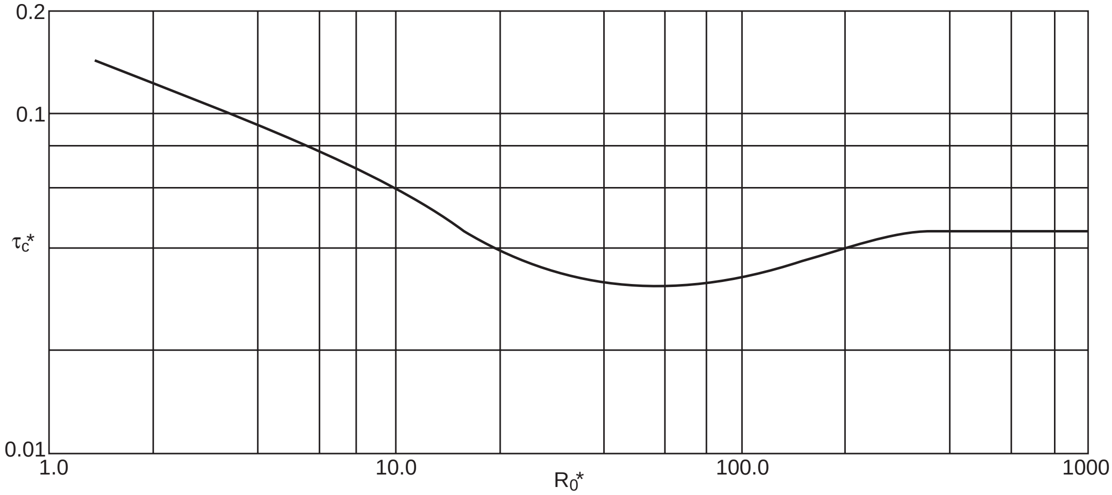

Fig. 7.3 Variation of R_0^* and τ_c^* based on Fig. 7.2

Example 7.1 Water flows at a depth of 0.3 m in a wide stream having a slope of 1×10^{-3}. The median diameter of the sand on the bed is 1.0 mm. Determine whether the grains are stationary or moving ($\nu = 10^{-6}$ m²/s).

Solution:

$$R_0^* = \left(\frac{\Delta\,\rho_s gd^3}{\rho\,\nu^2}\right)^{1/2} = \left(\frac{1.65 \times 9.81 \times (1 \times 10^{-3})^3}{(10^{-6})^2}\right)^{1/2} = 127.23$$

From Fig. 7.3, $$\tau_c^* = 0.035 = \frac{\tau_c}{\Delta\,\rho_s\, gd} = \frac{\tau_c}{1650 \times 9.81 \times (1.0 \times 10^{-3})}$$

$\therefore$ $\tau_c = 0.5665$ N/m²

Shear stress on the bed, $\tau_0 = \rho\, ghS = 9810 \times 0.3 \times 10^{-3} = 2.943$ N/m²

Since $\tau_0 > \tau_c$, the grains would move. τ_c can also be computed from Eq. (7.3).

$$\tau_c = 0.155 + \frac{0.409\,(1.0)^2}{[1 + 0.177\,(1.0)^2]^{1/2}}$$
$$= 0.532 \text{ N/m}^2.$$

7.3. REGIMES OF FLOW

When the average shear stress on the bed of an alluvial channel exceeds the critical shear stress, the bed particles are set in motion and thus disturb the plane bed condition. Depending upon the prevailing flow conditions and other influencing parameters, the bed and the water surface attain different forms. The features that form on the bed of an alluvial channel due to the flow of water are called 'bed forms', 'bed irregularities' or 'sand waves'. Garde and Albertson (5) introduced another term 'regimes of flow' defined in the following manner:

'As the sediment characteristics, the flow characteristics and/or fluid characteristics are changed in alluvial channel, the nature of the bed surface and the water surface changes accordingly. These types of the bed and water surfaces are classified according to their characteristics and are called regimes of flow.'

Regimes of flow will affect considerably the velocity distribution, resistance relations, and the transport of sediment. The regimes of flow can be divided into the following four categories:

(*i*) Plane bed with no motion of sediment particles,

(*ii*) Ripples and dunes,

(*iii*) Transition, and

(*iv*) Antidunes.

Plane Bed with no Motion of Sediment Particles

When sediment and flow characteristics are such that the average shear stress on the bed is less than the critical shear stress, the sediment particles on the bed do not move. The bed remains plane and the channel boundary can be treated as a rigid boundary. 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 Manning's equation can be used for prediction of the mean velocity of flow with Manning's n obtained from the Strickler's equation, as discussed later in this chapter.

Ripples and Dunes

The sediment particles on the bed start moving when the average shear stress of the flow τ_0 exceeds the critical shear τ_c. As a result of this sediment motion, small triangular undulations known as ripples form on the bed [Fig. 7.4 (*a*)]. Ripples do not occur if the sediment is coarser than 0.6 mm. The length (between two adjacent troughs or crests) of the ripples is usually less than 0.4 m and the height (trough to crest) does not exceed 40 mm. The sediment motion is confined to the region near the bed and the sediment particles move either by sliding or taking a series of hops.

With the increase in discharge (and, hence, the average shear stress τ_0) the ripples grow into dunes [Fig. 7.4 (*b*)]. Dunes too are triangular undulations but of larger dimensions. These undulations are also unsymmetrical with a flat upstream face inclined at about 10-20° with the horizontal and steep downstream face whose angle of inclination with the horizontal is approximately equal to the angle of repose of the sediment material. Sometimes, ripples appear on the upstream face of a dune. The dunes in laboratory flumes may have length and height up to about 3 m and 0.4 m, respectively. But, in large rivers, the dunes may be several hundred metres long and up to about 15 m in height. The water surface falls over the crest of dunes and, hence, the water surface waves are out of phase with the bed waves. The flow conditions still correspond to the subcritical range. While most of the sediment particles move along the bed, some finer particles of the sediment may go in suspension.

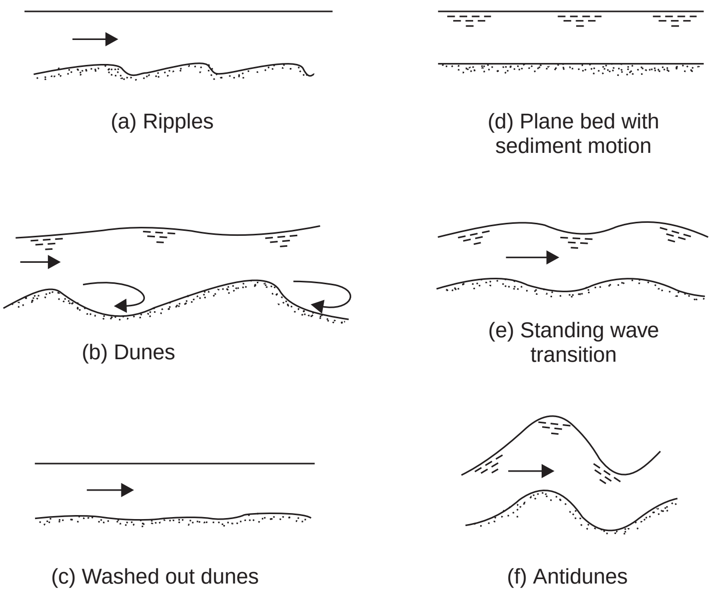

Fig. 7.4 Regimes of flow in alluvial channels

Ripples and dunes have many common features and, hence, are generally dealt with together as one regime of flow. Both ripples and dunes move downstream slowly. Kondap and Garde (6) have given an approximate equation for the advance velocity of ripples and dunes, U_w, as follows:

$$\frac{U_w}{U} = 0.021 \left(\frac{U}{\sqrt{gh}}\right)^3 \tag{7.5}$$

Here, U is the mean velocity of flow, and h is the average depth of flow. The average length (L) and height (H) of ripples and dunes can be estimated from the equations proposed by Ranga Raju and Soni (7):

$$\frac{H}{d}\left[\frac{U}{\sqrt{gR}}\right]^3 \frac{U}{\sqrt{(\Delta \rho_s / \rho)\, gd}} = 6500\, (\tau_*')^{8/3} \tag{7.6}$$

$$\left[\frac{U}{\sqrt{\Delta \rho_s / \rho)\, gd}}\right]^3 \frac{U}{\sqrt{gR}} \times \frac{L}{D} = 1.8 \times 10^8\, (\tau_*')^{10/3} \tag{7.7}$$

in which,

$$\tau_*' = \frac{\rho R'\, s}{\Delta \rho_s d} \tag{7.8}$$

Here, R' (*i.e.,* hydraulic radius corresponding to the grain roughness) is obtained from the equation,

$$U = \frac{1}{n_s} R'^{2/3} S^{1/2} \tag{7.9}$$

n_s (*i.e.,* Manning's roughness coefficient for the grains alone) is calculated from Strickler's equation,

$$n_s = \frac{d^{1/6}}{25.6} \tag{7.10}$$

in which d is in metres. R is the hydraulic radius of the channel.

Transition

With further increase in the discharge over the duned bed, the ripples and dunes are washed away, and only some very small undulations are left [Fig. 7.4 (*c*)]. In some cases, however, the bed becomes plane but the sediment particles are in motion [Fig. 7.4 (*d*)]. With slight increase in discharge, the bed and water surfaces attain the shape of a sinusoidal wave form. Such waves, known as standing waves [Fig. 7.4 (*e*)], form and disappear and their size does not increase much. Thus, in this regime of transition, there is considerable variation in bed forms from washed out dunes to plane bed with sediment motion and then to standing waves. The Froude number is relatively high. Large amount of sediment particles move in suspension besides the particles moving along the bed. This regime is extremely unstable. The resistance to flow is relatively small.

Antidunes

When the discharge is further increased and flow becomes supercritical (*i.e.,* the Froude number is greater than unity), the standing waves (*i.e.,* symmetrical bed and water surface waves) move upstream and break intermittently. However, the sediment particles keep on moving downstream only. Since the direction of movement of bed forms in this regime is opposite to that of the dunes, the regime is termed antidunes, [Fig. 7.4 (*f*)]. The sediment transport rate is, obviously, very high. The resistance to flow is, however, small compared to that of the ripple and dune regime. In the case of canals and natural streams, antidunes rarely occur.

7.3.1. Importance of Regimes of Flow

In case of rigid boundary channels, the resistance to flow is on account of the surface roughness (*i.e.,* grain roughness) only except at very high Froude numbers when wave resistance may also be present. But, in the case of alluvial channels, the total resistance to flow comprises the form resistance (due to bed forms) and the grain resistance. In the ripple and dune regime, the form resistance may be an appreciable fraction of the total resistance. Because of the varying conditions of the bed of an alluvial channel, the form resistance is a highly varying quantity. Any meaningful resistance relation for alluvial channels shall, therefore, be regime-dependent. It is also evident that the stage-discharge relationship for an alluvial channel will also be affected by regimes of flow.

The form resistance, which is on account of the difference in pressures on the upstream and downstream side of the undulations, acts normal to the surface of the undulations. As such, the form resistance is rather ineffective in the transport of sediment. Only grain shear (*i.e.,* the shear stress corresponding to grain resistance) affects the movement of sediment.

7.3.2. Prediction of Regimes of Flow

There are several methods for the prediction of regimes. The method described here has been proposed by Garde and Ranga Raju (8).

The functional relationship for resistance of flow in alluvial channels was written, following the principles of dimensional analysis, as follows:

$$\frac{U}{\sqrt{(\Delta \rho_s/\rho)\, gR}} = f\left[\frac{R}{d}, \frac{S}{\Delta \rho_s / \rho}, \frac{g^{1/2} d^{3/2}}{\nu}\right] \tag{7.11}$$

Here, S is the slope of the channel bed. Since resistance to flow and the regime of flow are closely related with each other, it was assumed that the parameters on the right-hand side of Eq. (7.11) would predict the regime of flow. The third parameter (*i.e.,* $g^{1/2}\, d^{3/2}/\nu$) was dropped from the analysis on the plea that the influence of viscosity in the formation of bed waves is rather small. The data from natural streams, canals, and laboratory flumes in which the regimes had also been observed, were used to develop Fig. 7.5 on which lines demarcating the regimes of flow have been drawn. The data used in developing Fig. 7.5 cover a wide range of depth of flow, slope, sediment size, and the density of sediment.

It should be noted that the lines of 45° slope on Fig. 7.5 – such as the line demarcating 'no motion' and 'ripples and dunes' regimes – represent a line of constant value of $\tau_* \left(= \dfrac{\rho\, RS}{\Delta\, \rho_s d} \right)$.

This means that different regimes of flow can be obtained at the same shear stress by varying suitably the individual values of R and S. Therefore, shear stress by itself cannot adequately define regimes of flow.

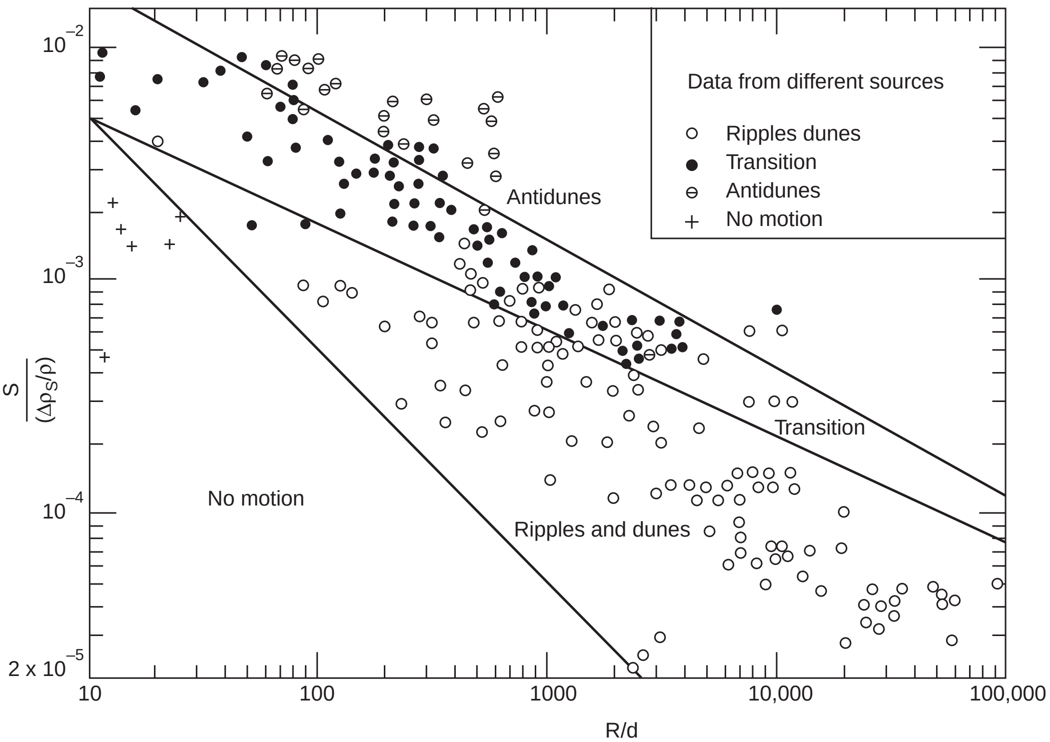

Fig. 7.5 Predictor for regimes of flow in alluvial channels (8)

The method of using Fig. 7.5 for prediction of regimes of flow consists of simply calculating the parameters R/d and $S/(\Delta\, \rho_s/\rho)$ and then finding the region in which the corresponding point falls. One obvious advantage of this method is that it does not require knowledge of the mean velocity U and is, therefore, suitable for prediction of regimes for resistance problems.

Example 7.2 An irrigation canal has been designed to have $R = 2.5$ m and $S = 1.6 \times 10^{-4}$. The sediment on the bed has a median size of 0.30 mm. Find: (i) the bed condition that may be expected, (*ii*) the height and spacing of undulations, and (*iii*) the advance velocity of the undulations. Assume depth of flow and mean velocity of flow to be 2.8 m and 0.95 m/s, respectively.

Solution:

$$\frac{R}{d} = \frac{2.5}{0.3 \times 10^{-3}} = 8333.33$$

$$\frac{S}{\Delta \rho_s / \rho} = \frac{1.6 \times 10^{-4}}{1.65} = 9.7 \times 10^{-5}$$

From Fig. 7.5, the expected bed condition would correspond to 'ripples and dunes' regime.

From Eq. (7.10),

$$n_s = \frac{d^{1/6}}{25.6} = \frac{(0.3 \times 10^{-3})^{1/6}}{25.6} = 0.01$$

and from the Manning's equation [Eq. (7.9)],

$$U = \frac{1}{n_s} R'^{2/3} S^{1/2}$$

$$\therefore \qquad R' = \left(\frac{Un_s}{S^{1/2}}\right)^{3/2} = \left[\frac{0.95 \times 0.01}{(1.6 \times 10^{-4})^{1/2}}\right]^{3/2} = 0.651 \text{ m}$$

$$\tau_*' = \frac{\rho R' S}{\Delta \rho_s d} = \frac{0.651 \times 1.6 \times 10^{-4}}{1.65 \times 0.3 \times 10^{-3}} = 0.21$$

Using Eq. (7.6),

$$\left(\frac{H}{d}\right)\left(\frac{U}{\sqrt{gR}}\right)^3 \frac{U}{\sqrt{(\Delta \rho_s / \rho)\, gd}} = 6500\, (\tau_*')^{8/3}$$

$$\frac{H}{0.3 \times 10^{-3}}\left(\frac{0.95}{\sqrt{9.81 \times 2.5}}\right)^3 \left(\frac{0.95}{\sqrt{9.81 \times 1.65 \times 0.3 \times 10^{-3}}}\right) = 6500\, (0.21)^{8/3}$$

$$\therefore \qquad H = 0.316 \text{ m}$$

Similarly, from Eq. (7.7)

$$\left[\frac{U}{\sqrt{(\Delta \rho_s / \rho)\, gd}}\right]^3 \frac{U}{\sqrt{gR}} \times \frac{L}{d} = 1.8 \times 10^8\, (\tau_*')^{10/3}$$

i.e.,

$$\left[\frac{0.95}{\sqrt{9.81 \times 1.65 \times 0.3 \times 10^{-3}}}\right]^3 \frac{0.95}{\sqrt{9.81 \times 2.5}} \times \frac{L}{0.3 \times 10^{-3}} = 1.8 \times 10^8\, (0.21)^{10/3}$$

$$\therefore \qquad L = 0.611 \text{ m}$$

$$\frac{U_w}{U} = 0.021\left[\frac{U}{\sqrt{gh}}\right]^3$$

$$\therefore \qquad U_w = 0.95 \times 0.021\left[\frac{0.95}{\sqrt{9.81 \times 2.8}}\right]^3 = 0.43 \text{ m/hr}$$

7.4. RESISTANCE TO FLOW IN ALLUVIAL CHANNELS

The resistance equation expresses relationship among the mean velocity of flow U, the hydraulic radius R, and the characteristics of the channel boundary. For steady and uniform flow in rigid boundary channels, the Keulegan's equations (logarithmic type) or power-law type of equations (like the Chezy's and the Manning's equations) are used. Keulegan (9) obtained the following logarithmic relations for rigid boundary channels:

For smooth boundaries,

$$\frac{U}{u_*} = 5.75 \log\left(\frac{u_* R}{v}\right) + 3.25 \tag{7.12}$$

For rough boundaries,

$$\frac{U}{u_*} = 5.75 \log (R/k_s) + 6.25 \tag{7.13}$$

For the range $5 < \frac{R}{k_s} < 700$, the Manning's equation,

$$U = \frac{1}{n} R^{2/3} S^{1/2} \tag{7.14}$$

has been found (9) to be as satisfactory as the Keulegan's equation [Eq. (7.13)] for rough boundaries. In Eq. (7.14), n is the Manning's roughness coefficient which can be calculated using the Strickler's equation,

$$n = \frac{k_s^{1/6}}{25.6} \tag{7.15}$$

Here, k_s is the equivalent sand grain roughness in metres. Another power-law type of equation is given by Chezy in the following form:

$$U = C \sqrt{RS} \tag{7.16}$$

Comparing the Manning's equations,

$$\frac{U}{u_*} = \frac{C}{\sqrt{g}} = \frac{R^{1/6}}{n\sqrt{g}} = \left(\frac{R}{k_s}\right)^{1/6} \frac{25.6}{\sqrt{g}} \tag{7.17}$$

In case of an alluvial channel, so long as the average shear stress τ_0 on boundary of the channel is less than the critical shear τ_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 develop 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 are 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 effects 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 method for computing resistance in alluvial channels can be grouped into two broad categories. The first includes such methods which deal with the overall resistance and use either a logarithmic type relation or a power-law type relation for the mean velocity. The second category of methods separates the total resistance into grain resistance and form resistance (*i.e.,* the resistance that develops on account of undulations on the channel bed). Both categories of methods generally deal with uniform steady flow.

7.4.1. Resistance Relationships based on Total Resistance Approach

The following equation, proposed by Lacey (10) on the basis of analysis of stable channel data from India, is the simplest relationship for alluvial channels:

$$U = 10.8R^{2/3} S^{1/3} \tag{7.18}$$

However, this equation is applicable only under regime conditions (see Art. 8.5) and, hence, has only limited application.

Garde and Ranga Raju (11) analysed data from streams, canals, and laboratory flumes to obtain an empirical relation for prediction of mean velocity in an alluvial channel. The functional relation, [Eq. (7.11)] may be rewritten (11) as

$$\frac{U}{\sqrt{(\Delta\rho_s / \rho)\, g\, R}} = f\left[\frac{R}{d}, \frac{S}{\Delta\rho_s / \rho}, \frac{g^{1/2} d^{3/2}}{\nu}\right] \tag{7.19}$$

By employing usual graphical techniques and using alluvial channel data of canals, rivers, and laboratory flumes, covering a large range of d and depth of flow, a graphical relation between $K_1 \dfrac{U}{\sqrt{(\Delta\rho_s / \rho)\, g R}}$ and $K_2 \left(\dfrac{R}{d}\right)^{1/3} \dfrac{S}{\Delta\rho_s / \rho}$, Fig. 7.6, was obtained for the prediction of

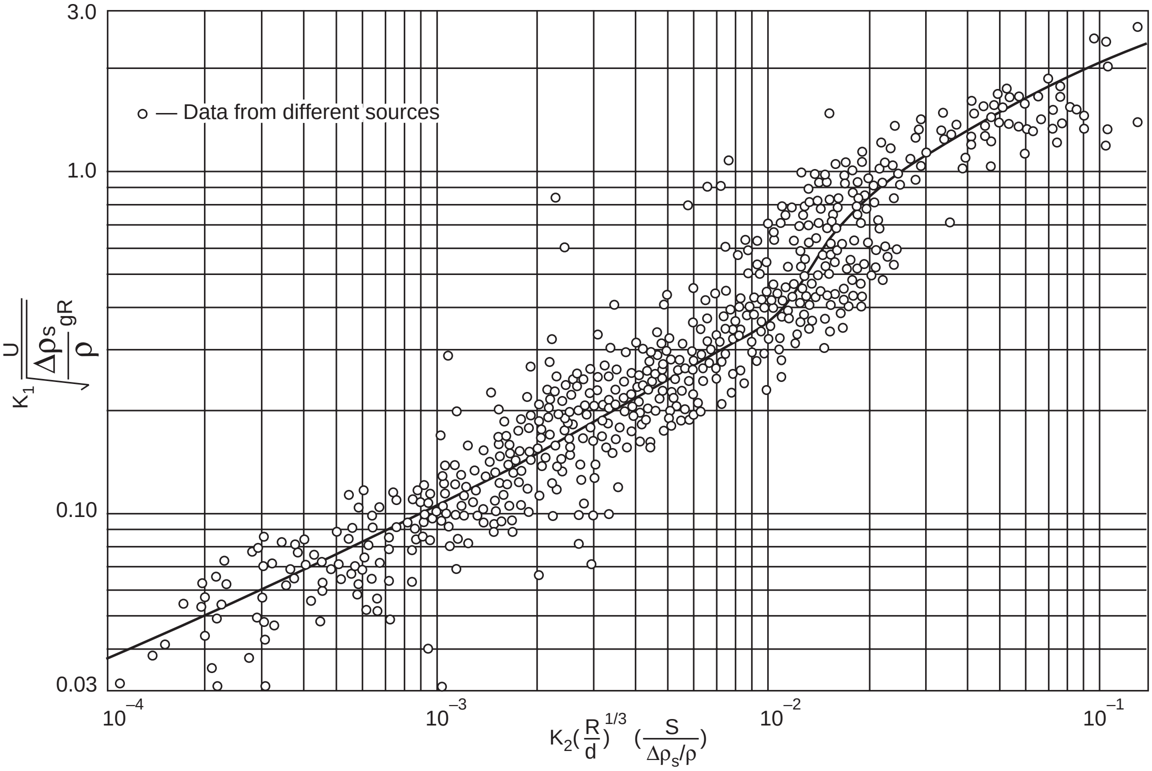

Fig. 7.6 Resistance relationship for alluvial channels (12)

the mean velocity U. The coefficients K_1 and K_2 were related to the sediment size d by the graphical relations shown in Fig. 7.7. It should be noted that the dimensionless parameter $g^{1/2}\, d^{3/2}/\nu$ has been replaced by the sediment size alone on the plea that the viscosity of the liquid for a majority of the data used in the analysis did not change much (12). This method is expected to yield results with an accuracy of ± 30 per cent (13). For given S, d, $\Delta\rho_s$, ρ, and the stage-hydraulic radius curve and stage-area curve of cross-section, the stage-discharge curve for an alluvial channel can be computed as follows:

(*i*) Assume a stage and find hydraulic radius R and area of cross-section A from stage-hydraulic radius and stage-area curves, respectively.

(*ii*) Determine K_1 and K_2 for known value of d using Fig. 7.7.

(*iii*) Compute $K_2\,(R/d)^{1/3}\,\dfrac{S}{\Delta\rho_s/\rho}$ and read the value of $K_1\,\dfrac{U}{\sqrt{(\Delta\rho_s/\rho)\,g\,R}}$ from Fig. 7.6.

(*iv*) Calculate the value of the mean velocity U and, hence, the discharge.

(*v*) Repeat the above steps for other values of stage.

Finally, a graphical relation between stage and discharge can be prepared.

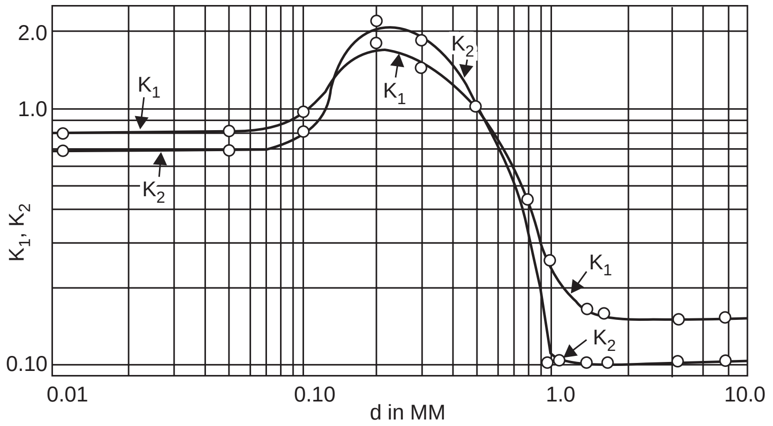

Fig. 7.7 Variation of K_1 and K_2 with sediment size (12)

Example 7.3 An alluvial stream (d = 0.60 mm) has a bed slope of 3×10^{-4}

Find the mean velocity of flow when the hydraulic radius is 1.40 m.

Solution:

From Fig. 7.7, $K_1 = 0.75$ and $K_2 = 0.70$

$$K_2\,(R/d)^{1/3}\,\frac{S}{\Delta\rho_s/\rho} = 0.70\left(\frac{1.40}{0.6\times10^{-3}}\right)^{1/3}\times\frac{3\times10^{-4}}{1.65} = 1.69\times10^{-3}$$

From Fig. 7.6,

$$K_1\,\frac{U}{\sqrt{(\Delta\rho_s/\rho)\,g\,R}} = 0.135 \qquad \text{or} \qquad 0.75\times\frac{U}{\sqrt{1.65\times9.81\times1.4}} = 0.135$$

$$\therefore \qquad U = 0.86 \text{ m/s.}$$

7.4.2. Resistance Relationship Based on Division of Resistance

In dealing with open channel flows, hydraulic radius R of the flow cross-section is taken as the characteristic depth parameter. The use of this parameter requires that the roughness over the whole wetted perimeter is the same. Such a condition can be expected in a very wide channel with alluvial bed and banks. However, laboratory flumes with glass walls and sand bed would have different roughnesses on the bed and side walls. In such cases, therefore, the hydraulic radius of the bed R_b is used instead of R in the resistance relations. The hydraulic radius of the bed R_b can be computed using Einstein's method (14) which assumes that the velocity is uniformly distributed over the whole cross-section. Assuming that the total area of cross-section of flow A can be divided into areas A_b and A_w corresponding to the bed and walls, respectively, one can write

$$A = A_w + A_b$$

For rectangular channels, one can, therefore, write

$$(B + 2h)\, R = 2\, hR_w + B\, R_b$$

$$\therefore \qquad R_b = \left(1 + \frac{2h}{B}\right) R - \frac{2h}{B} R_w$$

$$= (B + 2h)(R/B) - 2hR_w/B$$

$$= (PR/B) - 2hR_w/B$$

$$= (A/B) - 2hR_w/B$$

$$= h - 2hR_w/B \qquad (7.20)$$

Using Manning's equation for the walls, *i.e.,*

$$U = \frac{1}{n_w} R_w^{2/3} S^{1/2} \qquad (7.21)$$

one can calculate the hydraulic radius of the wall R_w if the Manning's coefficient for the walls, n_w is known. Using Eq. (7.20), the hydraulic radius of the bed R_b can be computed.

Example 7.4 A 0.40m wide laboratory flume with glass walls ($n_w = 0.01$) and mobile bed of 2.0 mm particles carries a discharge of 0.1 m³/s at a depth of 0.30m. The bed slope is 3×10^{-3}. Determine whether the particles would move or not. Neglect viscous effects.

Solution:

Hydraulic radius, $$R = \frac{0.3 \times 0.4}{0.4 + 2 \times 0.3} = 0.12 \text{ m}$$

Mean velocity of flow in the flume $$= \frac{0.1}{0.3 \times 0.4} = 0.833 \text{ m/s}$$

Using Eq. (7.21), $$R_w = \left(\frac{Un_w}{S^{1/2}}\right)^{3/2}$$

$$= \left(\frac{0.833 \times 0.01}{(3 \times 10^{-3})^{1/2}}\right)^{3/2}$$

$$= 0.0593 \text{ m}$$

Using Eq. (7.20), $R_b = 0.3 - \dfrac{2 \times 0.3}{0.4} \times 0.0593$

$= 0.211$ m

$\therefore$ Bed shear, $\tau_b = \rho g R_b S$

$= 9810 \times 0.211 \times 3 \times 10^{-3}$

$= 6.21$ N/m^2

On neglecting viscous effects and using Yalin and Karahan's curve,

$$\frac{\tau_c}{\Delta\rho_s g d} = 0.045$$

$\therefore$ Critical shear, $\tau_c = 0.045 \times 1.65 \times 9810 \times 2 \times 10^{-3}$

$= 1.457$ N/m^2

Since $\tau_b > \tau_c$, the particles would move.

Einstein and Barbarossa (15) obtained a rational solution to the problem of resistance in alluvial channels by dividing the total bed resistance (or shear) τ_{ob} into resistance (or shear) due to sand grains τ'_{ob} and resistance (or shear) due to the bed forms τ''_{ob}, *i.e.*,

$$\tau_{ob} = \tau'_{ob} + \tau''_{ob} \tag{7.22}$$

or

$$\rho g R_b S = \rho g R'_b S + \rho g R''_b S$$

i.e.,

$$R_b = R'_b + R''_b \tag{7.23}$$

where R'_b and R''_b are hydraulic radii of the bed corresponding to grain and form resistances (or roughnesses).

For a hydrodynamically rough plane boundary, the Manning's roughness coefficient for the grain roughness n_s is given by the Strickler's equation *i.e.,*

$$n_s = \frac{d_{65}^{\;1/6}}{24.0} \tag{7.24}$$

Here, d_{65} (in metres) represents the sieve diameter through which 65 per cent of the sediment will pass through, *i.e.,* 65 per cent of the sediment is finer than d_{65}. Therefore, Manning's equation can be written as

$$U = \frac{1}{n_s} R_b'^{2/3} S^{1/2}$$

$$U = \frac{24}{d_{65}^{\;1/6}} R_b'^{2/3} S^{1/2} \tag{7.25}$$

Since

$$U'_* = \sqrt{\tau'_{ob}/\rho} = \sqrt{g R'_b S}$$

$$\frac{U}{U'_*} = \frac{24}{d_{65}^{\;1/6}} \frac{R_b'^{2/3} S^{1/2}}{\sqrt{g R'_b S}}$$

or

$$\frac{U}{U'_*} = 7.66 \left(\frac{R'_b}{d_{65}}\right)^{1/6} \tag{7.26}$$

Einstein and Barbarossa (15) replaced this equation with the following logarithmic relation having theoretical support.

$$\frac{U}{U_*'} = 5.75 \log \frac{12.27\, R_b'}{d_{65}} \tag{7.27}$$

Equation (7.27) is valid for a hydrodynamically rough boundary. A viscous correction factor x (which is dependent on d_{65}/δ', Table 7.1, Fig. 7.8) was introduced in this equation to make it applicable to boundaries consisting of finer material ($d_{65}/\delta' < 10$). The modified equation is (15)

$$\frac{U}{U_*'} = 5.75 \log \frac{12.27\, R_b' x}{d_{65}} \tag{7.28}$$

Table 7.1 Variation of x with d_{65}/δ' (15)

d_{65}/δ'	0.2	0.3	0.5	0.7	1.0	2.0	4.0	6.0	10
x	0.7	1.0	1.38	1.56	1.61	1.38	1.10	1.03	1.0

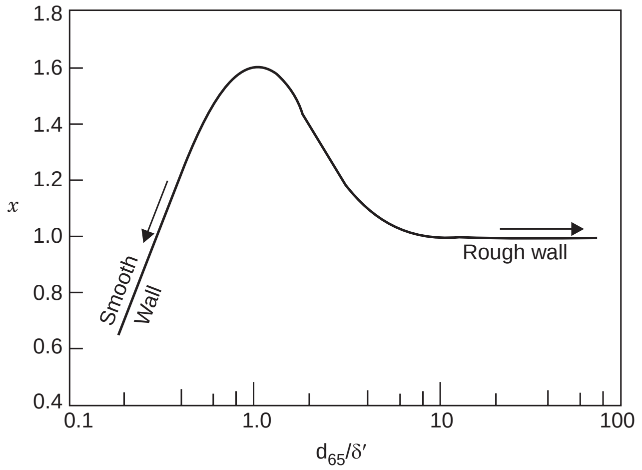

Fig. 7.8 Correction x in Eq. (7.28) (15)

Einstein and Barbarossa (15) recommended that one of the equations, Eq. (7.26) or Eq. (7.27) may be used for practical problems. The resistance (or shear) due to bed forms τ_{ob}'' is computed by considering that there are N undulations of cross-sectional area a in a length of channel L with total wetted perimeter P. Total form drag F on these undulation is given by

$$F = C_D a \left(\frac{1}{2}\right) \rho U^2 N \tag{7.29}$$

Here, C_D is the average drag coefficient of the undulations. Since this drag force acts on area LP, the average shear stress τ_{ob}'' will be given as

$$\tau_{ob}'' = \frac{F}{LP} = \frac{C_D a N}{LP} \rho \frac{U^2}{2}$$

$$\therefore \qquad \frac{\tau_{ob}''}{\rho} = U_*''^2 = \frac{C_D a N}{LP} \frac{U^2}{2}$$

or

$$\frac{U}{U_*''} = \sqrt{2LP/(C_D a N)} \tag{7.30}$$

Here, U_*'' is the shear velocity corresponding to bed undulations. According to Einstein and Barbarossa, the parameters on the right hand side of Eq. (7.30) would primarily depend on sediment transport rate which is a function of Einstein's parameter $\Psi' = \Delta\rho_s\, d_{35}/\rho\, R_b' S$. Therefore, they obtained an empirical relation, Fig. 7.9., between $\dfrac{U}{U_*''}$ and Ψ' using field data natural streams. The relationship proposed by Einstein and Barbarossa can be used to compute mean velocity of flow for a given stage (*i.e.,* depth of flow) of the river and also to prepare stage– discharge relationship.

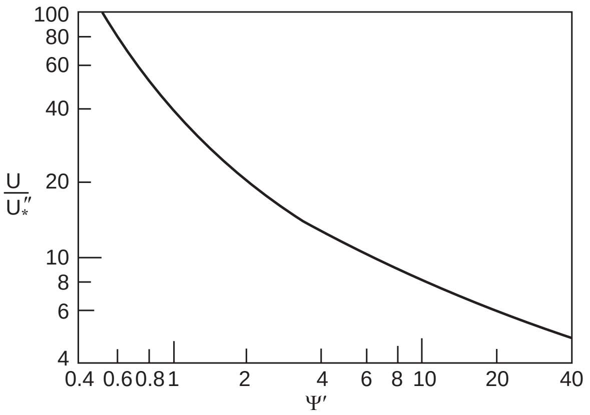

Fig. 7.9 Einstein and Barbarossa relation between U/U_*'' and Ψ' (15)

The computation of mean velocity of flow for a given stage requires a trial procedure. From the known channel characteristics, the hydraulic radius R of the flow area can be determined for a given stage (or depth of flow) of the river for which the mean velocity of flow is to be predicted. For a wide alluvial river, this hydraulic radius R approximately equals R_b. A value of R_b' smaller than R_b is assumed and a trial value of the mean velocity U is calculated from Eq. (7.25) or Eq. (7.26) or Eq. (7.27). The value of $\dfrac{U}{U_*''}$ is read from Fig. 7.9 for Ψ' corresponding to the assumed value of R_b'. From known values of U (trial value) and U/U_*'', U_*'' and, hence, R_b'' can be computed. If the sum of R_b' and R_b'' equals R_b the assumed value of R_b' and, hence, the corresponding mean velocity of flow U computed from Eq. (7.25) or Eq. (7.26) or Eq. (7.27) are okay. Otherwise, repeat the procedure for another trial value of R_b' till the sum of R_b' and R_b'' equals R_b. The computations can be carried out easily in a tabular form as illustrated in the following example:

Example 7.5 Solve Example 7.3 using Einstein and Barbarossa method.

Solution: For given d = 0.6 mm and bed slopes $S = 3 \times 10^{-4}$

$$U_*' = \sqrt{gR_b'S} = \sqrt{9.81 \times R_b' \times 3 \times 10^{-4}} = 0.054\sqrt{R_b'}$$

From Eq. (7.25)

$$U = 7.66\, U_*'(R_b'/d)^{1/6} = 7.66 \times 0.054\sqrt{R_b'}\left(\frac{R_b'}{0.6 \times 10^{-3}}\right)^{1/6}$$

$$\therefore \qquad U = 1.4243\, R_b'^{\,2/3}$$

$$\psi' = \frac{\Delta\rho_s d_{35}}{\rho R_b' S} = \frac{1.65 \times 0.6 \times 10^{-3}}{R_b'(3 \times 10^{-4})} = \frac{3.3}{R_b'}$$

From $$U_*'' = \sqrt{g R_b'' S}$$

$$R_b'' = \frac{(u_*'')^2}{gS} = \frac{(U_*'')^2}{9.81 \times 3 \times 10^{-4}} = 339.79\,(U_*'')^2$$

The trial procedure for computation of mean velocity can now be carried out in a tabular form. It is assumed that the alluvial river is wide and, therefore,

$$R_b \cong R$$

Trial No.	R_b' *(m)*	U_*' *(m/s)*	*U* *(m/s)*	ψ'	U/U_*''	U_*'' *(m/s)*	R_b'' *(m)*	R_b *(m)*	*Comments*
1	1.2	0.059	1.6084	2.75	16.0	0.1005	3.432	4.632	higher than 1.4
2	0.5	0.038	0.8973	6.60	10.5	0.0855	2.484	2.984	higher than 1.4
3	0.2	0.024	0.4871	16.50	7.0	0.0696	1.646	1.846	higher than 1.4
4	0.1	0.017	0.3069	33.00	5.0	0.0614	1.281	1.381	close to 1.4
5	0.11	0.018	0.3270	30.00	5.2	0.0629	1.344	1.444	higher than 1.4
6	0.105	0.175	0.3170	31.43	5.1	0.0622	1.315	1.420	close to 1.4

Values of R_b ($\cong R$) in row nos. 4 and 6 are reasonably close to the given value of 1.4 m. Thus, the velocity of flow is taken as the average of 0.3069 m/s and 0.3170 m/s *i.e.*, 0.312 m/s. The difference in the value of mean velocity obtained by Einstein and Barbarossa method compared with that obtained by Garde and Ranga Raju method (Example 7.3) should be noted.

For preparing a stage-discharge curve, one needs to obtain discharges corresponding to different stages of the river. If one neglects bank friction (*i.e.,* $R = R_b$), the procedure, requiring no trial, is as follows:

For an assumed value of R_b', the mean velocity of flow U is computed from Eq. (7.26) and U/U_*'' is read from Fig. 7.9 for Ψ' corresponding to the assumed value of R_b'. From known values of U and U/U_*'' one can determine U_*'' and, hence, R_b''. The sum of R_b' and R_b'' gives R_b which equals R (if bank friction is neglected). Corresponding to this value of R, one can determine the stage and, hence, the area of flow cross-section A. The product of U and A gives the discharge, Q corresponding to the stage. Likewise, for another value of R_b', one can determine stage and the corresponding discharge.

7.5. TRANSPORT OF SEDIMENT

When the average shear stress τ_o on the bed of an alluvial channel exceeds the critical shear τ_c, the sediment particles start moving in different ways depending on the flow condition, sediment size, fluid and sediment densities, and the channel condition.

At relatively low shear stresses, the particles roll or slide along the bed. The particles remain in continuous contact with the bed and the movement is generally discontinuous. Sediment material transported in this manner is termed *contact load.*

On increasing the shear stress, some sediment particles lose contact with the bed for some time, and 'hop' or 'bounce'. The sediment particles moving in this manner fall into the category of *saltation load.* This mode of transport is significant only in case of noncohesive materials of relatively high fall velocities such as sand in air and, to a lesser extent, gravel in water.

Since saltation load is insignificant in case of flow of water and also because it is difficult to distinguish between saltation load and contact load, the two are grouped together and termed *bed load,* which is transported on or near the bed.

With further increase in the shear stress, the particles may go in suspension and remain so due to the turbulent fluctuations. The particles in suspension move downstream. Such sediment material is included in the *suspended load.* Sediment particles move in suspension when $u_*/w_o > 0.5$. Here, w_o is the fall velocity for sediment particles of given size.

The material for bed load as well as a part of the suspended load originates from the bed of the channel and, hence, both are grouped together and termed *bed-material load*.

Analysis of suspended load data from rivers and canals has shown that the suspended load comprises the sediment particles originating from the bed and the sediment particles which are not available in the bed. The former is the bed-material load in suspension and the latter is the product of erosion in the catchment and is appropriately called *wash load.* The wash load, having entered the stream, is unlikely to deposit unless the velocity (or the shear stress) is greatly reduced or the concentration of such fine sediments is very high. The transport rate of wash load is related to the availability of fine material in the catchment and its erodibility and is, normally, independent of the hydraulic characteristics of the stream. As such, it is not easy to make an estimate of wash load.

When the bed-material load in suspension is added to the bed-material load moving as bed load, one gets the total bed-material load which may be a major or minor fraction of the total load comprising bed-material load and wash load of the stream depending on the catchment characteristics.

Irrigation channels carrying silt-laden water and flowing through alluvial bed are designed to carry certain amounts of water and sediment discharges. This means that the total sediment load transport will affect the design of an alluvial channel. Similarly, problems related to reservoir sedimentation, aggradation, degradation, etc. can be solved only if the total sediment load being transported by river (or channel) is known. One obvious method of estimation of total load is to determine bed load, suspended load, and wash load individually and then add these together. The wash load is usually carried without being deposited and is also not easy to estimate. This load is, therefore, ignored while analysing channel stability.

It should, however, be noted that the available methods of computation of bed-material load are such that errors of the order of one magnitude are not uncommon. If the bed-material load is only a small fraction of the total load, the foregoing likely error would considerably reduce the validity of the computations. This aspect of sediment load computations must always be kept in mind while evaluating the result of the computations.

7.5.1. Bed Load

The prediction of the bed load transport is not an easy task because it is interrelated with the resistance to flow which, in turn, is dependent on flow regime. Nevertheless, several attempts have been made to propose methods – empirical as well as semi-theoretical – for the computation of bed load. The most commonly used empirical relation is given by Meyer-Peter and Müller (16). Their relation is based on: (*i*) the division of total shear into grain shear and form shear, and (*ii*) the premise that the bed load transport is a function of only the grain shear. Their equation, written in dimensionless form, is as follows:

$$\left[\frac{n_s}{n}\right]^{3/2}\frac{\rho RS}{\Delta\rho_s d_a} = 0.047 + 0.25\,\frac{{q_B}^{2/3}}{{\rho_s}^{2/3} g d_a \left(\dfrac{\Delta\rho_s}{\rho}\right)^{1/3}} \qquad (7.31)$$

which is rewritten as

$$\tau_*' = 0.047 + 0.25\,\phi_B^{2/3} \qquad (7.32)$$

or

$$\phi_B = 8\,(\tau_*' - 0.047)^{3/2} \qquad (7.33)$$

where, ϕ_B is the bed load function and equals $\dfrac{q_B}{\rho_s g^{3/2}\, {d_a}^{3/2}\sqrt{\Delta\rho_s/\rho}}$,

τ_*' is the dimensionless grain shear and equals $\dfrac{\tau_o'}{\Delta\rho_s g d_a}$,

and τ_o' is the grain shear and equals $\left[\dfrac{n_s}{n}\right]^{3/2}\rho g RS$.

Here, q_B is the rate of bed load transport in weight per unit width, *i.e.,* N/m/s and d_a is the arithmetic mean size of the sediment particles which generally varies between d_{50} and d_{60}(1).

From Eq. (7.33), it may be seen that the value of the dimensionless shear τ_*' at the incipient motion condition (*i.e.,* when q_B and, hence, ϕ_B is zero) is 0.047. Thus, $(\tau_*' - 0.047)$ can be interpreted as the effective shear stress causing bed load movement.

The layer in which the bed load moves is called the bed layer and its thickness is generally taken as $2d$.

Example 7.6 Determine the amount of bed load in Example 7.2

Solution:

From the solution of Example 7.2,

$$\tau_*' = 0.21$$

From Eq. (7.33),

$$\therefore \quad \phi_B = 8 \times (0.21 - 0.047)^{3/2} = 0.5265$$

i.e.,

$$\frac{q_B}{\rho_s g^{3/2} d^{3/2}\sqrt{\Delta\rho_s/\rho}} = 0.5265$$

$$\therefore \quad q_B = 0.5265 \times 2650 \times (9.81 \times 0.3 \times 10^{-3})^{1.5}\,(1.65)^{1/2} \text{ N/m/s}$$

$$= 0.286 \text{ N/m/s}$$

A semi-theoretical analysis of the problem of the bed load transport was first attempted by Einstien (14) in 1942 when he did not consider the effect of bed forms on bed load transport. Later, he presented a modified solution (17) to the problem of bed load transport. Einstein's solution does not use the concept of critical tractive stress but, instead, is based on the assumption that a sediment particle resting on the bed is set in motion when the instantaneous hydrodynamic lift force exceeds the submerged weight of the particle. Based on his semi-theoretical analysis, a curve, Fig. 7.9, between the Einstein's bed load parameter

$$\phi_B\left(= \frac{q_B}{\rho_s g^{3/2} d^{3/2}\sqrt{\Delta\rho_s/\rho}}\right) \quad \text{and} \quad \psi'\ (= \Delta\rho_s\, d/\rho R'S)$$

can be used to compute the bed load transport in case of uniform sediment. The coordinates of the curve of Fig. 7.10 are given in Table 7.2. The method involves computation of Ψ' for given sediment characteristics and flow conditions and reading the corresponding value of ϕ_B from Fig. 7.10 to obtain the value of q_B.

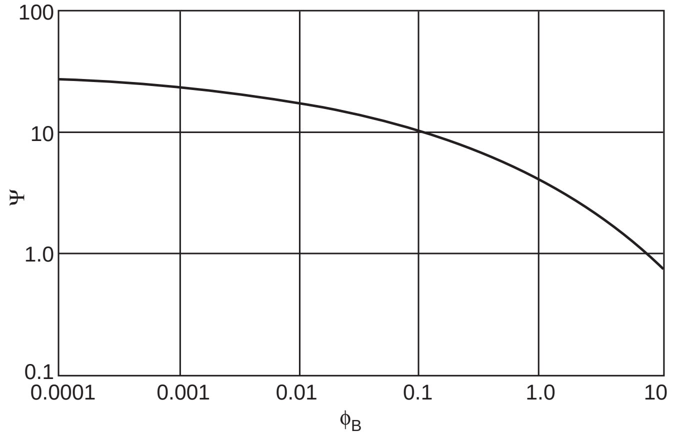

Fig. 7.10 Einstein's bed load transport relation (17)

Table 7.2 Relationship between ϕ_B and Ψ' (17)

ψ'	27.0	24.0	22.4	18.4	16.4	11.5	9.5	5.5	4.08	1.4	0.70
ϕ_B	10^{-4}	5×10^{-4}	10^{-3}	5×10^{-3}	10^{-2}	5×10^{-2}	10^{-1}	5×10^{-1}	1.0	5.0	10.0

Example 7.7 Determine the amount of bed load in Example 7.2 using Einstein's method.

Solution: From the solution of Example 7.2,

$$R' = 0.651 \text{ m}$$

$$\therefore \qquad \psi' = \frac{\Delta\rho_s d}{\rho R' S}$$

$$= \frac{1.65 \times 0.3 \times 10^{-3}}{0.651 \times 1.6 \times 10^{-4}}$$

$$= 4.752$$

$$\therefore \qquad \phi_B = 0.763 \quad \text{(from Fig. 7.10)}$$

$$\therefore \qquad q_B = 0.763 \times 2650 \times (9.81 \times 0.3 \times 10^{-3})^{1.5} (1.65)^{1/2}$$

$$= 0.415 \text{ N/m/s}$$

7.5.2. Suspended Load

At the advanced stage of bed load movement the average shear stress is relatively high and finer particles may move into suspension. With the increase in the shear stress, coarser fractions of the bed material will also move into suspension. The particles in suspension move with a velocity almost equal to the flow velocity. It is also evident that the concentration of sediment particles will be maximum at or near the bed and that it would decrease as the distance from the bed increases. The concentration of suspended sediment is generally expressed as follows:

(*i*) *Volume concentration:* The ratio of absolute volume of solids and the volume of sediment-water mixture is termed the volume concentration and can be expressed as percentage by volume. 1 % of volume concentration equals 10,000 ppm by volume.

(*ii*) *Weight concentration:* The ratio of weight of solids and the weight of sediment-water mixture is termed the weight concentration and is usually expressed in parts per million (ppm).

Variation of Concentration of Suspended Load

Starting from the differential equation for the distribution of suspended material in the vertical and using an appropriate diffusion equation, Rouse (18) obtained the following equation for sediment distribution (*i.e.,* variation of sediment concentration along a vertical):

$$\frac{C}{C_a} = \left(\frac{h-y}{y} \times \frac{a}{h-a}\right)^{Z_o} \tag{7.34}$$

where, C = the sediment concentration at a distance y from the bed,

C_a = the reference concentration at $y = a$,

h = the depth of flow,

$Z_o = \dfrac{w_o}{U_* k}$ and is the exponent in the sediment distribution equation,

w_o = the fall velocity of the sediment particles,

and k = Karman's constant.

Rouse's equation, Eq. (7.34), assumes two-dimensional steady flow, constant fall velocity and fixed Karman's constant. However, it is known that the fall velocity as well as Karman's constant vary with concentration and turbulence. Further, a knowledge of some reference concentration C_a at $y = a$ is required for the use of Eq. (7.34).

Knowledge of the velocity distribution and the concentration variation (Fig. 7.11) would enable one to compute the rate of transport of suspended load q_s. Consider a strip of unit width and thickness dy at an elevation y. The volume of suspended load transported past this strip in a unit time is equal to $\dfrac{1}{100}$ *Cudy*.

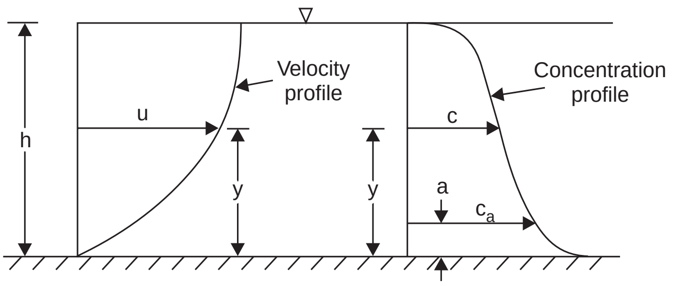

Fig. 7.11 Variation of velocity of flow and sediment concentration in a vertical

Here, C is the volume concentration (expressed as percentage) at an elevation y where the velocity of flow is u. Thus,

$$q_s = \frac{\rho_s g}{100} \int_a^h Cudy \tag{7.35}$$

where, q_s is the weight of suspended load transported per unit width per unit time. Since the suspended sediment moves only on top of the bed layer, the lower limit of integration, a, can be considered equal to the thickness of the bed layer, *i.e.*, $2d$.

Instead of using the curves of the type shown in Fig. 7.11, one may use a suitable velocity distribution law and the sediment distribution equation, Eq. (7.34). For the estimation of the reference concentration C_a appearing in Eq. (7.34), Einstein (17) assumed that the average concentration of bed load in the bed layer equals the concentration of suspended load at $y = 2d$. This assumption is based on the fact that there will be continuity in the distribution of suspended load and bed load. Making use of suitable velocity distribution laws, the velocity of the bed layer was determined as $11.6\, u_*'$ and as such the concentration in the bed layer was obtained as $\dfrac{(q_B/\rho_s g)}{(11.6U_*')(2d)}$. Hence, the reference concentration C_a (in per cent) at $y = 2d$ is given as

$$C_a = C_{2d} = \frac{(q_B/\rho_s g)}{(23.2U_*')(d)} \times 100 \tag{7.36}$$

Equation (7.35) can now be integrated in a suitable manner.

Example 7.8 Prepare a table for the distribution of sediment concentration in the vertical for Example 7.2. Assume fall velocity of the particles as 0.01 m/s.

Solution: From the solution of Example 7.2 and 7.3,

$$q_B = 0.286 \text{ N/m/s}$$

and $$R' = 0.651 \text{ m}$$

Using Eq. (7.36)

$$C_a = \frac{(q_B / \rho_s g)}{23.2U_*'d} \times 100$$

$$= \frac{0.286 \times 100}{(2650 \times 9.81) \times 23.2 \times (9.81 \times 0.651 \times 1.6 \times 10^{-4})^{1/2} (0.3 \times 10^{-3})}$$

$$= 5\%$$

Now $$\frac{C}{C_a} = \left[\frac{h-y}{y} \times \frac{a}{h-a}\right]^{Z_o}$$

$$a = 2d = 2 \times 0.3 \times 10^{-3} \text{ m}$$

$$Z_o = \frac{w_o}{U_* k} = \frac{0.01}{(9.81 \times 2.5 \times 1.6 \times 10^{-4})^{1/2} \times 0.4} = 0.4$$

$$\therefore \quad \frac{C}{5.0} = \left[\frac{2.8-y}{y} \times \frac{2 \times 0.3 \times 10^{-3}}{2.8 - (2 \times 0.3 \times 10^{-3})}\right]^{0.4}$$

$$\therefore \quad C = 0.17\left(\frac{2.8-y}{y}\right)^{0.4}$$

The variation of C with y can now be computed as shown in the following table:

y (m)	0.1	0.2	0.5	1.0	1.5	2.0	2.5	2.7	2.8
C (%)	0.635	0.474	0.313	0.215	0.161	0.118	0.0728	0.0455	0
C (ppm)	6350	4740	3130	2150	1610	1180	728	455	0

7.5.3. Total Bed-Material Load

The total bed-material load can be determined by adding together the bed load and the suspended load. There is, however, another category of methods too for the estimation of the total bed-material load. The supporters of these methods argue that the process of suspension is an advanced stage of tractive shear along the bed, and, therefore, the total load should be related to the shear parameter. One such method is proposed by Engelund and Hansen (19) who obtained a relationship for the total bed-material load q_T (expressed as weight per unit width per unit time) by relating the sediment transport to the shear stress and friction factor f. The relationship is expressed as

$$f\phi_T = 0.4\,\tau_*^{5/2} \tag{7.37}$$

where,

$$\phi_T = \frac{q_T}{\rho_s g^{3/2} d^{3/2} \sqrt{\Delta\rho_s/\rho}} \tag{7.38}$$

and

$$f = \frac{8gRS}{U^2} \tag{7.39}$$

The median size d_{50} is used for d in the above equation.

Example 7.9 Determine the total bed-material transport rate for Example 7.2.

Solution: Using Eqs. (7.37) and (7.39)

$$f\phi_T = 0.4\,\tau_*^{5/2}$$

$$\therefore \quad \frac{8gRS}{U^2}\phi_T = 0.4\left(\frac{\rho RS}{\Delta\rho_s d}\right)^{5/2}$$

or

$$\phi_T = 0.4\left[\frac{2.5\times1.6\times10^{-4}}{1.65\times0.3\times10^{-3}}\right]^{5/2}\times\left[\frac{0.95\times0.95}{8\times9.81\times2.5\times1.6\times10^{-4}}\right]$$

$$\therefore \quad \frac{q_T}{\rho_s g^{3/2} d^{3/2}\sqrt{\Delta\rho_s/\rho}} = 6.75$$

$$\therefore \quad q_T = 6.75\times2650\times(9.81\times0.3\times10^{-3})^{3/2}\,(1.65)^{1/2}$$
$$= 3.67 \text{ N/m/s}$$

EXERCISES

7.1 What is the meaning of the term 'incipient motion of sediment'? How would you calculate critical tractive stress for given sediment?

7.2 Describe various bed forms that occur in alluvial channels.

7.3 Describe various modes of sediment transport in an alluvial channel.

7.4 Determine the depth of flow which will cause incipient motion condition in a wide channel having a mean sediment size of 5.0 mm and a channel slope of 0.0004. The specific gravity of the material is 2.65. Also determine the corresponding velocity of flow.

7.5 Find the depth and velocity of flow at which the bed material of average size 5.0 mm will just move in a wide rectangular channel at a slope of 1×10^{-3}. Assume specific gravity of the material to be 2.65 and neglect viscous effects.

7.6 Water flows at a depth of 0.20 m in a wide flume having a slope of 1×10^{-4}. The median diameter of the sand placed on the flume bed is 1.0 mm. Determine whether the sand grains are stationary or moving.

7.7 An irrigation canal has been designed to have hydraulic radius equal to 2.50 m, depth of flow equal to 2.80 m, and bed slope equal to 1.5×10^{-4}. The sediment on the canal bed has median size of 0.25 mm. Find: (*i*) the bed condition that may be expected, (*ii*) height and length of bed forms, and (*iii*) the advance velocity of the bed forms.

7.8 For an alluvial stream, the average width is 120.0 m and the cross-section may be considered as rectangular. The longitudinal slope of the stream is 0.0002. Prepare a stage-discharge curve up to a depth of 4.0 m. The size and specific gravity of the sediment are 0.30 mm and 2.65, respectively.

7.9 Determine the bed load transport in a wide alluvial stream for the following conditions:

Depth of flow = 2.50 m
Velocity of flow = 1.50 m/s
Average slope of water surface = 8×10^{-4}
Mean size of sediment = 5.0 mm
Specific gravity of the sediment = 2.65

7.10 Determine the rate of bed load transport in a wide alluvial stream for the following data:

Depth of flow = 4.50 m
Velocity of flow = 1.30 m/s
Slope = 2.0×10^{-4}

Size distribution of the sediment is as follows:

d (mm)	0.20	0.44	0.78	1.14	1.65	2.6	5.20
% finer	2.0	10.0	30.0	50.0	70.0	80.0	100.0

7.11 For an alluvial stream having a slope of 0.00015 and depth of flow equal to 2.40 m, the following velocity profile was observed:

y (m)	0.215	0.30	0.425	0.670	0.885	1.035	1.28	1.77	2.07	2.35
Velocity (m/s)	1.31	1.37	1.45	1.56	1.65	1.66	1.68	1.69	1.70	1.65

If the fall velocity for the average size of the suspended load is 8.00 mm/s, plot the distribution of suspended load in a vertical section. Assume Karman's constant k equal to 0.4 and the concentration of sediment at $y = 0.215$ m as equal to 4 N/litre.

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8

DESIGN OF STABLE CHANNELS

8.1. GENERAL

Surface water for irrigation is conveyed from its source to the field by means of canals or channels. These channels generally have alluvial boundaries and carry sediment-laden water. A hydraulic engineer is concerned with the design, construction, operation, maintenance, and improvement of such channels.

Lane (1) gave the definition of stable channel as follows:

"A *stable channel* is an unlined earth channel : (*a*) which carries water, (*b*) the banks and bed of which are not scoured objectionably by moving water, and (*c*) in which objectionable deposits of sediment do not occur". This means that over a long period, the bed and banks of a stable channel remain unaltered even if minor deposition and scouring occur in the channel. Obviously, silting and scouring in a stable channel balance each other over a long period of time.

An irrigation channel can have either a rigid boundary or one consisting of alluvial material. These channels may have to carry either clear water or sediment-laden water. Accordingly, there can be four different types of problems related to the design of a stable channel.

(*i*) Rigid-boundary (*i.e.*, non-erodible) channels carrying clear water,

(*ii*) Rigid-boundary channels carrying sediment-laden water,

(*iii*) Alluvial channels carrying clear water, and

(*iv*) Aluvial channels carrying sediment-laden water.

The design of a stable channel aims at obtaining the values of mean velocity, depth (or hydraulic radius), width and slope of the channel for known values of discharge Q, sediment discharge Q_T, sediment size d, and the channel roughness characteristics without causing undue silting or scouring of the channel bed.

8.2. RIGID BOUNDARY CHANNELS CARRYING CLEAR WATER

Channels of this type have rigid, *i.e.*, non-erodible, boundaries like rock cuts or artificial lining. These channels reduce the seepage loss and thus conserve water and reduce the water logging of the lands adjacent to the channel. Cost of operation and maintenance of lined channels are also less. Lined channel sections are relatively more stable.

The design of such channels is based on the Manning's equation combined with the continuity equation. Thus,

$$Q = \frac{1}{n} AR^{2/3} S^{1/2} \tag{8.1}$$

in which, A is the area of cross-section of flow and the hydraulic radius $R = A/P$. Here, P is the wetted perimeter. Equation (8.1) can also be written as

$$Q = \frac{A^{5/3} S^{1/2}}{nP^{2/3}} \tag{8.2}$$

This means that for specified values of Manning's n and the slope S, the discharge Q is maximum for a given area of cross-section A when the wetted perimeter P is minimum. A channel section with the minimum wetted perimeter for a given cross-sectional area A is said to be the most efficient hydraulic section or, simply, the best hydraulic section. Since a circle has the least perimeter for a given area, the circular section is the most efficient section. But, construction difficulties in having a circular section rule out the possibilities of having a circular channel in most cases. Thus, the problem reduces to determining the geometric elements of the most efficient hydraulic section for a specified geometric shape.

Considering a rectangular section of width B and depth of flow h, one can determine the dimensions of the most efficient rectangular section as follows:

$$A = Bh$$

$$P = B + 2h$$

$$= \frac{A}{h} + 2h$$

For P to be minimum, $$\frac{dP}{dh} = -\frac{A}{h^2} + 2 = 0$$

i.e., $$A = 2h^2$$

or $$B = 2h$$

$\therefore$ $$P = 4h$$

Thus, $$R = \frac{h}{2}$$

Likewise, such relationships for the geometric elements of other shapes, summarized in Table 8.1, can be determined.

In practice, the sharp corners in a cross-section are rounded so that these may not become zones of stagnation. Sometimes, the side slopes may also have to be adjusted depending upon the type of bank soil. In India, lined channels carrying discharges less than 55 m^3/s are generally of triangular section of the permissible side slope and rounded bottom (Fig. 8.1). A trapezoidal section with rounded corners (Fig. 8.2) is adopted for lined channels carrying discharges larger than 55 m^3/s. The side slopes depend on the properties of the material through which the channel is to pass. Table 8.2 gives the recommended values of the side slopes for channels excavated through different types of material.

To avoid damage to the lining, the maximum velocity in lined channels is restricted to 2.0 m/s. Thus, the design is based on the concept of a limiting velocity. The expressions for geometric elements of the lined channel section, Figs. 8.1 and 8.2, can be written as follows:

Table. 8.1 Geometric elements of the most efficient hydraulic sections (2)

Shape of cross-section	*Area A*	*Wetted perimeter P*	*Hydraulic radius R*	*Water surface width T*	*Hydraulic depth D*
Rectangle	$2h^2$	$4h$	$0.500h$	$2h$	h
Triangle (side slope 1 : 1)	h^2	$2.83h$	$0.354h$	$2h$	$0.500h$
Trapezoid (half of hexagon)	$1.73h^2$	$3.46h$	$0.500h$	$2.31h$	$0.750h$
Semicircle	$0.5\ \pi h^2$	πh	$0.500h$	$2h$	$0.250\ \pi h$
Parabola ($T = 2\sqrt{2h}$)	$1.89h^2$	$3.77h$	$0.500h$	$2.83h$	$0.667h$

Table 8.2 Suitable side slopes for channels excavated through different types of material (3)

Material	*Side slopes (H : V)*
Rock	Nearly vertical
Muck and peat soil	0.25 : 1
Stiff clay or earth with concrete lining	0.5 : 1 to 1 : 1
Earth with stone lining	1 : 1
Firm clay	1.5 : 1
Loose, sandy soil	2 : 1

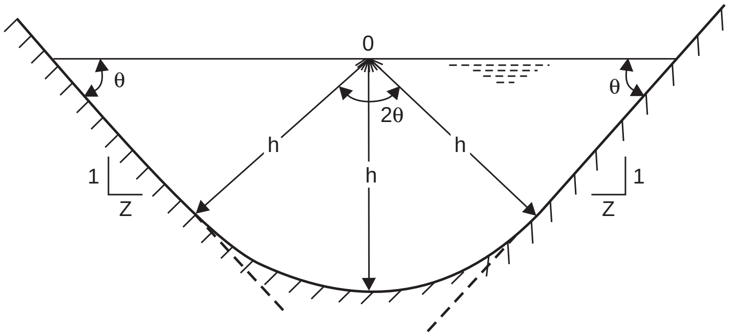

Fig. 8.1 Lined channel section for $Q < 55\ m^3/s$

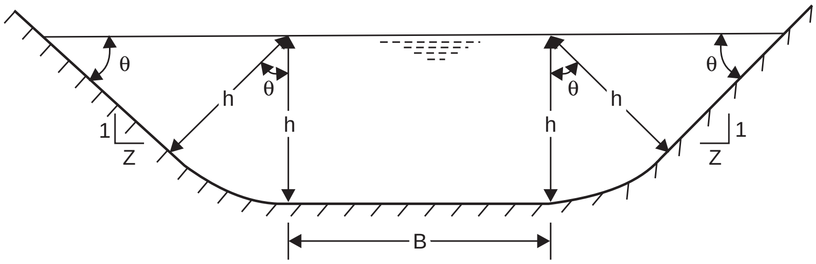

Fig. 8.2 Lined channel section for $Q > 55\ m^3s$

For triangular section, Fig. 8.1,

Area, $$A = 2\left(\frac{1}{2}h^2 \cot\theta\right) + \frac{1}{2}h^2(2\theta)$$

$$= h^2(\theta + \cot\theta)$$

Wetted perimeter, $$P = 2h\cot\theta + h(2\theta)$$

$$= 2h(\theta + \cot\theta)$$

Hydraulic radius $$R = \frac{A}{P} = \frac{h^2(\theta + \cot\theta)}{2h(\theta + \cot\theta)} = \frac{h}{2}$$

Similarly, for trapezoidal section, Fig. 8.2,

$$A = Bh + 2\left(\frac{1}{2}h^2 \cot\theta\right) + 2\left(\frac{1}{2}h^2\theta\right)$$

$$= Bh + h^2(\theta + \cot\theta)$$

$$P = B + 2h(\theta + \cot\theta)$$

$$R = \frac{Bh + h^2(\theta + \cot\theta)}{B + 2h(\theta + \cot\theta)}$$

In all these expressions for A, P, and R, the value of θ is in radians. For designing a lined channel, one needs to solve these equations alongwith the Manning's equation. For given Q, n, S, and A, and R expressed in terms of h for known Q, the Manning's equation will yield, for triangular section, an explicit relation for h as shown below :

$$Q = \frac{1}{n}[h^2(\theta + \cot\theta)]\left[\frac{h}{2}\right]^{2/3} S^{1/2}$$

$$\therefore \qquad h = \left[\frac{nQ(2)^{2/3}}{\sqrt{S}(\theta + \cot\theta)}\right]^{3/8} \tag{8.3}$$

However, in case of trapezoidal section, Fig. 8.2, the design calculations would start with an assumed value of velocity (less than the maximum permissible velocity of 2.0 m/s) and the expression for h will be in the form of a quadratic expression as can be seen from the following :

From the Manning's equation

$$R = \left(\frac{Un}{\sqrt{S}}\right)^{3/2}$$

$$\therefore \qquad \left(\frac{Un}{\sqrt{S}}\right)^{3/2}[B + 2h(\theta + \cot\theta)] = A = \frac{Q}{U}$$

$$\therefore \qquad B = \frac{Q}{U}\left(\frac{\sqrt{S}}{Un}\right)^{3/2} - 2h(\theta + \cot\theta)$$

On substituting this value of B in the expression for area of flow A, one gets,

$$h\frac{Q}{U}\left(\frac{\sqrt{S}}{Un}\right)^{3/2} - 2h^2(\theta + \cot\theta) + h^2(\theta + \cot\theta) = A = \frac{Q}{U}$$

$$\therefore \qquad h^2 (\theta + \cot \theta) - h \frac{Q}{U}\left(\frac{\sqrt{S}}{Un}\right)^{3/2} + \frac{Q}{U} = 0$$

$$\therefore \qquad h = \frac{\frac{Q}{U}\left(\frac{\sqrt{S}}{Un}\right)^{3/2} \pm \sqrt{\frac{Q^2}{U^2}\left(\frac{\sqrt{S}}{Un}\right)^{3} - 4(\theta + \cot\theta)\frac{Q}{U}}}{2(\theta + \cot\theta)}$$

Therefore, in order to have a feasible solution,

$$\frac{Q^2}{U^2}\left(\frac{\sqrt{S}}{Un}\right)^{3} \geq 4 (\theta + \cot \theta) \frac{Q}{U}$$

i.e.,

$$\frac{S^{3/2}}{U^4} \geq \frac{4n^3 (\theta + \cot\theta)}{Q}$$

or

$$U \leq \left[\frac{QS^{3/2}}{4n^3(\theta + \cot\theta)}\right]^{1/4} \qquad (8.4)$$

This means that for designing a trapezoidal section for a lined channel, the velocity will have to be suitably chosen so as not to violate the above criterion in order to have a feasible solution (see Example 8.2 for illustration).

Example 8.1 A lined canal (n = 0.015) laid at a slope of 1 in 1600 is required to carry a discharge of 25 m^3/s. The side slopes of the canal are to be kept at 1.25 H : 1 V. Determine the depth of flow.

Solution: Since Q < 55 m^3/s, a triangular section with rounded bottom, Fig. 8.1, is considered suitable. Here,

$$\cot \theta = 1.25$$

$$\therefore \qquad \theta = 38.66° \quad \text{or} \quad 0.657 \text{ radian}$$

Thus, from Fig. 8.1, $A = h^2 (\theta + \cot \theta) = h^2 (0.675 + 1.25)$

$$= 1.925h^2$$

and

$$P = 2h(\theta + \cot\theta) = 2h(0.675 + 1.25) = 3.85h$$

$$\therefore \qquad R = \frac{1.925h^2}{3.85h} = \frac{h}{2}$$

From the Manning's equation,

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

Hence,

$$25 = \frac{1.925h^2}{0.015}\left(\frac{h}{2}\right)^{2/3}\left(\frac{1}{1600}\right)^{1/2}$$

$$\therefore \qquad h^{8/3} = \frac{25 \times 0.015 \times 2^{2/3} \times 40}{1.925}$$

$$\therefore \qquad h = 2.57 \text{ m.}$$

Example 8.2 Design a lined channel to carry a discharge of 300 m^3/s through an alluvium whose angle of repose is 31°. The bed slope of the channel is 7.75×10^{-5} and Manning's n for the lining material is 0.016.

Solution: Since $Q > 55$ m^3/s, trapezoidal section with rounded corners, Fig. 8.2, is to be designed. Here,

Side slope $\theta = 31° = 0.541$ radians

$\therefore$ $\cot\theta = 1.664$

$\therefore$ $\theta + \cot\theta = 2.205$

$\therefore$ $A = Bh + 2.205\,h^2$

$P = B + 4.41h$

Adopting $U = 2$ m/s

$$A = \frac{300}{2} = 150 \text{ m}^2$$

$\therefore$ $Bh + 2.205h^2 = 150$

and
$$R = \left(\frac{Un}{\sqrt{S}}\right)^{3/2} = \left(\frac{2 \times 0.016}{\sqrt{7.75 \times 10^{-5}}}\right)^{3/2} = 6.93 \text{ m}$$

$\therefore$ $6.93\,(B + 4.41h) = 150$

or $B = 21.645 - 4.41h$

$\therefore$ $21.645h - 4.41h^2 + 2.205h^2 = 150$

or $2.205h^2 - 21.645h + 150 = 0$

$\therefore$
$$h = \frac{21.645 \pm \sqrt{(21.645)^2 - 4 \times 2.205 \times 150}}{4.41}$$

Obviously, the roots of h are imaginary. Using the criterion, Eq. (8.4), one gets,

$$U \le \left[\frac{QS^{3/2}}{4n^3(\theta + \cot\theta)}\right]^{1/4}$$

$$\le \left[\frac{300 \times (7.75 \times 10^{-5})^{3/2}}{4(0.016)^3(2.205)}\right]^{1/4}$$

$$\le 1.543 \text{ m/s}$$

$\therefore$ Adopt $U = 1.5$ m/s

$\therefore$ $A = 200$ m^2

$$R = \left(\frac{1.5 \times 0.016}{\sqrt{7.75 \times 10^{-5}}}\right)^{3/2} = 4.50 \text{ m}$$

$4.50(B + 4.41h) = 200$

$\therefore$ $B = 44.44 - 4.41h$

Again, using $Bh + 2.205h^2 = A$, one gets,

$$44.44h - 4.41h^2 + 2.205h^2 = 200$$

or $$2.205h^2 - 44.44h + 200 = 0$$

$$\therefore \quad h = \frac{44.44 \pm \sqrt{(44.44)^2 - 4 \times 2.205 \times 200}}{4.41}$$

$$= 13.37 \text{ m} \quad \text{or} \quad 6.784 \text{ m}$$

$$\therefore \quad B = 44.44 - 4.41h$$

$$= 14.52 \text{ m for } h = 6.784 \text{ m}$$

Other value of h (= 13.37 m) gives negative value of B which is meaningless.

$$\therefore \quad B = 14.52 \text{ m} \quad \text{and} \quad h = 6.784 \text{ m.}$$

8.3. RIGID BOUNDARY CHANNELS CARRYING SEDIMENT–LADEN WATER

These channels are to be designed in such a way that the sediment in suspension does not settle on the channel boundary. The design is, therefore, based on the concept of minimum permissible velocity which will prevent both sedimentation as well as growth of vegetation. In general, velocities of 0.7 to 1.0 m/s will be adequate for this purpose if the sediment load is small. If the sediment concentration is large, Fig. 8.3 can be used to ensure that the sediment does not deposit. In Fig. 8.3, C_s is the concentration of sediment in ppm (by volume), f_b the friction factor of the channel bed, D_o the central depth, T the top width and S_c equals $S/(\Delta\rho_s/\rho)$. If the designed channel section is not able to carry the specified sediment load, the slope S of the channel is increased.

Example 8.3 *A rectangular channel 5 m wide is to carry 2.5 m³/s on a slope of 1 in 2000 at a depth of 0.75 m. It is expected that fine silt of 0.04 mm size will enter the channel. What is the maximum concentration of this sediment that can be allowed into the channel without causing objectionable deposition ? Assume that the fall velocity of the given sediment in water is 1.5 mm/s, kinematic viscosity of water is 10^{-6} m²/s, and specific gravity of the sediment is 2.65.*

Solution: Since the wall and the bed of the channel are of the same material, the friction factor for the channel bed f_b is given as

$$f_b = \frac{8gRS}{U^2}$$

Since $$U = \frac{2.5}{5 \times 0.75} = 0.7143 \text{ m/s}$$

and $$R = \frac{5 \times 0.75}{5 + 2 \times 0.75} = 0.5769 \text{ m}$$

$$\therefore \quad f_b = \frac{8 \times 9.81 \times 0.5769 \times (1/2000)}{(0.7143)^2} = 0.0444$$

$$S_c = S/(\Delta\rho_s/\rho) = \frac{1}{2000 \times 1.65} = \frac{1}{3300}$$

Now $$\frac{qS_c^{2.5}}{\nu f_b^2}\,\frac{1}{(w_0\,d/\nu)^{0.6}}\left(\frac{A}{TD_o}\right)^2$$

$$= \frac{(2.5/5.0)(1/3300)^{2.5}}{(10^{-6})(0.0444)^2(1.5\times10^{-3}\times0.4\times10^{-3}/10^{-6})^{0.6}}\left(\frac{5\times0.75}{5\times0.75}\right)^2 = 2.2$$

Therefore, from Fig. 8.3, maximum concentration for no deposition,

$$C_s = 500 \text{ ppm}$$

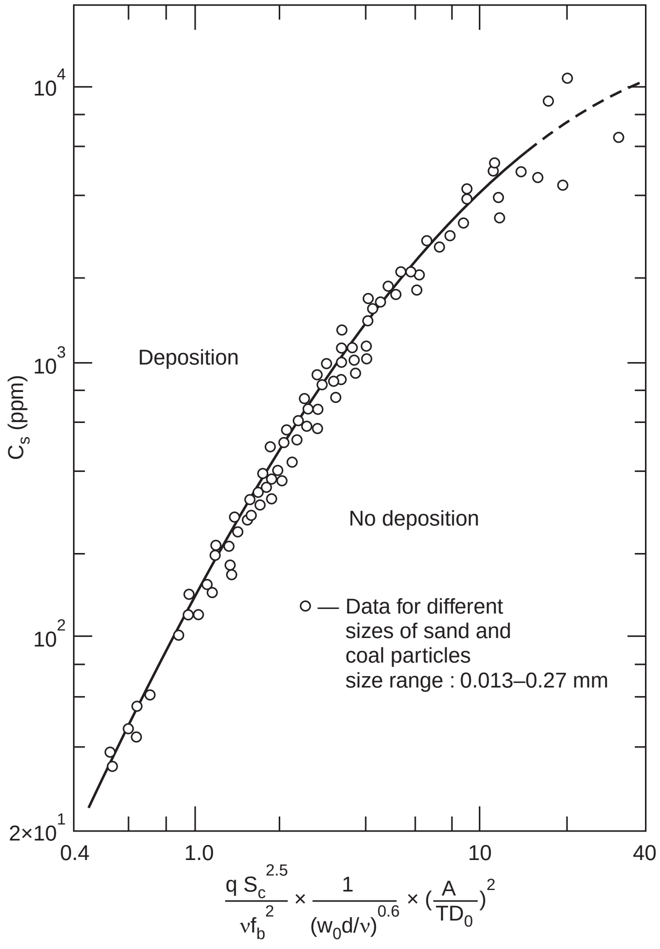

Fig. 8.3 Relation for the limiting concentration of suspended sediment in channels (4)

8.4. ALLUVIAL CHANNELS CARRYING CLEAR WATER

Compared to the design methods used for rigid boundary channels, the design of stable alluvial channels is more complex. Alluvial channels carrying clear water should be designed such that the erodible material of the channel boundary is not scoured. The design of such channels is, therefore, based on the concept of tractive force.

Scour on a channel bed occurs when the tractive force on the bed exerted by the flow is adequate to cause the movement of the bed particles. A sediment particle resting on the sloping side of a channel will move due to the resultant of the tractive force in the flow direction and the component of gravitational force which makes the particle roll or slide down the side slope. If the tractive force acting on the bed or the resultant of the tractive force and the component of the gravitational force both acting on the side slopes is larger than the force resisting the movement of the particle, erosion starts. The following method of design of stable alluvial channels based on this principle was proposed by Lane (1) and is also known as the USBR method.

In uniform flow, the average tractive stress, τ_0 is given as

$$\tau_o = \rho g R S \tag{8.5}$$

The shear stress is not uniformly distributed over the channel perimeter. Figure 8.4 shows the variation of the maximum shear stresses acting on the side, τ_{sm} and the bed, τ_{bm} of a channel. It should be noted that for the trapezoidal channels, the maximum tractive shear on the sides is approximately 0.76 times the tractive shear on the channel bed.

For a particle resisting on a level or mildly sloping bed, on can write the following expression for the incipient motion condition :

$$\tau_{bl}\, a = W_s \tan\theta \tag{8.6}$$

where, W_s is the submerged weight of the particle and a, the effective area of the particle over which the tractive stress τ_{bl} is acting, and θ is the angle of repose for the particle. τ_{bl} is, obviously, the critical shear stress τ_c for the bed particles. Since the aim is to avoid the movement of the movement of the particles (5), τ_{bl} may be kept less than τ_c , say 0.9 τ_c.

For a particle resting on the sloping side of a channel (Fig. 8.5), the condition for the incipient motion is

$$(W_s \sin\alpha)^2 + (\tau_{sl}\, a)^2 = (W_s \cos\alpha \tan\theta)^2$$

$$\therefore \quad W_s^2 \sin^2\alpha + (\tau_{sl}\, a)^2 = W_s^2 \cos^2\alpha \tan^2\theta$$

or

$$\tau_{sl} = \frac{W_s}{a} \cos\alpha \tan\theta \sqrt{1 - \frac{\tan^2\alpha}{\tan^2\theta}} \tag{8.7}$$

On combining Eqs. (8.6) and (8.7)

$$k = \frac{\tau_{sl}}{\tau_{bl}} = \cos\alpha \sqrt{1 - \frac{\tan^2\alpha}{\tan^2\theta}} \tag{8.8}$$

i.e.,

$$\tau_{sl} = k\, \tau_{bl} = 0.9k\, \tau_c$$

For non-scouring condition, the design rules become

$$\tau_{bm} \le \tau_{bl}$$

$$\tau_{sm} \le \tau_{sl}$$

Lane also observed that the channels, which are curved in alignment, scour more readily. He, therefore, suggested some correction factors which should be multiplied with the critical value of tractive stress (1). The values of critical shear stress (and also τ_{bl}) for bed and sides of curved channels are given in Table 8.3.

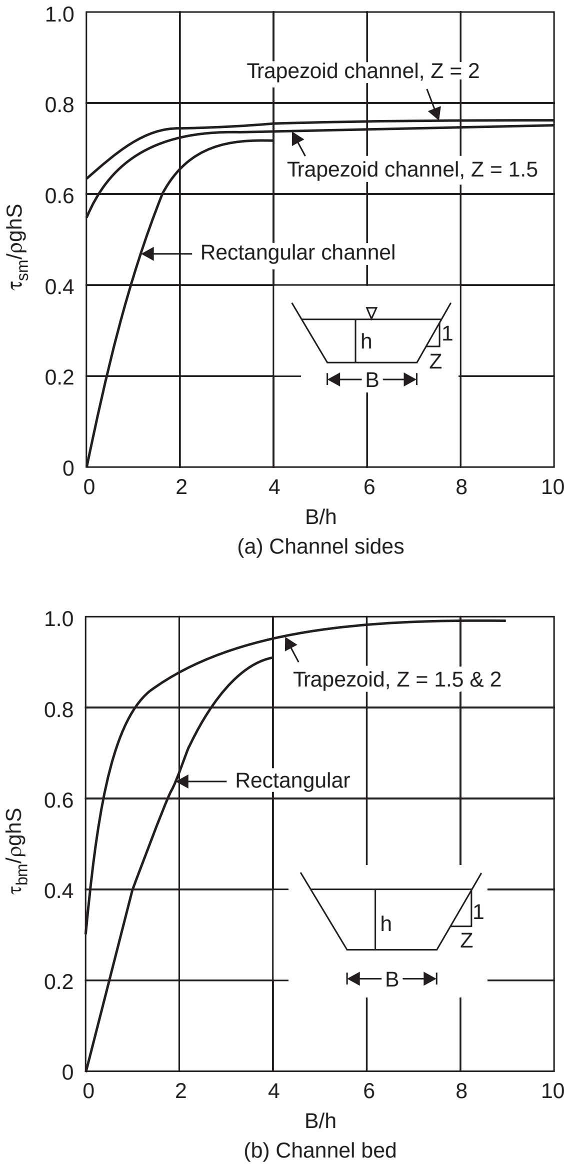

Fig. 8.4 Maximum shear stress on (*a*) sides and (*b*) bed of smooth channels in uniform flow

Table 8.3 Critical shear stress and the values of τ_{bl} for curved channels

Type of channel	*Critical shear stress* (τ_c)	τ_{bl}
Straight channels	τ_c	0.900 τ_c
Slightly curved channels	0.90 τ_c	0.810 τ_c
Moderately curved channels	0.75 τ_c	0.675 τ_c
Very curved channels	0.60 τ_c	0.540 τ_c

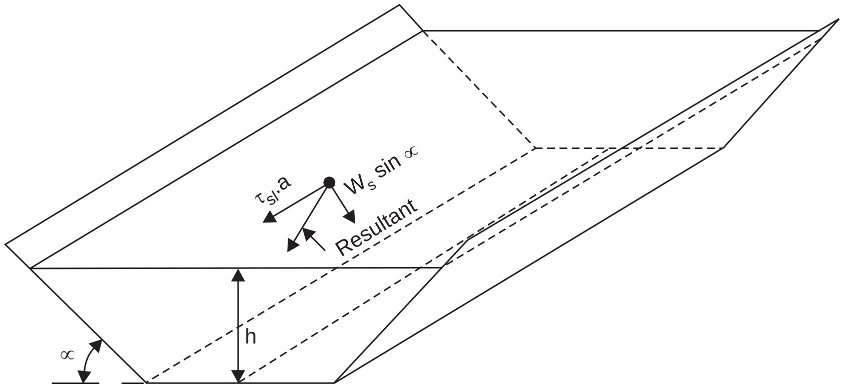

Fig. 8.5 Forces causing movement of a particle resting on a channel bank

Example 8.4 Design a trapezoidal channel (side slopes $2H : 1V$) to carry 25 m^3/s of clear water with a slope equal to 10^{-4}. The channel bed and banks comprise gravel (angle of repose = 31°) of size 3.0 mm. The kinematic viscosity of water may be taken as 10^{-6} m^2/s.

Solution: From Eq. (7.2),

$$R_0{}^* = \sqrt{\frac{\Delta\rho_s g d^3}{\rho\nu^2}} = \sqrt{\frac{(1.65)\,(9.81)\,(3.0\times10^{-3})^3}{(10^{-6})^2}}$$

$\therefore$ $\qquad R_0{}^* = 661.1$

From Fig. 7.3, $\qquad \tau_c{}^* = 0.045$

$\therefore$ $\qquad \tau_c = (0.045)\ \Delta\rho_s gd = 0.045 \times 1650 \times 9.81 \times (3.0 \times 10^{-3})$

$\qquad = 2.185$ N/m^2

Taking $\qquad \tau_{bl} = 0.9\ \tau_c$

$\qquad = 1.97$ N/m^2

$\therefore$ $\qquad \tau_{bm} = 1.97$ N/m^2

Now, $\qquad k = \cos\alpha\sqrt{1-\dfrac{\tan^2\alpha}{\tan^2\theta}} = \dfrac{2}{\sqrt{5}}\sqrt{1-\dfrac{(0.5)^2}{(0.6)^2}} = 0.494$

$\therefore$ $\qquad \tau_{sl} = 0.494 \times \tau_{bl} = 0.494 \times 1.97 = 0.973$ N/m$^2 = \tau_{sm}$

Rest of the computations are by trial and can be carried out as follows :

Assume $\qquad B/h = 10$

Therefore, from Fig. 8.4,

$$\frac{\tau_{sm}}{\rho g h S} = 0.78$$

$$h = \frac{0.973}{9810 \times 10^{-4} \times 0.78} = 1.27 \text{ m}$$

Also from Fig. 8.4,

$$\frac{\tau_{bm}}{\rho g h S} = 0.99$$

$$\therefore \qquad h = \frac{1.97}{9810 \times 10^{-4} \times 0.99} = 2.03 \text{ m}$$

Choosing the lesser of the two values of h

$$h = 1.27 \text{ m}$$

$$\therefore \qquad B = 10h = 12.7 \text{ m}$$

$$A = Bh + 2h^2 = 12.7 \times 1.27 + 2 \times (1.27)^2 = 19.355 \text{ m}^2$$

$$P = B + 2\sqrt{5}\,h = 12.7 + (2\sqrt{5} \times 1.27) = 18.38 \text{ m}$$

$$R = 1.053 \text{ m}$$

and
$$n = \frac{d^{1/6}}{25.6} = \frac{(3 \times 10^{-3})^{1/6}}{25.6} = 0.0148$$

$$\therefore \qquad Q = \frac{1}{n} AR^{2/3} S^{1/2} = \frac{1}{0.0148} \times 19.355 \times (1.053)^{2/3} (10^{-4})^{1/2}$$
$$= 13.5 \text{ m}^3/\text{s}$$

Since this value of Q is less than the given value, another value of B/h, say, 20.0 is assumed. Using Fig. 8.4, it will be seen that h = 1.27 m.

$$\therefore \qquad B = 25.4 \text{ m}$$

$\therefore$ A = 35.484 m^2, P = 31.08 m, R = 1.142 m and Q = 26.195 m^3/s

This value of Q is only slightly greater than the desired value 25.00 m^3/s.

Hence, B = 25.4 m and h = 1.27 m. The trial calculations can be done in a tabular form as shown below:

B/h	$\frac{\tau_{sm}}{\rho g h S}$	$\frac{\tau_{bm}}{\rho g h S}$	$h(m)$	$B(m)$	$A(m^2)$	$P(m)$	$R(m)$	$Q(m^3/s)$
10.0	0.78	0.99	1.27	12.7	19.355	18.38	1.053	13.500
20.0	0.78	0.99	1.27	25.4	35.484	32.08	1.142	26.195

8.5. ALLUVIAL CHANNELS CARRYING SEDIMENT-LADEN WATER

The cross-section of a stable alluvial channel would depend on the flow rate, sediment transport rate, and the sediment size. There are two methods commonly used for the design of alluvial channels carrying sediment-laden water. The first is based on the 'regime' approach in which a set of empirical equations is used. These equations have been obtained by analysing the data of stable field channels. A more logical method of design of stable alluvial channel should include the sediment load also.

8.5.1. Regime Methods

Regime Methods for the design of stable channels were first developed by the British engineers working for canal irrigation in India in the nineteenth century. At that time, the problem of sediment deposition was one of the major problems of channel design in India. In order to find a solution to this problem, some of the British engineers studied the behaviour of such stretches of the existing canals where the bed was in a state of stable equilibrium. The stable reaches had not required any sediment clearance for several years of the canal operation. Such channels were called regime channels. These channels generally carried a sediment load smaller than 500 ppm. Suitable relationships for the velocity of flow in regime channels were evolved. These relationships are now known as regime equations which find acceptance in other parts of the world as well. The regime relations do not account for the sediment load and, hence, should be considered valid when the sediment load is not large.

8.5.1.1. Kennedy's Method

Kennedy (6) collected data from 22 channels of Upper Bari Doab canal system in Punjab. His observations on this canal system led him to conclude that the sediment in a channel is kept in suspension solely by the vertical component of the eddies which are generated on the channel bed. In his opinion, the eddies generating on the sides of the channel had horizontal movement for greater part and, therefore, did not have sediment supporting power. This means that the sediment supporting power of a channel is proportional to its width (and not wetted perimeter).

On plotting the observed data, Kennedy obtained the following relation, known as Kennedy's equation.

$$U_o = 0.55h^{0.64} \tag{8.9}$$

Kennedy termed U_o as the critical velocity[1] (in m/s) defined as the mean velocity which will not allow scouring or silting in a channel having depth of flow equal to h (in metres). This equation is, obviously, applicable to such channels which have the same type of sediment as was present in the Upper Bari Doab canal system. On recognising the effect of the sediment size on the critical velocity, Kennedy modified Eq. (8.9) to

$$U = 0.55mh^{0.64} \tag{8.10}$$

in which m is the critical velocity ratio and is equal to U/U_o. Here, the velocity U is the critical velocity for the relevant size of sediment while U_o is the critical velocity for the Upper Bari Doab sediment. This means that the value of m is unity for sediment of the size of Upper Bari Doab sediment. For sediment coarser than Upper Bari Doab sediment, m is greater than 1 while for sediment finer than Upper Bari Doab sediment, m is less than 1. Kennedy did not try to establish any other relationship for the slope of regime channels in terms of either the critical velocity or the depth of flow. He suggested the use of the Kutter's equation along with the Manning's roughness coefficient. The final results do not differ much if one uses the Manning's equation instead of the Kutter's equation. Thus, the equations

$$U = 0.55mh^{0.64} \tag{8.10}$$

$$Q = AU \tag{8.11}$$

and

$$U = \frac{1}{n}R^{2/3}S^{1/2} \tag{8.12}$$

enable one to determine the unknowns, B, h, and U for given Q, n, and m if the longitudinal slope S is specified.

The longitudinal slope S is decided mainly on the basis of the ground considerations. Such considerations limit the range of slope. However, within this range of slope, one can obtain different combinations of B and h satisfying Eqs. (8.10) to (8.12). The resulting channel section can vary from very narrow to very wide. While all these channel sections would be able to carry the given discharge, not all of them would behave satisfactorily. Table 8.4 gives values of recommended width-depth ratio, *i.e.*, B/h for stable channel (5).

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[1]**This critical velocity should be distinguished from the critical velocity of flow in open channels corresponding to Froude number equal to unity.**

Table 8.4 Recommended values of *B/h* for stable channels

Q (m^3/s)	5.0	10.0	15.0	50.0	100.0	200.0	300.0
B/h	4.5	5.0	6.0	9.0	12.0	15.0	18.0

Several investigations carried out on similar lines indicated that the constant C' and the exponent x in the Kennedy's equation, $U = C' mh^x$ are different for different canal systems. Table 8.5 gives the value of C' and x in the Kennedy's equation for some regions.

Table 8.5 Values of C' and x in the Kennedy's equation for different regions (7)

Region	C'	x
Egypt	0.25 to 0.31	0.64 to 0.73
Thailand	0.34	0.66
Rio Negro (Argentina)	0.66	0.44
Krishna River (India)	0.61	0.52
Chenab River (India)	0.62	0.57
Pennar River (India)	0.60	0.64
Shwebo (Burma)	0.60	0.57
Imperial Valley (USA)	0.64 to 1.20	0.61 to 0.64

The design procedure based on Kennedy's theory involves trial. For known Q, n, m, and S, assume a trial value of h and obtain the critical velocity U from the Kennedy's equation [Eq. (8.10)]. From the continuity equation [Eq. (8.11)] one can calculate the area of cross-section A and, thus, know the value of B for the assumed value of h. Using these values of B and h, compute the mean velocity from the Manning's equation [Eq. (8.12)]. If this value of the mean velocity matches with the value of the critical velocity obtained earlier, the assumed value of h and the computed value of B provide channel dimensions. If the two velocities do not match, assume another value of h and repeat the calculations.

Ranga Raju and Misri (8) suggested a simplified procedure which does not involve trial. The method is based on the final side slope of $1H : 2V$ attained by an alluvial channel. During construction, the side slopes of a channel are kept flatter than the angle of repose of the soil. But, after some time of canal running, the side slopes become steeper due to the deposition of sediment. The final shape of the channel cross-section is approximately trapezoidal with side slopes $1H : 2V$. For this final cross-section of channel, one can write,

$$A = Bh + 0.5h^2 = h^2(p + 0.5) \tag{8.13}$$

$$P = B + 2.236h = h(p + 2.236) \tag{8.14}$$

where, $p = B/h$

Now
$$R = \frac{h(p+0.5)}{p+2.236} \tag{8.15}$$

and
$$U = \frac{Q}{h^2(p+0.5)} \tag{8.16}$$

Substituting the value of U and R in the Manning's equation [Eq. (8.10)], one obtains

$$S = \frac{Q^2 n^2 (p + 2.236)^{4/3}}{h^{16/3} (p + 0.5)^{10/3}} \tag{8.17}$$

Similarly, substituting the value of U in the Kennedy's equation [Eq. (8.10)], one gets,

$$h = \left[\frac{1.818Q}{(p + 0.5)m}\right]^{0.378} \tag{8.18}$$

On substituting the value of h from Eq. (8.18) in Eq. (8.17), one finally obtains

$$\frac{SQ^{0.02}}{n^2 m^2} = 0.299 \frac{(B/h + 2.236)^{1.333}}{(B/h + 0.5)^{1.313}} \tag{8.19}$$

Figure 8.6 shows the graphical form of Eq. (8.19).

For given Q, n, m, and a suitably selected value of S, compute $SQ^{0.02}/n^2 m^2$ and read the value of B/h from the Fig. 8.6. From Table 8.4, check if this value of B/h is satisfactory for the given discharge. If the value of B/h needs modification, choose another slope. Having obtained B/h, calculate h from Eq. (8.18) and then calculate B.

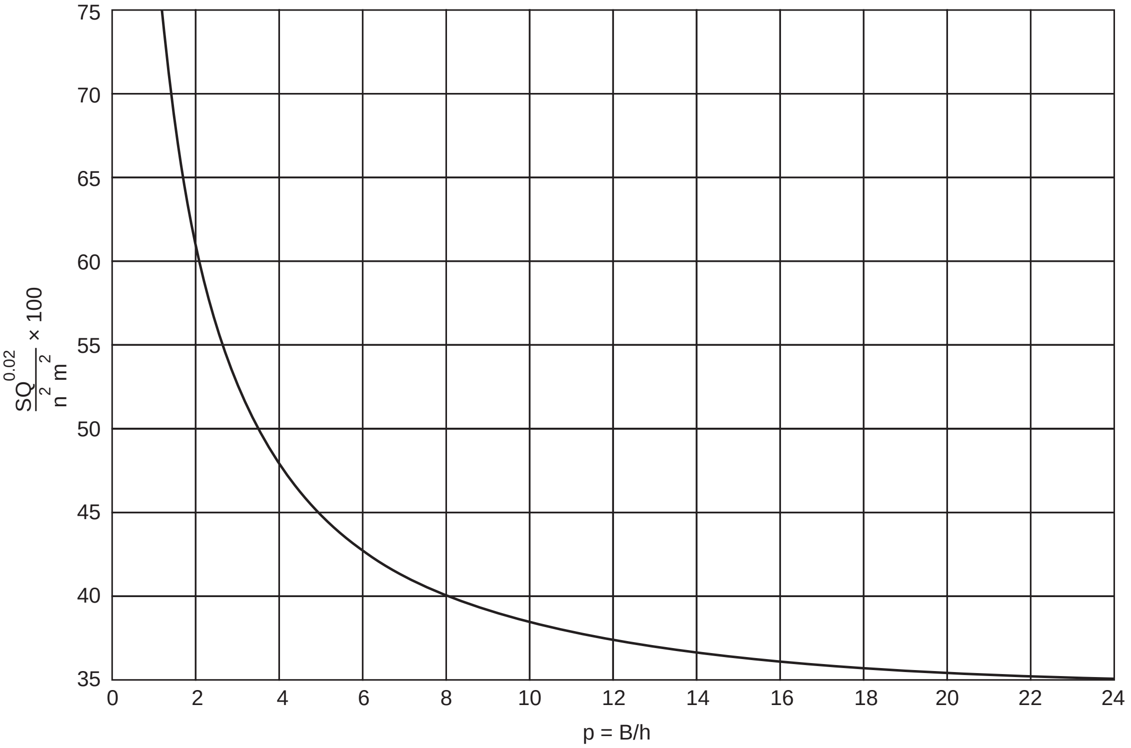

Fig. 8.6 Diagram for design of alluvial channels using the Kennedy's equation (8)

Example 8.5 Design a channel carrying a discharge of 30 m^3/s with critical velocity ratio and Manning's n equal to 1.0 and 0.0225, respectively. Assume that the bed slope is equal to 1 in 5000.

Solution:

Kennedy's method:

Assume h = 2.0 m. From Kennedy's equation [Eq. (8.10)]

$$U = 0.55\, mh^{0.64} = 0.55 \times 1 \times (2.0)^{0.64}$$

$$= 0.857 \text{ m/s}$$

$$\therefore \qquad A = Q/U = \frac{30}{0.857} = 35.01 \text{ m}^2$$

For a trapezoidal channel with side slope 1 H : 2V,

$$Bh + \frac{h^2}{2} = B(2.0) + \frac{2 \times 2}{2} = 2B + 2 = 35.01$$

$$\therefore \qquad B = 16.51 \text{ m}$$

$$\therefore \qquad R = \frac{35.01}{16.51 + 2.0\sqrt{5}} = 16.7 \text{ m}$$

Therefore, from the Manning's equation [Eq. (8.12)],

$$U = \frac{1}{n} R^{2/3} S^{1/2} = \frac{1}{0.0225} (1.67)^{2/3} \left(\frac{1}{5000}\right)^{1/2} = 0.885 \text{ m/s}$$

Since the velocities obtained from the Kennedy's equation and Manning's equation are appreciably different, assume h = 2.25 m and repeat the above steps.

$$U = 0.55 \times 1 \times (2.25)^{0.64} = 0.924 \text{ m/s}$$

$$A = \frac{30}{0.924} = 32.47 \text{ m}^2$$

$$\therefore \quad B(2.25) + (0.5)(2.25)^2 = 32.47$$

$$\therefore \qquad B = 13.31 \text{ m}$$

$$R = \frac{32.47}{13.31 \times (\sqrt{5} \times 2.25)} = 1.77 \text{ m}$$

$$U = \frac{1}{0.0225} (1.77)^{2/3} \left(\frac{1}{5000}\right)^{1/2}$$

$$= 0.92 \text{ m/s}$$

Since the two values of the velocities are matching, the depth of flow can be taken as equal to 2.25 m and the width of trapezoidal channel = 13.31 m.

Ranga Raju and Misri's method :

$$\frac{SQ^{0.02}}{n^2 m^2} = \frac{\frac{1}{5000}(30)^{0.02}}{(0.0225)^2 \times (1)^2} = 0.423$$

Hence, from Fig. 8.6,

$$p = \frac{B}{h} = 6.0$$

and

$$h = \left[\frac{1.818\,Q}{(p + 0.5)m}\right]^{0.378} = \left(\frac{1.818 \times 30}{(6.0 + 0.5) \times 1}\right)^{0.378} = 2.235 \text{ m}$$

$\therefore \qquad B = 6.0\ h = 13.41$ m

and
$$U = \frac{Q}{h^2 (p + 0.5)} = \frac{30}{(2.235)^2\ (6.0 + 0.5)}$$
$$= 0.924 \text{ m/s.}$$

8.5.1.2. Lindley's Method

Lindley (9) was the first to recognise that width, depth, and the slope of a channel can all adjust in an alluvial channel for a given set of conditions. He stated that when an artificial channel is used to carry sediment-laden water, both the bed and banks either scour or silt and thus change depth, gradient, and width until a state of balance is attained at which condition the channel is said to be in regime.

The observed width, slope, and depth of the Lower Chenab canal system were analysed by Lindley (9) using $n = 0.025$ and side slopes as $0.5H : 1V$. He obtained the following equations :

$$U = 0.57\ h^{0.57} \qquad (8.20)$$

$$U = 0.27\ B^{0.355} \qquad (8.21)$$

From Eqs. (8.20) and (8.21), one can get

$$B = 7.8\ h^{1.61} \qquad (8.22)$$

It should be noted that these equations do not include the effect of sediment size on the multiplying coefficient. Woods (10) also proposed equations similar to Eqs. (8.20) and (8.21).

8.5.1.3. Lacey's Method

Lacey (11) stated that the dimensions width, depth, and slope of a regime channel to carry a given water discharge loaded with a given sediment discharge are all fixed by nature. According to him, the fundamental requirements for a channel to be in regime are as follows :

(*i*) The channel flows uniformly in incoherent alluvium. Incoherent alluvium is the loose granular material which can scour or deposit with the same ease. The material may range from very fine sand to gravel, pebbles, and boulders of small size.

(*ii*) The characteristics and the discharge of the sediment are constant.

(*iii*) The water discharge in the channel is constant.

The perfect 'regime' conditions rarely exist. The channels which have lateral restraint (because of rigid banks) or imposed slope are not considered as regime channels. For example, an artificial channel, excavated with width and longitudinal slope smaller than those required, will tend to widen its width and steepen its slope if the banks and bed are of incoherent alluvium and non-rigid. In case of rigid banks, the width is not widened but the slope becomes steeper. Lacey termed this regime as the *initial regime*. A channel in initial regime is narrower than it would have been if the banks were not rigid. This channel has attained working stability. If the continued flow of water overcomes the resistance to bank erosion so that the channel now has freedom to adjust its perimeter, slope, and depth in accordance with the discharge, the channel is likely to attain what Lacey termed the *final regime*.

The river bed material may not be active at low stages of the river particularly if the bed is composed of coarse sand and boulders. However, at higher stages, the bed material becomes active, *i.e.*, it starts moving. As such, it is only during the high stages that the river may achieve regime conditions. This fact is utilised in solving problems related to floods in river channels.

Lacey also suggested that for a regime channel the roughness coefficient as well as the critical velocity ratio should be dependent on sediment size alone. However, it is now well known that in a movable bed channel, the total roughness includes grain as well as form roughness. Likewise, the non-silting and non-scouring velocity (included in the critical velocity ratio) shall depend on the sediment load and the size of the sediment.

Lacey felt that the sediment in an alluvial channel is kept in suspension by the vertical components of eddies generated at all points along the wetted perimeter. He, therefore, plotted the available data of regime channels to obtain a relationship between the regime velocity U (in m/s) and the hydraulic radius R(in metres). He, thus, found that $U \propto R^{1/2}$ and that the exponential power did not change with data. He, therefore, formulated

$$U = C'\sqrt{R} \tag{8.23}$$

in which C' is a proportionality constant.

Including a factor f_1 to account for the size and density of sediment, Lacey finally obtained

$$U = \sqrt{\frac{2}{5}f_1 R} = 0.632\sqrt{f_1 R} \tag{8.24}$$

The factor f_1 has been named as the *silt factor*. For natural sediment of relative density equal to 2.65, the silt factor f_1 can be obtained by Lacey's relation

$$f_1 = 1.76\sqrt{d} \tag{8.25}$$

where, d is the median size of sediment in millimetre.

On plotting Lindley's data and other data of regime channels, Lacey obtained

$$R^{1/2}\, S = C''$$

and $$C' = 10.8C''^{1/3}$$

$\therefore$ $$\frac{U}{R^{1/2}} = 10.8\,(R^{1/2}\,S)^{1/3}$$

which gives $$U = 10.8R^{2/3}\,S^{1/3} \tag{8.26}$$

Equation (8.26) is known as Lacey's regime equation and is of considerable use in evaluating flood discharges.

On the basis of the data of regime channels, Lacey also obtained

$$Af_1^{\,2} = 140U^5 \tag{8.27}$$

i.e., $$Qf_1^{\,2} = 140U^6 \tag{8.28}$$

on substituting the value of f_1 from Eq. (8.24),

$$Q\left(\frac{5U^2}{2R}\right)^2 = 140U^6$$

or $$\frac{25}{4}\times\frac{Q}{R^2} = 140U^2$$

or $$\frac{25}{4}Q\,\frac{P^2}{A^2} = 140U^2$$

or $$P^2 = \frac{560}{25}\,Q$$

i.e., $$P = 4.733\sqrt{Q} \cong 4.75\sqrt{Q} \tag{8.29}$$

Equation (8.29) with multiplying constant modified to 4.75 has been verified by a large amount of data and is very useful for fixing clear waterways for structures, such as bridges on rivers. Again, substituting the value of f_1 in Eq. (8.27),

$$A\left(\frac{5U^2}{2R}\right)^2 = 140U^5$$

$$\therefore \qquad \frac{A}{R^2} = \frac{560}{25}U$$

or
$$P = 22.4\ RU$$

or
$$4.75\sqrt{Q} = 22.4RU$$

or
$$RU = 0.212\ Q^{1/2} \tag{8.30}$$

For wide channels, RU equals the discharge per unit width. Hence,

$$q = 0.212\ Q^{1/2} \tag{8.31}$$

Equation (8.31) relates the discharge per unit width of a regime channel with the total discharge flowing in the channel.

On substituting the value of U from Eq. (8.24) in Eq. (8.30), one gets

$$R\sqrt{\frac{2}{5}f_1R} = 0.212\ Q^{1/2}$$

$$\therefore \qquad R = 0.48\left(\frac{Q}{f_1}\right)^{1/3} \tag{8.32}$$

For wide channels, the hydraulic radius is almost equal to the depth of flow. Equation (8.32), therefore, gives the depth of scour below high flood level. Hence, Eq. (8.32) can be utilised to estimate the depth of flow in a river during flood. This information forms the basis for the determination of the levels of foundations, vertical cutoffs, and lengths of launching aprons of a structure constructed along or across a river.

On combining Eqs. (8.31) and (8.32),

$$R = 1.35\left(\frac{q^2}{f_1}\right)^{1/3} \tag{8.33}$$

Further, eliminating U from Eqs. (8.24) and (8.26), one can obtain a relationship for the slope of a regime channel. Thus,

$$\sqrt{\frac{2}{5}f_1R} = 10.8R^{2/3}\ S^{1/3}$$

$$\therefore \qquad S = 0.0002\ \frac{f_1^{3/2}}{R^{1/2}} \tag{8.34}$$

On substituting the value of R from either Eq. (8.32) or Eq. (8.33), in Eq. (8.34), one obtains,

$$S = 0.0003\ \frac{f_1^{5/3}}{Q^{1/6}} \tag{8.35}$$

and
$$S = 0.000178\frac{f_1^{5/3}}{q^{1/3}} \tag{8.36}$$

Lacey's regime relations, Eqns. (8.23) to (8.36), are valid for regime channels and can be used suitably to design a regime channel for a given discharge and sediment size.

The following flow equation was also obtained by Lacey (11) :

$$U = \frac{1}{N_a} R^{3/4} S^{1/2} \tag{8.37}$$

where,

$$N_a = 0.0225 f_1^{\,1/4} \tag{8.38}$$

This means that the absolute roughness coefficient N_a is dependent only on the sediment size. On examining the data from various channels, the value of N_a was, however, not found to be constant. Lacey introduced the concept of 'shock' to explain the variation in N_a. He contended that a non-regime channel requires a larger slope (*i.e.*, large value of N_a) to overcome what he termed as ' shock resistance' or the resistance due to bed irregularities. The shock resistance can, therefore, be considered similar to the form resistance of the bed undulations. This concept of Lacey leads one to conclude that a regime channel is free from shock. It is, however, known that the geometry of bed undulations can change even for the same sediment size and, hence, Lacey's contention that a regime channel is free from shock, is unacceptable (12).

Lacey's equations, commonly used for the design of alluvial channels, are summarised below :

$$f_1 = 1.76\sqrt{d} \tag{8.25}$$

$$U = 10.8\, R^{2/3}\, S^{1/3} \tag{8.26}$$

$$P = 4.75\sqrt{Q} \tag{8.29}$$

$$R = 0.48\left(\frac{Q}{f_1}\right)^{1/3} \tag{8.32}$$

$$S = 0.0003\frac{f_1^{\,5/3}}{Q^{1/6}} \tag{8.35}$$

8.5.1.4. Comments on Regime Equations

The regime equations have been empirically derived using data of regime channels. These channels carried relatively less sediment load (approximately 500ppm). Thus, the equations can be expected to yield meaningful results only for the conditions in which the sediment load is of the same order as was being carried by the channels whose data have been used in obtaining the regime relations. The dimensions of a stable channel will be affected by the water discharge, the sediment characteristics and the amount of sediment load in the channel. The regime relations take into account only the water discharge and the sediment size but do not take into account the amount of sediment material being transported by the channels. Nevertheless, the regime equations do provide useful information which is very helpful in the design of unlined channels and other structures on alluvial rivers.

Example 8.6 Design a stable channel for carrying a discharge of 30 m^3/s using Lacey's method assuming silt factor equal to 1.0.

Solution: From Eq. (8.29),

$$P = 4.75\sqrt{Q} = 4.75\sqrt{30.0} = 26.02 \text{ m}$$

From Eq. (8.32),

$$R = 0.48(Q/f_1)^{1/3} = 0.48\left(\frac{30.0}{1.0}\right)^{1/3} = 1.49 \text{ m}$$

From Eq. (8.35),

$$S = 3 \times 10^{-4} f_1^{5/3}/Q^{1/6} = 3 \times 10^{-4} (1.0)^{5/3}/(30)^{1/6}$$
$$= 1.702 \times 10^{-4}$$

Form Eq. (8.26),

$$U = 10.8 R^{2/3} S^{1/3} = 10.8(1.49)^{2/3} (1.702 \times 10^{-4})^{1/3}$$
$$= 0.781 \text{ m/s}$$

Assuming final side slope of the channel as $0.5H : 1V$ (generally observed field value),

$$P = B + (\sqrt{5})h = 26.02 \text{ m}$$

$$\therefore \quad B = 26.02 - 2.24h$$

and

$$A = Bh + \frac{h^2}{2} = PR = 26.02 \times 1.49 = 38.77 \text{ m}^2$$

$$\therefore \quad 26.02h - 2.24h^2 + 0.5h^2 = 38.77$$

or

$$1.74h^2 - 26.02h + 38.77 = 0$$

$$\therefore \quad h = \frac{26.02 \pm \sqrt{(26.02)^2 - 4 \times 1.74 \times 38.77}}{2 \times 1.74}$$
$$= \frac{26.02 \pm 20.18}{3.48}$$
$$= 13.28 \text{ m and } 1.68 \text{ m}$$

The value of h equal to 13.28 m gives negative B and is, therefore, not acceptable. Hence, $h = 1.68$ m, and

$$B = 26.02 - 2.24 \times 1.68$$
$$= 22.23 \text{ m}$$

Example 8.7 An irrigation channel is to be designed for a discharge of 50 m^3/s adopting the available ground slope of 1.5×10^{-4}. The river bed material has a median size of 2.00 mm. Design the channel and recommend the size of coarser material to be excluded or ejected from the channel for its efficient functioning.

Solution: From Eqs. (8.25) and (8.35),

$$f_1 = 1.76\sqrt{d}$$
$$= 1.76\sqrt{2}$$
$$= 2.49$$

and

$$S = 0.0003 f_1^{5/3}/Q^{1/6}$$
$$= 0.0003 (2.49)^{5/3}/(50)^{1/6}$$
$$= 7.15 \times 10^{-4}$$

The computed slope is much large than the available ground slope of 1.5×10^{-4} which is to be adopted as the channel bed slope. Therefore, the median size of sediment which the channel would be able to carry can be determined by computing the new value of f_1 for $S = 1.5 \times 10^{-4}$ and the given discharge and then obtaining the value of d for this value of f_1 using Eq. (8.25). Thus,

$$1.5 \times 10^{-4} = 0.0003 f_1^{5/3}/(50)^{1/6}$$

$\therefore \qquad f_1 = 0.976$

and $\qquad d = (f_1/1.76)^2 = 0.30$ mm

Therefore, the material coarser than 0.30 mm will have to be removed for the efficient functioning of the channel. The hydraulic radius of this channel R is obtained from Eq. (8.32) :

$$R = 0.48\left(\frac{50}{0.976}\right)^{1/3}$$

$\therefore \qquad R = 1.783$ m

Using Eq. (8.29)

$$P = 4.75\sqrt{50}$$
$$= 33.59 \text{ m}$$

$\therefore \qquad B + \sqrt{5}\,h = 33.59$

or $\qquad B = 33.59 - 2.24h$

and $\qquad A = Bh + \dfrac{h^2}{2} = PR = 33.59 \times 1.783 = 59.89 \text{ m}^2$

or $\quad 33.59h - 2.24h^2 + 0.5h^2 = 59.89$

or $\quad 1.74h^2 - 33.59h + 59.89 = 0$

$$\therefore \qquad h = \frac{33.59 \pm \sqrt{(33.59)^2 - 4 \times 1.74 \times 59.83}}{2 \times 1.74}$$
$$= \frac{33.59 \pm 26.67}{3.84}$$
$$= 17.32 \text{ m} \quad \text{or} \quad 1.99 \text{ m}$$

Obviously, $h = 1.99$ m as the other root of h would result in too narrow a channel section.

$\therefore \qquad B = 33.59 - 2.24(1.99)$

$$= 29.13 \text{ m}$$

8.5.2. Method of Design of Alluvial Channels Including Sediment Load as a Variable

Experiments have revealed (13) that the stable width of a regime channel is practically independent of sediment load. Hence, one can use Lacey's equation [Eq. (8.29)] for stable perimeter P even when the sediment load is varying. If the bank soil is cohesive, the perimeter may be kept smaller than that given by Eq. (8.29). However, the sediment load is known to affect the regime slope of a channel and, hence, the sediment load should also be included in the design of stable alluvial channels.

If the sediment load is moving mainly in the form of bed load, one can determine the unknowns R and S for given Q, q_B, d, and n using the Meyer-Peter's equation [Eq. (7.33)] and the Manning's equation [Eq.(8.12)]

If the suspended sediment load is also considerable, then one may use the total load equation instead of Meyer-Peter's equation. Combining Engelund and Hansen equation [Eq. (7.37)] for sediment load,

$$f\phi_T = 0.4\tau_*^{5/2} \qquad (8.39)$$

and their resistance equation (14),

$$U = 10.97\, d^{-3/4}\, h^{5/4}\, S^{9/8} \tag{8.40}$$

a design chart (Fig. 8.7) was prepared. This chart can be used to determine h and S, for known values of

$$\phi_T \left[= \frac{q_T/\rho_s g}{\sqrt{\dfrac{\Delta\rho_s}{\rho}\, gd^3}} \right] \text{ and } \frac{q}{\sqrt{\dfrac{\Delta\rho_s}{\rho}\, gd^3}}$$

For the width of stable channel, Engelund and Hansen (14) suggested the use of the empirical relation

$$B = \frac{0.786 Q^{0.525}}{d^{0.316}} \tag{8.41}$$

Equation (8.41) is valid for channels with sandy bed and banks (12). However, because of cohesive soils of India, the width of stable channels is smaller than that given by Eq. (8.41).

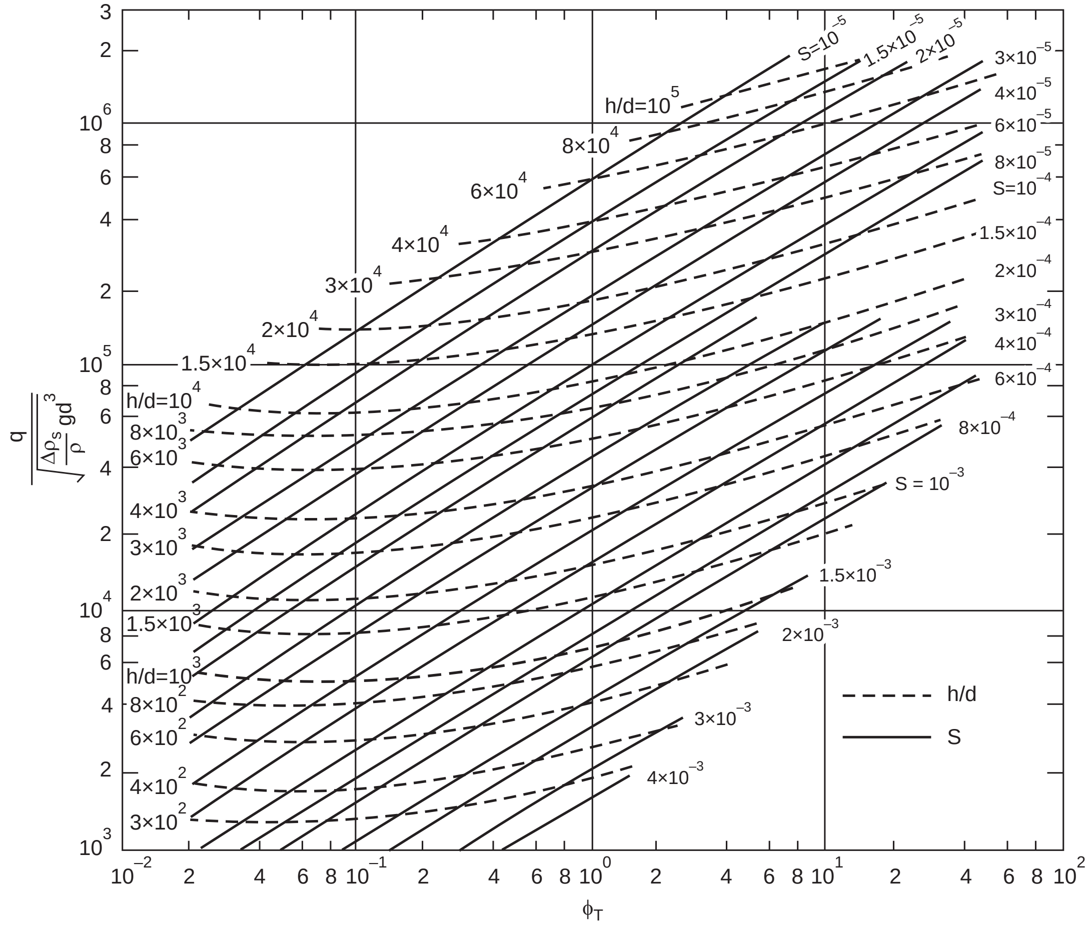

Fig. 8.7 Engelund and Hansen's chart for stable channel design

Therefore, one can adopt a value of B which is slightly less than or equal to P obtained from Lacey's equation [Eq. (8.29)].

Equations (8.39) and (8.40) can be solved for h and S to yield the following explicit relations :

$$h = 0.17 \frac{q^{20/21}}{(g\phi_T)^{2/7} d^{3/7} (\Delta\rho_s/\rho)^{5/7}} \tag{8.42}$$

$$S = 8.4 \frac{q^{8/9} d^{2/3}}{h^2} \tag{8.43}$$

or

$$S = 4.12 \frac{(g\phi_T)^{4/7} d^{32/21} (\Delta\rho_s/\rho)^{10/7}}{q^{64/63}} \tag{8.44}$$

Equations (8.41), (8.42), and (8.43) or (8.44) will yield direct solutions for B, h and S, and avoid interpolation errors involved in the use of the design chart (Fig. 8.7).

In Eqs. (8.40) to (8.44), U is expressed in m/s, Q in m^3/s, q in m^2/s, q_T in N/m/s, and h, d, and B in m. It should be noted that Eq. (8.42) is valid for duned-bed conditions.

Example 8.8 Design a channel to carry a discharge of 30 m^3/s with sediment load concentration of 50 ppm by weight. The average grain size of the bed material is 0.3 mm. Assume the cross-section of the channel as trapezoidal with side slopes $\frac{1}{2}$ $H : 1V$.

Solution: Using Eq. (8.29)

$$P = 4.75\sqrt{Q} = 4.75\sqrt{30} = 26.02 \text{ m}$$

Choose B to be slightly less than P, say B = 24.0 m.

$$Q_T = 50 \times 10^{-6} \times 9810 \times 30 = 14.71 \text{ N/s}$$

$$\therefore \quad q_T = \frac{14.71}{24.0} = 0.613 \text{ N/m/s}$$

and

$$q = \frac{30.0}{24.0} = 1.25 \text{ m}^2\text{/s}$$

Now

$$\phi_T = \frac{0.613/(2.65 \times 9810)}{\sqrt{1.65 \times 9.81 \times (0.3 \times 10^{-3})^3}} = 1.128$$

and

$$\frac{q}{\sqrt{\left(\frac{\Delta\rho_s}{\rho}\right) g d^3}} = \frac{1.25}{\sqrt{1.65 \times 9.81 \times (0.3 \times 10^{-3})^3}} = 59793.2$$

From Fig. 8.7,

$$S = 1.1 \times 10^{-4}$$

and

$$\frac{h}{d} = 7.2 \times 10^3$$

$$\therefore \quad h = 7.2 \times 10^3 \times (0.3 \times 10^{-3}) = 2.16 \text{ m}$$

Alternatively, using Eqs. (8.42) and (8.43),

$$h = \frac{0.17 \times (1.25)^{20/21}}{(9.81 \times 1.128)^{2/7}\,(0.3 \times 10^{-3})^{3/7}\,(1.65)^{5/7}}$$

$$= 2.38 \text{ m}$$

$$S = \frac{8.4 \times (1.25)^{8/9} \times (0.3 \times 10^{-3})^{2/3}}{(2.38)^2}$$

$$= 8.1 \times 10^{-3}$$

Example 8.9 Solve Example 8.8 considering the entire sediment load as bed load.

Solution: $Q_B = 50 \times 10^{-6} \times 9810 \times 30 = 14.71$ N/s

Silt factor, $f_1 = 1.76\sqrt{d} = 1.76\sqrt{0.3} = 0.964$

Since the value of Manning's n is 0.0225 for $f_1 = 1.0$, one can take $n = 0.022$ for $f_1 = 0.964$

Using Eq. (8.29),

$$P = 4.75\sqrt{30.0} = 26.02 \text{ m}$$

Let $B = 24.0$ m

$$\therefore \quad q_B = \frac{14.71}{24.0} = 0.613 \text{ N/m/s}$$

Hence,

$$\phi_B = \frac{q_B/(\rho_s g)}{\sqrt{\frac{\Delta\rho_s}{\rho} g d^3}} = \frac{0.613/(2650 \times 9.81)}{\sqrt{1.65 \times 9.81 \times (0.3 \times 10^{-3})^3}} = 1.128$$

and from Eq. (7.10)

$$n_s = \frac{d^{1/6}}{25.6}$$

$$= \frac{(0.3 \times 10^{-3})^{1/6}}{25.6}$$

$$= 0.01$$

$$\therefore \quad \tau'_* = \left(\frac{n_s}{n}\right)^{3/2} \frac{\rho RS}{\Delta\rho_s d} = \left(\frac{0.01}{0.022}\right)^{3/2} \frac{RS}{1.65 \times (0.3 \times 10^{-3})}$$

$$= 619.10\, RS$$

$\therefore$ Using Meyer-Peter's equation [Eq. (7.33)],

$$\phi_B = 8.0(\tau'_* - 0.047)^{3/2}$$

or

$$1.128 = 8.0(619.10RS - 0.047)^{3/2}$$

$$\therefore \quad RS = 5.135 \times 10^{-4}$$

Now using the Manning's equation,

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

$$30 = \frac{1}{0.022}\,(26.02 \times R^{5/3}\, S^{1/2})$$

$\therefore \qquad R^{5/3}\, S^{1/2} = 0.0254$

or $\qquad R^{7/6}(5.135 \times 10^{-4})^{1/2} = 0.0254$

$\therefore \qquad R = 1.103$ m

and $\qquad S = 4.66 \times 10^{-4}$

$$A = Bh + \frac{h^2}{2} = 24h + \frac{h^2}{2} = PR = 26.02 \times 1.103$$

or $\qquad h^2 + 48h - 57.4 = 0$

$$h = \frac{-48 \pm \sqrt{(48)^2 + 4 \times 57.4}}{2}$$

$$h = 1.167 \text{ m}$$

$$P = B + \sqrt{5}\,h$$

$$= 24 + \sqrt{5} \times 1.67$$

$$= 26.61 \text{ m}$$

which is close to Lacey's perimeter (= 26.02 m)

Hence,

$$B = 24.0 \text{ m} \quad h = 1.167 \text{ m} \quad \text{and} \quad S = 4.66 \times 10^{-4}$$

8.6. PROCEDURE FOR DESIGN OF IRRIGATION CHANNELS

A canal system includes the main canal, branches, distributaries, and watercourses. Main canals, branches and distributaries are owned, constructed, controlled, and maintained by the government. Watercourses are owned, constructed, controlled, and maintained by the irrigators. Sometimes, the watercourses are constructed by the government agencies also. Their maintenance is, however, the responsibility of irrigators.

The main canal takes its supply from the source directly. The source may be either a river or a reservoir. There may be several branch canals and/or distributaries taking supplies from the main canal at various points along the main canal. A branch channel, similarly, feeds distributaries which take off at suitable locations along the branch channel. A distributary feeds watercourses which take water directly to the field. The discharge and size of irrigation channels progressively decrease from the main canal to the watercourses.

8.6.1. Layout and Discharge Requirements of Irrigation Channels

The layout of irrigation channels should ensure equitable distribution of water with minimum expenditure. It is, obviously, advantageous to align all irrigation channels on the watershed. This will ensure least expense on cross-drainage structures and cause gravity flow on either side of the channel. In hilly regions, the channels are aligned along the contours. Alignment of irrigation channels should be such that there are very few curves. However, if the channel curves are unavoidable, the radius of curvature of these curves should be as large as possible. For large channels, the minimum radius of curvature should be about 60 times the bed width of the channel. In case of smaller channels, the minimum radius of curvature of the curves should be about 45 times the bed width of the channel.

The maximum discharge to be carried by an irrigation channel at its head to satisfy the irrigation requirements for its command area under the worst conditions during any part of a year is said to be the designed full supply discharge (or capacity) of the channel. The water level in the channel when it is carrying its full supply discharge is termed full supply level (FSL) and the corresponding depth of flow is called full supply depth (FSD).

After fixing the layout of all irrigation channels (*i.e.*, canal system) on a map, the total command area proposed to be irrigated by each of the channels of the canal system is estimated. Knowing the crop requirements and intensity of irrigation, and allowing for seepage and evaporation loses, the amount of the discharge requirement at various points of the canal system are determined. Obviously, this requirement will progressively increase from the tail end of a channel to its head reach. Accordingly, the size (*i.e.*, width and depth) of a channel will progressively decrease form its head reach to its tail end. The size of a watercourse is, however, kept uniform.

8.6.2. Longitudinal Section of Irrigation Channels

8.6.2.1. Longitudinal Slope

An irrigation channel flowing in an alluvium can be designed by any of the methods discussed in Secs. 8.4 and 8.5. The slope of the channel would thus be known. However, it is convenient to choose a slope close to Lacey's regime slope [Eq. (8.35)] for the first trial design. This may need modification subsequently. The slope of an irrigation channel is governed by the country slope too. If the ground slope is more than Lacey's regime slope (or the slope satisfying the critical velocity ratio and bed width-depth ratio, or that calculated on the basis of other methods), then the slope of the channel bed is kept as the required slope (*i.e.*, the calculated one) and extra fall in the ground level is compensated for by providing vertical falls in the channel. On the other hand, if the required bed slope is more than the available ground slope, the maximum allowed by the ground slope is provided. In such cases, it is advisable to prevent the entry of coarse sediment at the head of the channel (see Example 8.7). Usually, ground slope is steeper than the desired bed slope and falls are invariably provided along the channels. The slope of reservoir-fed channels, carrying water with practically no sediment, may be determined by the consideration of the maximum permissible velocity.

For convenience, the slope is kept in multiples of 2.5 cm/km. The advantage of such values is that the drop in the bed level for every 200 m would be 0.5 cm which can be easily read on a levelling staff.

8.6.2.2. Full Supply Level (or Full Supply Line)

Since the channel has been aligned on the watershed, the available transverse slopes would be sufficient for flow irrigation provided that the full supply level (FSL) is slightly above the ground level. The full supply level of a channel should, therefore, be about 10-30 cm higher than the ground level for most of its length so that most of the command area of the channel is irrigated by gravity flow. It is not advisable to keep the FSL higher than the isolated high patches of land existing in the command area. These may be irrigated either by lifting water from the watercourses or from the upstream outlets, if feasible. Too high an FSL would result in uneconomical channel cross-section of higher banks. Besides, the available higher head would increase the seepage loss, wasteful use of water by the farmers, and the chances of waterlogging in the area.

The FSL of the offtaking channel at its head should always be kept at least 15 cm lower than the water level of the parent (or supply) channel. This provision is made to account for the head loss in the regulator and for the possibility of the offtaking channel bed getting silted up in its head reaches.

8.6.2.3. Canal Falls

When the desired bed slope is flatter than the ground slope, as is usually the case, a canal fall has to be provided well before the channel bed comes into filling and the FSL becomes too much above the ground level. The location of a fall depends on a number of factors. It may be combined with a regulator or a bridge for economic benefits.

The alignment of another smaller offtaking channel may require that the fall be provided downstream of the head of the offtaking canal. This will ensure that the FSL of the offtaking channel is sufficiently high to irrigate its command area by flow irrigation. One may provide either a larger number of smaller falls or a smaller number of larger falls. Relative economy of these two possible alternatives should be worked out and the location of falls will be, accordingly, decided. The distributaries and watercourses offtaking from the upstream of a fall may easily irrigate some areas downstream of the fall. As such the FSL in the channel may be allowed to remain below ground level for about 200-400 m downstream of a fall.

8.6.2.4. Balanced Earthwork

The longitudinal section of a channel should be such that it results in balanced earthwork which means that the amount of excavated soil is fully utilised in fillings. This will minimise the necessity of borrowpits (from where the extra earth needed for filling will be taken) and spoil banks (where the extra earth from excavation will be deposited). The earthwork is paid for on the basis of either the excavation or the filling in embankments, whichever is more. A channel in balanced earthwork will, therefore, reduce the cost of the project. For the small length of a channel in the vicinity of a fall there is always an unbalanced earthwork which should be kept minimum for obvious economic reasons.

8.6.3. Evaporation and Seepage Losses in Canals

When water flows in an alluvial channel, some water is lost due to evaporation from the water surface and seepage through the bed and banks of the channel. These losses are termed transit losses or canal losses. The transit losses are calculated at the rate of about 3 m^3/s per million square metre of the exposed water surface area. These losses may be of the order 10 to 40 per cent of the discharge at the head of a channel in case of large channels.

8.6.4. Cross-Section of Irrigation Channels

An irrigation channel may be in cutting or in filling or in partly cutting and partly filling. When the ground level is higher than the full supply level of the channel, the channel is said to be in cutting [Fig. 8.8 (*a*)]. If the channel bed is at or higher than the surrounding ground level, the channel is in filling, [Fig. 8.8(*b*)]. When the ground level is in between the supply level and the bed level of the channel, the channel is partly in cutting and partly in filling [Fig. 8.8 (*c*)]. A channel section partly in cutting and partly in filling results in balanced earthwork and is preferred. Various components of cross-section of an irrigation channel have been discussed in the following paragraphs.

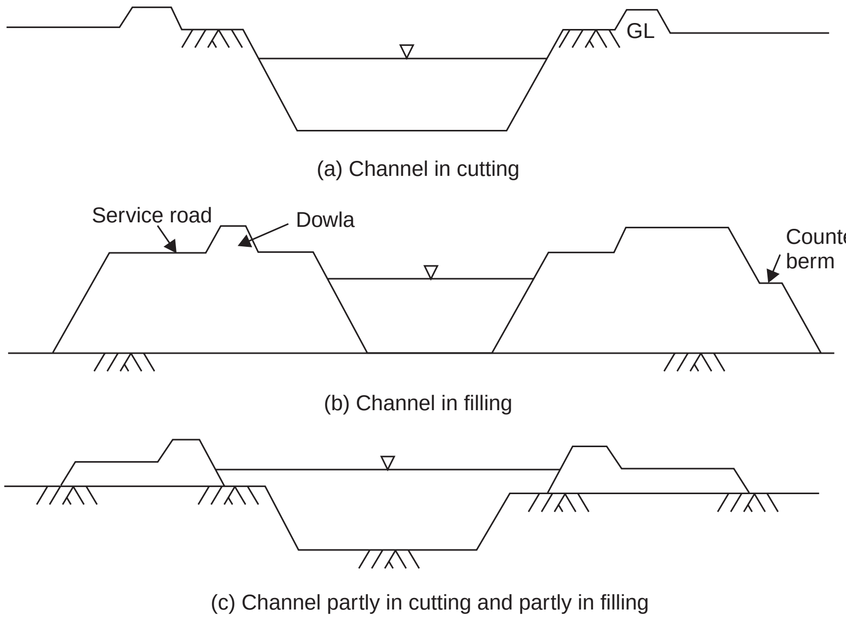

Fig. 8.8 Typical cross-sections of irrigation channels

8.6.4.1. Side Slopes

Side slopes in an unlined channel depend mainly on the nature of geological formations through which the channel is excavated. Side slopes in an unlined channel should be flatter than the angle of repose of saturated bank soil so that portions of side slopes will not slough into the channel. For similar conditions, the prevalent practice is to keep side slopes flatter for channels in filling than the side slopes for channels in cutting. However, there is no justification for this practice as a natural earthfill should not behave differently from an earthfill compacted sufficiently and having the same characteristics as those of the natural earthfill. Initially, flatter slopes are provided for reasons of stability. Later, with the deposition of fine sediments, the side slopes become steeper and attain a value of $(1/2)H : 1V$ irrespective of the initial side slope provided. These steeper side slopes are stable and the design is usually based on these slopes. Side slopes for unlined channels in different types of soil, as recommended by the Central Water Commission of India, are given in Table 8.6.

Table 8.6 Side slopes for unlined channel

Type of soil	*Side slopes (H : V)*
Loose sand to average sandy soil	1.5 : 1 to 2 : 1 (in cutting) 2 : 1 to 3 : 1 (in filling)
Sandy loam and black cotton soil	1 : 1 to 1.5 : 1 (in cutting) 2 : 1 (in filling)
Gravel	1 : 1 to 2 : 1
Murum or hard soil	0.75 : 1 to 1.5 : 1
Rock	0.25 : 1 to 0.5 : 1

8.6.4.2. Berms

A berm is a narrow horizontal strip of land between the inner toe of the bank and the top edge of cutting. Berms between water section and inner bank slopes are required along the channels where bank materials are susceptible to sloughing. Berms slope towards water section to facilitate drainage.

Because of irregular changes in the ground level and regular changes in the channel bed level, the depth of cutting d_1 and the depth of filling, $(d_2 - d_1)$ (Fig. 8.9) would vary. This will cause variation in the horizontal distance between the bed and top of the bank (*i.e.*, between X and Y). If $r_1\ (H) : 1(V)$ is the side slope in cutting and $r_2(H) : 1(V)$ is the inner side slope of embankment, then the berm width is kept equal to $(r_2 - r_1)\ d_1$. This will ensure that the horizontal distance between the bed and top of the bank (*i.e.*, between X and Y) will remain constant. As can be seen from Fig. 8.9, this distance is equal to $r_1 d_1 + (r_2 - r_1)\ d_1 + r_2(d_2 - d_1)$ which equals $r_2 d_2$. Since the total height from the bed to top of the bank, d_2, remains practically constant, the horizontal distance between X and Y (*i.e.*, $r_2 d_2$) also remains constant if the berm width is equal to $(r_2 - r_1)d_1$.

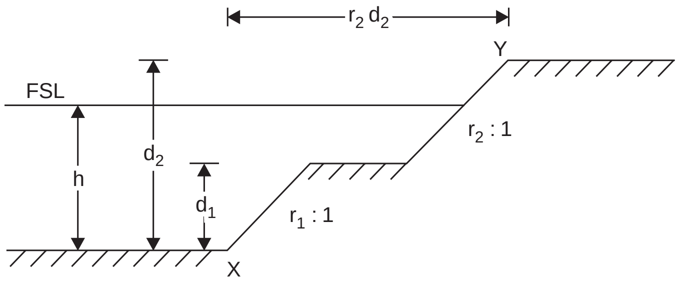

Fig. 8.9 Berm

For usual side slopes of 1.5 $(H) : 1\ (V)$ (in filling) and $1(H) : 1(V)$ (in cutting), a berm of width equal to half the depth of cutting will make the horizontal distance between the bed and top of the bank (*i.e.*, XY) equal to 1.5 times the height of the top of the bank with respect to the channel bed.

As a result of silting on berms, an impervious lining is formed on the banks. This helps in reduction of seepage losses. In addition, the berms protect the banks against breaches and the eroding action of waves. These berms break the flow of rain water down the bank slope and, thus, prevent guttering. An additional berm may also be provided in channel sections which are in deep cutting.

8.6.4.3. Freeboard

Freeboard is the vertical distance from the water surface at full supply level to the top of bank. Freeboard provides the margin of safety against overtopping of the banks due to sudden rise in the water surface of a channel on account of improper operation of gates at the head regulator, accidents in operation, wave action, land slides, and inflow during heavy rainfall. The excessive growth of vegetation or accumulation of sediment deposits may also result in the gradual rise of water surface levels above the design levels. Design of channels should specify adequate freeboards to prevent overtopping of the banks during sudden rises in water surface. Adequate freeboard would depend on dimensions of the flow section, flow condition, bank material, method of construction of banks, and resulting damage due to failure of banks.

Freeboard in unlined channels vary from about 0.3 m in small channels to about 2 m in large channels. For channels of intermediate size, freeboard is sometimes estimated by adding 0.3 m to one quarter of the flow depth.

The Central Water Commission of India (CWC) has recommended the value of freeboard as given in Table 8.7.

Table 8.7 Freeboard of irrigation channels as suggested by CWC

Discharge (m^3/s)	up to 0.7	0.7 to 1.4	1.4 to 8.5	over 8.5
Freeboard (m)	0.46	0.61	0.76	0.92

Alternatively, Table 8.8 may be used for the estimation of freeboard.

Table 8.8 Freeboard in irrigation channels (15)

Bed width (m)	*Discharge (m^3/s)*	*Freeboard(m)*
Less than 1.0	–	0.30
1 to 1.5	–	0.35
Greater than 1.5	Less than 3.0	0.45
–do–	3 to 30	0.60
–do–	30 to 60	0.75
– do–	Greater than 60	0.90

USBR has proposed the following formula for the estimation of freeboard F (in metre) in canals.

$$F = \sqrt{CD}$$

where, C is a constant varying form 0.45 (for discharges up to 0.07 m^3/s) to 0.76 (for discharges greater than 85 m^3/s) and D is the water depth in metre.

8.6.4.4. Canal Banks

Canal banks hold water within the water section of a channel. Suitable bank dimensions of an earth channel depend on size of channels, height of water surface above natural ground, amount and nature of excavated earth available for bank construction, and need of inspection roads along the channel. Bank widths at all elevations must provide stability against water pressure at the sides of the channel section. They should also keep percolating water below ground level outside the banks and prevent piping of bank materials.

A canal bank should be of such width that there is a minimum cover of 0.5 m above the saturation line. For large embankments of major canal projects, the position of the saturation line is determined as in case of earth dams and the stability of slope is computed using principles of soil mechanics. This has been discussed in Chapter 15. The saturation line for small embankments is drawn as a straight line from the point where full supply level meets the bank. The slope of the saturation line, *i.e.,* the hydraulic gradient, may very from $4H : 1V$ for relatively impermeable material (such as ordinary loam soil) to $10H : 1V$ for porous sand and gravel. For clayey soils, the hydraulic gradient may be steeper than $4(H) : 1(V)$. If the bank section does not provide a minimum cover of 0.5 m above the saturation line, a counter berm, shown in Fig. 8.8(*b*), is provided. Alternatively, the outer slope of the bank is flattened.

The Central Water Commission has recommended bank widths as given in Table 8.9.

Table 8.9 Bank widths in irrigation channels

Discharge (m^3/s)	*Top width of bank (m)*
less than 0.28	0.92
0.28 to 1.4	1.22
1.4 to 4.2	1.50
4.2 to 10.0	1.83
10.0 to 14.0	2.44
14.0 to 28.0	3.66
28.0 to 140.0	4.58

8.6.4.5. Service Road and 'Dowla'

For proper maintenance and inspection of irrigation channels, service roads are provided on both sides of main canals and major branches. In the case of smaller branches and distributaries, a service road on only one side (usually, the left bank) is provided. The inspection road on main canals is usually about 6.0 m wide and should not be smaller than 5.0 m wide. The canal roads are generally unsurfaced but made motorable by using compacted and dressed earth material. If the canal road is to be used for other purposes as well, it should be metalled and surfaced.

If a road is constructed on the ground along a banked canal, the canal is not visible while driving along the road. As such, the vehicle has to be stopped frequently at places to have a look at the canal. Thus, proper inspection of the canal is difficult. This problem can be avoided if the road and the bank are combined together, [Fig. 8.8(*b*)] and a 'dowla' of about 0.5 m height and about 0.5 m top width is provided for safety reasons. The level of the road is kept about 0.3 to 0.75 m above full supply level of the canal.

8.6.5. Schedule of Area Statistics and Channel Dimensions

The calculations for the design of an irrigation channel are usually carried out in tabular form. The table containing these calculated values is called the 'schedule of area statistics and channel dimensions'. The calculations start from the tail end and the design is usually carried out at every kilometre of the channel downstream of the head of the channel. Intermediate sections are designed only in special circumstances such as a large reduction in discharge due to an offtaking channel in between the adjacent two sections. Example 8.10 illustrates the procedure for the preparation of schedule of area statistics and channel dimensions for an irrigation channel.

Example 8.10 Design the first 5 km of a distributary channel which takes off from a branch the bed of which is at 224.0 m and the water depth in the branch is 2.0 m. The channel is to be designed for Rabi irrigation of intensity 30 per cent. Outlet discharge factor is 1800 hectares/m^3/s.

CCA	= 70 per cent of GCA
Evaporation and seepage losses	= 2.5 $m^3/s/10^6$ sq.m
Critical velocity ratio	= 1.05
Manning's n	= 0.023
Silt factor, f_1	= 1.0

Losses downstream of 5 km section = 0.28 m^3/s
Field application efficiency = 65%
Provide slope close of Lacey's slope

Distance from head of canal in km	*GCA (hectares)*
0	20,800
1	18,850
2	15,990
3	12,610
4	10,725
5	9,230

Ground Levels

Dist. (km)	*RL (m)*	*Dist. (km)*	*RL(m)*
0.0	225.00	2.6	223.20
0.2	225.00	2.8	223.05
0.4	225.15	3.0	222.90
0.6	225.10	3.2	222.80
0.8	225.05	3.4	222.65
1.0	224.95	3.6	222.50
1.2	224.25	3.8	222.55
1.4	224.10	4.0	222.40
1.6	224.05	4.2	222.30
1.8	224.00	4.4	222.25
2.0	223.95	4.6	222.20
2.2	223.35	4.8	222.15
2.4	223.20	5.0	222.00

Solution: Calculations for this problem have been shown in Table 8.10. Columns 1, 2 and 7 are as per the given data. CCA (col. 3) is obtained by multiplying GCA with given factor of 0.7. CCA (col. 3) is multiplied by the intensity of irrigation (30 per cent, *i.e.*, 0.3) to obtain the area to be irrigated for Rabi crop (col. 4). If intensities of irrigation for Kharif crops and sugarcane crop (or some other major crop) are also known, then corresponding areas to be irrigated for Kharif crop (col. 5) and sugarcane or other crop (col. 6) can also be similarly obtained. For known outlet discharge factors for Rabi, Kharif and Sugarcane (or other) crops, corresponding outlet discharges can be calculated by dividing the area to be irrigated with the corresponding outlet discharge factor and also the field application efficiency. Maximum of these outlet discharges is entered in col. 10 of Table 8.10.

Now a step-by-step method is followed from the tail end to determine channel dimensions in different reaches.

To the outlet discharge below 5.0 km (col. 10) is added the given losses downstream of 5 km section (*i.e.*, 0.28 m^3/s) entered in col. 12 to obtain the total discharge (col. 13).

Irrigation channels are designed for a discharge which is 10 per cent more than the total discharge required so that the channels can carry increased supplies in times of keen demand. The design discharge has been entered in col. 14.

Channel dimensions (*i.e.*, *S, B,* and *h*) can now be computed using a suitable method. Bed slope has been obtained from Lacey's equation [Eq. (8.35)]. This value of slope is modified to the nearest multiple of 2.5 and entered in col. 15. The method proposed by Ranga Raju and Misri (8) has been used to obtain *B* and *h*. Alternatively, Kennedy's method of trial or Lacey's equations can be used to obtain the channel dimensions.

In the present problem, since the adopted value of *p* is different from the value of *p* obtained from Fig. 8.7, the values of velocity obtained from the Manning's equation [Eq. (8.12)], and the Kennedy's equation [(Eq. 8.9)] should be compared. If the two values differ considerably, the channel dimensions should be revised suitably. For the present problem, the difference varies from 1.5 per cent to 6.5 per cent only for the chosen channel dimensions.

Having obtained the channel dimensions at the tail end (in this case at 5 km section), the losses in the reach between km 4 and km 5 are estimated. The water surface width at the km 5 section is 4.65 m (side slope of channel is $1/2H : 1V$). Assuming average water surface width in the reach (between km 4.0 and km 5.0) as 5.0 m, the loss in the reach is equal to $5 \times 1000 \times 2.5 \times 10^{-6} = 0.0125$ m^3/s (col. 11) which is added to the losses downstream of 5 km (0.28 m^3/s) to obtain total losses downstream of 4 km which comes to 0.2925 m^3/s (col. 12).

Channel dimensions can now be estimated for the reach between km 4.0 and km 5.0 as explained for the reach downstream of km 5.0. The average water surface width for the reach between km 4.0 and km 5.0 works out to (1/2) (4.9 + 4.65) = 4.775 m and the corresponding reach loss is 0.012 m^3/s which is not much different from the assumed reach loss of 0.0125 m^3/s. If the difference is large, the computations of channel dimensions may have to be revised.

Following the above procedure the channel dimensions up to the head of canal are determined. The computations have been shown in Table 8.10.

Longitudinal section is shown plotted in Fig. 8.10. Also shown in this figure are two falls which should be located keeping in view the requirements and guidelines discussed in Sec. 8.6.2.

8.7. BORROW PITS, SPOIL BANKS, AND LAND WIDTH FOR IRRIGATION CHANNEL

Borrow Pits and Spoil Banks

Although it is advisable to keep the channel in balanced earth work, it is generally not possible to do so. If the amount of earth required for filling is more than the amount of excavated earth, then the excess requirement of filling is met by digging from suitably selected areas known as borrow pits, Fig. 8.11 (*a*).

If unavoidable, borrow pits should be made in the bed or berms of the channel. These pits will silt up after sometime when water has flowed in the canal. The depth of the borrow pits is kept less than 1 m and the width is limited to half the bed width. The borrow pits are located centrally in the channel bed and are spaced such that the distance between adjacent borrow pits is at least half the length of the borrow pits. Borrow pits may be similarly located in wide berms of a channel.

If the material from internal borrow pits is not sufficient to meet the requirement then extra material is taken from external borrow pits which should be about 5 to 10 m away from the toe of the canal bank. These should not be deeper than 0.3 m and should always be connected to a drain.

Table 8.10 Schedule of area statistics and channel dimensions (Example 8.10)

Distance from head of canal	*GCA*	*CCA*	*Area to be irrigated*			*Outlet discharge factor*		
			Rabi	*Kharif*	*Sugarcane or other major crop*	*Rabi*	*Kharif*	*Sugarcane or other crop*
km	*ha*	*ha*	*ha*	*ha*	*ha*	$ha/m^3/s$	$ha/m^3/s$	$ha/m^3/s$
1	*2*	*3*	*4*	*5*	*6*	*7*	*8*	*9*
0	20800	14560	4368	–	–	1800	–	–
1	18850	13195	3958.5	–	–	1800	–	–
2	15990	11193	3357.9	–	–	1800	–	–
3	12610	8827	2648.1	–	–	1800	–	–
4	10725	7507.5	2252.25	–	–	1800	–	–
5	9230	6461	1938.3	–	–	1800	–	–

Table 8.10 (continued)

Distance from head of canal	*Outlet discharge*	*losses in reach*	*Total losses*	*Total discharge*	*Design discharge*	*Channel Dimensions*					
						Bed slope	$\frac{SQ^{0.02}}{(n^2m^2)}$	*p from Fig. 8.6*	*Adopted* $p = B/h$	*h from Eq.(8.16)*	*Channel width* $B = ph$
km	m^3/s	m^3/s	m^3/s	m^3/s	m^3/s					*m*	*m*
	10	*11*	*12*	*13*	*14*	*15*	*16*	*17*	*18*	*19*	*20*
0	3.733	0.0165	0.3540	4.0870	4.50	2.50×10^{-4}	0.442	5.0	4.0	1.23	4.92
1	3.383	0.0160	0.3375	3.7205	4.10	2.50×10^{-4}	0.441	5.0	4.0	1.19	4.76
2	2.870	0.0150	0.3215	3.1915	3.50	2.50×10^{-4}	0.440	5.0	4.0	1.12	4.48
3	2.263	0.0140	0.3065	2.5695	2.83	2.50×10^{-4}	0.438	5.0	4.0	1.03	4.12
4	1.925	0.0125	0.2925	2.2175	2.44	2.75×10^{-4}	0.480	4.0	4.0	0.98	3.92
5	1.657	–	0.2800	1.9370	2.13	2.75×10^{-4}	0.479	4.0	4.0	0.93	3.72

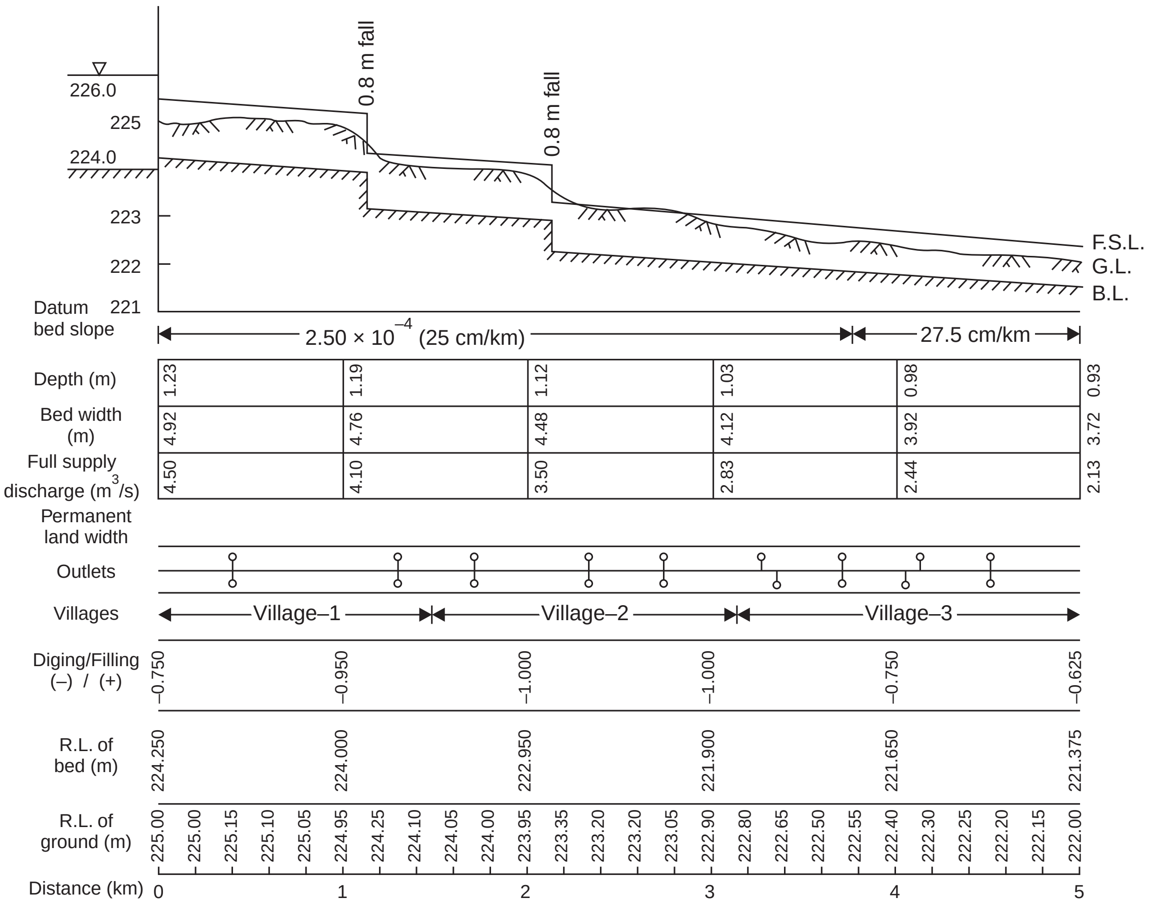

Fig. 8.10 Longitudinal section of distributary channel (Example 8.10)

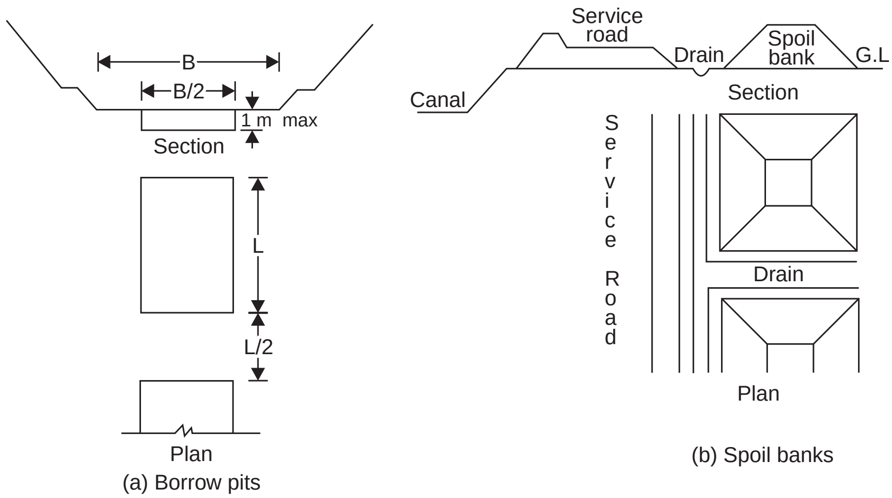

Fig. 8.11 Borrow pits and spoil banks

On the other hand, if the excavated earth exceeds the requirement of earth material for the construction of banks and service roads, the excess earth has to be suitably disposed of. If the excess earth is not much, it can be used to widen or raise the canal banks. If the quantity of excess earth is much larger, then it is utilised to fill up local depressions in the area or deposited in spoil banks, Fig. 8.11 (*b*), on one or both sides of the channel. The section of spoil banks depends on the cost of the land and labour. These should be provided with good cross slopes on all sides for ensuring proper drainage. The spoil banks should be discontinuous to allow cross drainage between them.

Land Width

Land width required for the construction of a channel includes the permanent and temporary land. The permanent land extends a little beyond the outer toe of the canal banks (or the drain, if provided) on either side. If trees are to be planted along the canals, then the land acquired for this purpose should also be included in the permanent land. Planting of trees adjacent to good culturable land should be avoided as the shade of the trees would affect the plants.

In addition to the permanent land, some land along the channel is required during construction for the storage of materials and equipment, and other purposes related to the construction work. This temporary land is returned to the concerned owners after use with due compensation.

8.8. SEDIMENT DISTRIBUTION IN AN ALLUVIAL CHANNEL

The sediment distribution along a vertical is given by the Rouse equation [Eq. (7.34)]. In the channels of north India, the concentration of sediment discharge by weight near the water surface is found to be 40 to 70 per cent of the average value which is generally found at about 0.6 times depth below the water surface. The percentage of coarse sediment continuously increases towards the bed.

Lateral sediment distribution is such that there is generally greater concentration of sediment near the banks than at the centre. This is due to cross currents in a channel (Fig. 8.12).

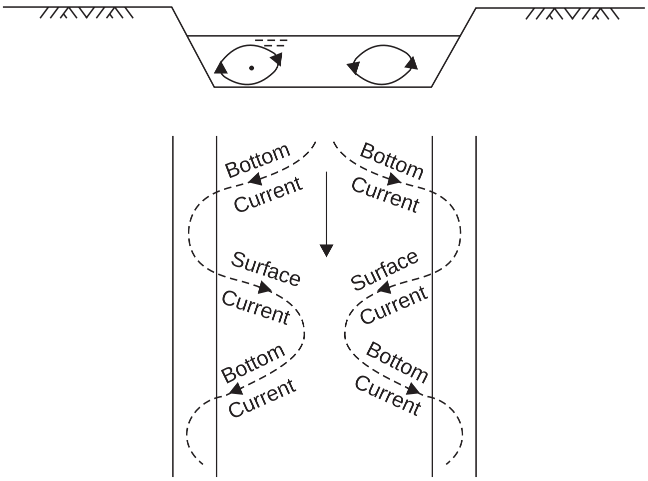

Fig. 8.12 Cross currents in a channel

Surface water flowing faster than the lower layer water, tends to topple over the slower moving water and goes to the bed of the channel. At the bed, this water is deflected towards the bank. This cross current at the bed (where the concentration of coarse sediment is the greatest) from the centre towards the bank pushes the coarse sediment towards the banks. The return current is toward the centre and upwards and, hence, the lifting up of the sediment is opposed by gravity.

The tendency of silting near the banks due to cross currents can be reduced by accelerating the flow in the bank region by some means such as pitching the bank slope.

Flow along the bends in the channel is such that there is heading up of water near the outer bank due to which there is a cross current of bottom water towards the inner bank. This leads to the deposition of sediment near the inner bank. Thus, there is shallow depth on the inner bank of the bend and greater depth on the outer bank of the bend. A channel should, therefore, offtake from the outer bank if it has to draw relatively clear water.

In a bell-mouthed converging channel, the sediment concentration is maximum near the central part of the channel. An offtaking channel from such a reach would, therefore, draw comparatively less sediment.

8.9. SILTING AND BERMING OF CHANNELS

In the head reaches of a distributary channel there may exist a tendency of silting up due to one or more of the following reasons :

(*i*) *Defective Head Regulator :* A defective head regulator may make the offtaking channel draw a higher percentage of coarser sediment. This increases the silt factor and sediment load of the channel and leads to the silting of the channel.

(*ii*) *Non-regime Section :* If the channel section is not able to carry its sediment load, the excess sediment is deposited on the channel bed. In due course, this deposition of sediment increases the channel slope so that the channel becomes capable of carrying the sediment which enters the channel. The channel may, then, continue in temporary equilibrium of 'initial regime'.

(*iii*) *Inadequate Slope :* If the channel slope is less than the regime slope, the sediment will deposit on the channel bed so as to increase the slope to the regime slope. In cases where the ground slope is less than the regime slope, the entry of coarse sediment into the channel should be minimised so that the silt factor of the channel sediment reduces.

(*iv*) *Fluctuation in Supply :* When the channel is running with low supplies for a long period, it gets silted up in the head reaches.

(*v*) *Defective Outlets :* If the outlets do not draw their share of sediment in proportion with the water discharge, the channel will have a tendency to silt up. This is due to the fact that downstream of the outlet, the parent channel has to carry relatively more sediment with reduced water discharge.

In the tail reaches of a channel, the discharge and velocity are both reduced. The velocities near the sides of the channel are very low and, hence, the silting takes place near the sides. The grass on the channel sides also catches the sediment. This is termed 'berming' of the channel and should be distinguished from the 'berms' which are deliberately provided in the channel section. The berming results in reduction of the channel section and, hence, rise in water levels. To improve the channel condition in the tail reaches, sometimes berm cutting may have to be carried out.

EXERCISES

8.1 What are the 'true regime' conditions in an alluvial channel as stipulated by Lacey ?

8.2 A trapezoidal channel is 5.0 km long and has a trapezoidal cross-section with the side slopes of 0.5 H : 1V. Up to the first 2 km, the bed-width of the channel is 4.90 m and the depth of flow is 1.05 m. Thereafter, the bed-width and the depth are 4.0 m and 0.885 m, respectively, right up to the downstream end of the channel. Estimate the total seepage loss through the channel if the rate of seepage loss is 3 m^3/s per million square metres of the water surface area exposed.

8.3 A stable channel is to be designed for a discharge of 40 m^3/s and the silt factor of unity. Calculate the dimensions of the channel using Lacey's regime equations. What would be the bed-width of this channel if it were to be designed on the basis of Kennedy's method with critical velocity ratio equal to unity and the ratio of bed-width to depth of flow the same as obtained from Lacey's method.

8.4 For the design of an irrigation channel through a flat ground it has been decided to avail fully the available slope of 25 cm/km, and adopt Lacey's method of design by restricting the discharge. Design the channel taking the average size of bed material as 0.0005 m and side slope of 0.5H : 1V.

8.5 Design an irrigation channel to carry 200 m^3/s of flow with a bed load concentration of 100 ppm by weight. The average grain diameter of the bed material is 1.0 mm. Use Lacey's method and a suitable bed load equation for the design. Side slope of the channel is 0.5H : 1V.

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9

SURFACE AND SUBSURFACE FLOW CONSIDERATIONS FOR DESIGN OF CANAL STRUCTURES

9.1. GENERAL

Canal structures include the following types of structures:

(*i*) Communication structures, such as roads and railways which have to be constructed across the channels. Such structures are in the form of bridges and are not included in this book.

(*ii*) Regulation structures which are meant for controlling discharges, velocity, and water levels in the channels. Canal falls, distributary head regulators, escapes, *etc.,* are examples of regulation structures.

(*iii*) Cross-drainage structures which are required to pass natural drainage across channels. Aqueducts, siphon aqueducts, siphons, and superpassages are examples of cross-drainage structures.

Canal structures listed at (*ii*) and (*iii*) above may fail on account of effects of either surface flow or subsurface flow. Water flowing over the structure causes hydrostatic forces, formation of hydraulic jumps, and scour upstream and downstream of the structure. Considerations of subsurface flow are important on all hydraulic structures which have foundations other than that of solid impervious rock. Subsurface flow endangers the stability of hydraulic structures in two ways—by piping and uplift pressure. Piping failure occurs when the seepage water is left with sufficient force to lift up soil particles at the downstream end of a hydraulic structure where it emerges. Uplift pressure is the pressure exerted by the seeping water on a hydraulic structure. If this pressure is not counterbalanced by the weight of concrete or masonry floor, the structure may fail because of rupture of the floor.

9.2. HYDRAULIC JUMP

Hydraulic jump occurs when, in the same reach of a channel, the upstream control causes supercritical flow while the downstream control dictates subcritical flow. Hydraulic jump is always accompanied by considerable turbulence and energy dissipation. The following are the useful applications of hydraulic jump:

(*i*) The dissipation of energy of flow downstream of hydraulic structures such as dams, spillways, weirs, *etc*.

(*ii*) The reduction of net uplift pressures under hydraulic structures by raising the water depth on the apron of the structure.

(*iii*) The maintenance of high water levels in channels for water distribution purposes.

(*iv*) The mixing of chemicals for water purification or other purposes (in chemical industries).

On applying Newton's second law of motion to the control volume, shown in Fig. 9.1, one obtains,

$$P_1 - P_2 + W \sin\theta - F_f = \rho Q(\beta_2 u_2 - \beta_1 u_1) \tag{9.1}$$

where P_1 ($= \rho g \bar{z}_1 A_1$) and P_2 ($= \rho g \bar{z}_2 A_2$) are the pressure forces at sections 1 and 2, W the weight of liquid between sections 1 and 2, F_f the component of unknown forces (along the direction of flow) acting between sections 1 and 2, θ the longitudinal slope of the channel, β_1 and β_2 the momentum correction coefficients at sections 1 and 2, u_1 and u_2 are the average velocities at sections 1 and 2 and $\bar{z}_1$ and $\bar{z}_2$ are the distances to centroids of respective flow areas A_1 and A_2 from the free surface.

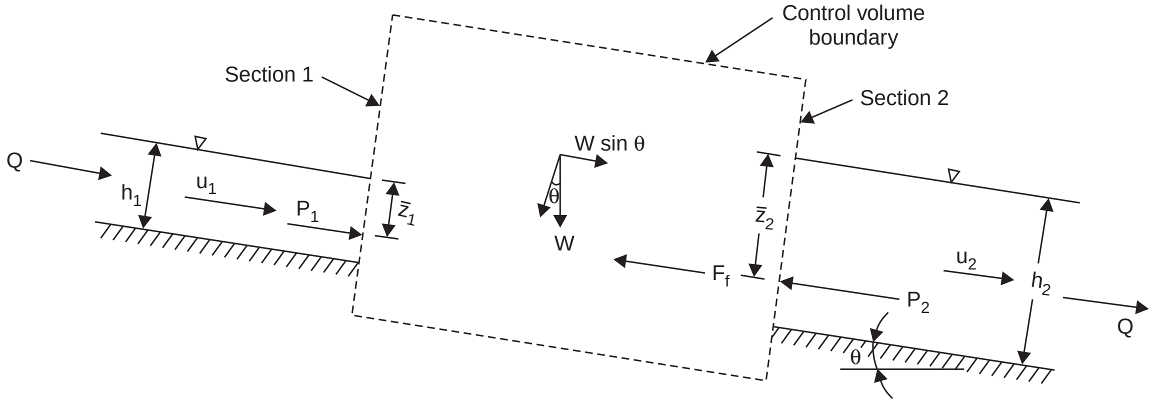

Fig. 9.1. Control volume for hydraulic jump

With the assumptions that θ is small (*i.e.,* $\sin\theta \cong 0$), and $\beta_1 = \beta_2 = 1$, Eq. (9.1) becomes

$$\rho g \bar{z}_1 A_1 - \rho g \bar{z}_2 A_2 - F_f = \rho Q (u_2 - u_1) \tag{9.2}$$

Equation (9.2) can be rewritten as

$$\frac{F_f}{\rho g} = \left(\frac{Q^2}{gA_1} + A_1 \bar{z}_1\right) - \left(\frac{Q^2}{gA_2} + A_2 \bar{z}_2\right) \tag{9.3}$$

or
$$\frac{F_f}{\rho g} = M_1 - M_2 \tag{9.4}$$

where,
$$M = \frac{Q^2}{gA} + A\bar{z} \tag{9.5}$$

and M is termed the specific momentum or force function or specific force.

If the jump occurs in a horizontal channel and is not assisted by any other means, such as baffle blocks, then $F_f \cong 0$, and Eq. (9.4) yields

$$M_1 = M_2 \tag{9.6}$$

or

$$\frac{Q^2}{gA_1} + A_1\,\bar{z}_1 = \frac{Q^2}{gA_2} + A_2\,\bar{z}_2 \tag{9.7}$$

9.2.1. Hydraulic Jump in Rectangular Channels

A hydraulic jump formed in a smooth, wide, and horizontal rectangular channel is termed classical hydraulic jump. For rectangular channel of width B,

$$Q = u_1 A_1 = u_2 A_2$$

$$A_1 = B_1\,h_1 \quad \text{and} \quad A_2 = B_2 h_2$$

$$\bar{z}_1 = h_1/2 \quad \text{and} \quad \bar{z}_2 = h_2/2$$

Substituting these values into Eq. (9.7) one can, after simplification, obtain

$$\frac{1}{2} h_1 h_2\,(h_1 + h_2) = \frac{q^2}{g} \tag{9.8}$$

where, $q = Q/B$. Equation (9.8) has the following solutions :

$$\frac{h_2}{h_1} = \frac{1}{2}\left(\sqrt{1 + 8F_1^{\,2}} - 1\right) \tag{9.9}$$

and

$$\frac{h_1}{h_2} = \frac{1}{2}\left(\sqrt{1 + 8F_2^{\,2}} - 1\right) \tag{9.10}$$

in which, F_1 and F_2 are the Froude numbers at sections 1 and 2, respectively. Froude number F equals $u/\sqrt{gD}$ in which, D is the hydraulic depth and equals A/T where, T is the top width of flow. Equations (9.9) and (9.10) are the well-known Belanger's momentum equations.

9.2.2. Energy Loss in Hydraulic Jump in a Rectangular Channel

In a horizontal rectangular channel with the channel bed chosen as the datum, the total energies at sections 1 and 2, (Fig. 9.1 with θ = 0) are equal to the specific energies E_1 and E_2 at sections 1 and 2, respectively, *i.e.,*

$$E_1 = h_1 + \frac{q^2}{2g\,h_1^{\,2}} \tag{9.11}$$

$$E_2 = h_2 + \frac{q^2}{2g\,h_2^{\,2}} \tag{9.12}$$

so that energy loss,

$$\Delta E = E_1 - E_2 \tag{9.13}$$

$$= h_1 - h_2 + \frac{q^2}{2g}\left[\frac{1}{h_1^{\,2}} - \frac{1}{h_2^{\,2}}\right]$$

$$= (h_1 - h_2) + \frac{1}{4} h_1 h_2 (h_1 + h_2)\left(\frac{h_2^{\,2} - h_1^{\,2}}{h_1^{\,2} h_2^{\,2}}\right)$$

$$= \frac{4h_1^2h_2 - 4h_2^2h_1 + h_1h_2^2 - h_1^3 + h_2^3 - h_1^2h_2}{4h_1h_2}$$

$$= \frac{h_2^3 - h_1^3 + 3h_1^2h_2 - 3h_1h_2^2}{4h_1h_2}$$

$$\therefore \qquad \Delta E = \frac{(h_2 - h_1)^3}{4h_1h_2} \tag{9.14}$$

Also,

$$\frac{\Delta E}{E_1} = \frac{[h_1 + (u_1^2/2g)] - [h_2 + (u_2^2/2g)]}{[h_1 + u_1^2/2g]} \tag{9.15}$$

$$= \frac{h_1\,[1 - (h_2/h_1)] + (q^2/2g\,h_1^2)\,[1 - (h_1/h_2)^2]}{(h_1/2)\,[2 + F_1^2]}$$

$$\therefore \qquad \frac{\Delta E}{E_1} = \frac{2 - 2\,(h_2/h_1) + F_1^2\,[1 - (h_1/h_2)^2]}{2 + F_1^2} \tag{9.16}$$

Combining Eq. (9.8), (9.11) and (9.14), one can obtain

$$\frac{\Delta E}{E_1} = \frac{8F_1^4 + 20F_1^2 - (8F_1^2 + 1)^{3/2} - 1}{8F_1^2\,(2 + F_1^2)} \tag{9.17}$$

Hence, for a supercritical Froude number F_1 equal to 20, the energy loss ΔE is equal to 0.86 E_1. This means that 86 per cent of the initial specific energy is dissipated. Because of this energy dissipating capability, hydraulic jump is widely used as an energy dissipator for spillways and other hydraulic structures. In a hydraulic jump, the mean kinetic energy is first converted into turbulence and then dissipated through the action of viscosity. Equation (9.17) can be rewritten as

$$\frac{E_2}{E_1} = \frac{(8F_1^2 + 1)^{3/2} - 4F_1^2 + 1}{8F_1^2\,(2 + F_1^2)} \qquad ...(9.18)$$

The term E_2/E_1 is called the efficiency of the jump.

9.2.3. Length of Hydraulic Jump

The length of a hydraulic jump is defined as the distance from the front of the jump to a section of the flow immediately downstream of the roller associated with the jump. Although a very important design parameter, the length of a hydraulic jump, L_j cannot be derived from theoretical considerations. The length of a hydraulic jump is approximately equal to five times the height of the jump which itself is $(h_2 - h_1)$. Silvester (1) has shown that for horizontal rectangular channels, the ratio L_j/h_1 is a function of the upstream supercritical Froude number. He obtained,

$$\frac{L_j}{h_1} = 9.75\,(F_1 - 1)^{1.01} \tag{9.19}$$

The length of a classical hydraulic jump L_j can also be estimated from Fig. 9.2 which shows the variation of L_j/h_2 with F_1.

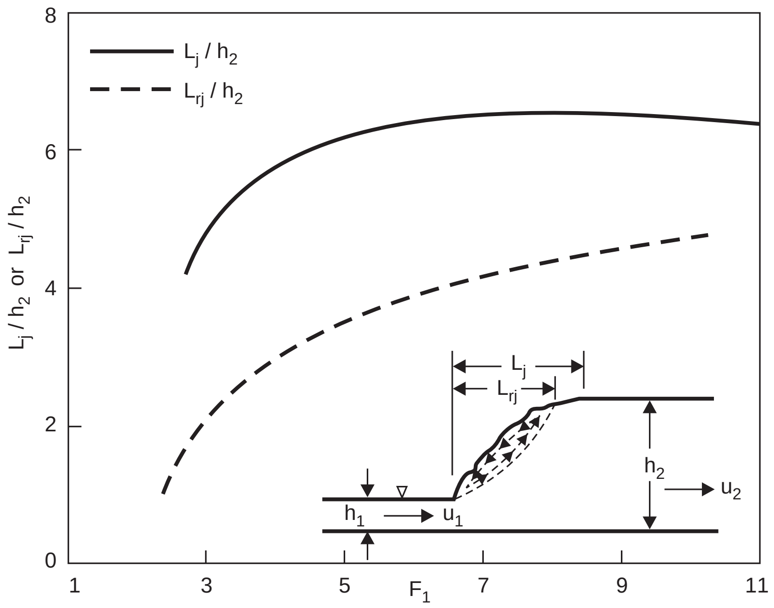

Fig. 9.2 Length characteristics of jump (2)

9.2.4. Profile of Hydraulic Jump

In case of overflow structures located on permeable foundations, the concrete aprons of the stilling basins are subjected to uplift pressures which are partly counterbalanced by the weight of water flowing on the apron. Therefore, in the hydraulic jump type stilling basins, determination of profile of the jump becomes necessary. Rajaratnam and Subramanya (3) have obtained an empirical relation, (Fig. 9.3), between $y/[0.75(h_2 - h_1)]$ and $x/\overline{X}$. Here, $\overline{X}$ is the distance from the beginning of the jump to the section where the depth measured above the x-axis is $0.75(h_2 - h_1)$. The length $\overline{X}$ is empirically related to h_1 and F_1 as

$$\frac{\overline{X}}{h_1} = 5.08\,F_1 - 7.82 \tag{9.20}$$

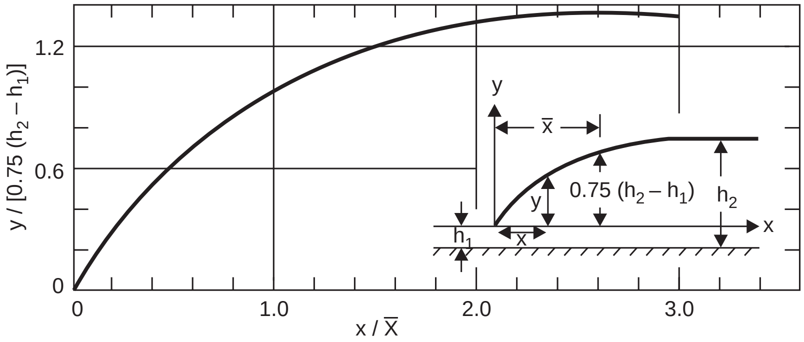

Fig. 9.3 Profile of the hydraulic jump in a rectangular channel (3)

9.2.5. Calculations for Hydraulic Jump in Horizontal Rectangular Channels

Equations (9.8), (9.11), (9.12), and (9.14) can be used to obtain direct solution of the jump parameters (*i.e.,* E_2, h_2, ΔE, and E_1) for horizontal rectangular channels, if h_1 and q are known. These equations would also yield direct solution for E_1, h_1, ΔE, and E_2, if h_2 and q are known. One can also use Eq. (9.9) or Eq. (9.10) instead of Eq. (9.8) depending upon whether pre-jump or post-jump conditions are known.

However, in an actual design problem, generally the discharge q and the levels of the upstream and downstream total energy lines are known. Thus, q and ΔE are known. Determination of the remaining four parameters of the jump from Eqs. (9.8), (9.11), (9.12), and (9.14) is rather difficult. This difficulty can be overcome by the use of critical depth h_c $(= (q^2/g)^{1/3})$ and defining

$$X = (h_1/h_c)\ ;\ Y = (h_2/h_c)\ ;\ Z = (\Delta E/h_c)\ ;$$
$$\xi = E_1/h_c, \text{ and } \eta = E_2/h_c$$

so that Eqs. (9.8), (9.11), (9.12), and (9.14) reduce to the following forms respectively :

$$XY(X + Y) = 2 \tag{9.21}$$

$$\xi = X + \frac{1}{2X^2} \tag{9.22}$$

$$\eta = Y + \frac{1}{2Y^2} \tag{9.23}$$

and

$$Z = \frac{(Y - X)^3}{4XY} \tag{9.24}$$

Here, X can vary from 0 to 1 only. Using Eqs. (9.21) to (9.24), one can obtain the sets of values of Y, ξ, η and Z for different values of X and, thus, the curves shown in Fig. 9.4. These curves are known as Crump's curves (4). The method to use these curves is as follows:

(*i*) Calculate h_c from $h_c = (q^2/g)^{1/3}$

(*ii*) Compute Z, *i.e.*, $\Delta E/h_c$.

(*iii*) Read h_2/h_c from the curve $\Delta E/h_c$ versus h_2/h_c.

(*iv*) Read E_2/h_c from the curve E_2/h_c versus h_2/h_c.

(*v*) Thus, $E_1 = \Delta E + E_2$.

(*vi*) For known E_1/h_c, obtain h_1/h_c from the curve E_1/h_c versus h_1/h_c.

Thus, for given q and ΔE, one can determine h_2, E_2, E_1, and h_1 from the relationships for Crump's coefficients.

Combining Eqs. (9.21) and (9.24), one can obtain (5),

$$Z = \frac{-X^6 + 20X^3 + 8 - (X^4 + 8X)^{3/2}}{16X^2} \tag{9.25}$$

and also

$$Z = \frac{-Y^6 - 20Y^3 - 8 - (Y^4 + 8Y)^{3/2}}{16Y^2} \tag{9.26}$$

For different values of X and Y, the values of Z can be obtained and the curves Y-Z prepared. Approximate equations for these curves were obtained as (5)

$$Y = 1 + 0.93556\ Z^{0.368} \text{ for } Z < 1 \tag{9.27}$$

and

$$Y = 1 + 0.93556\ Z^{0.240} \text{ for } Z > 1 \tag{9.28}$$

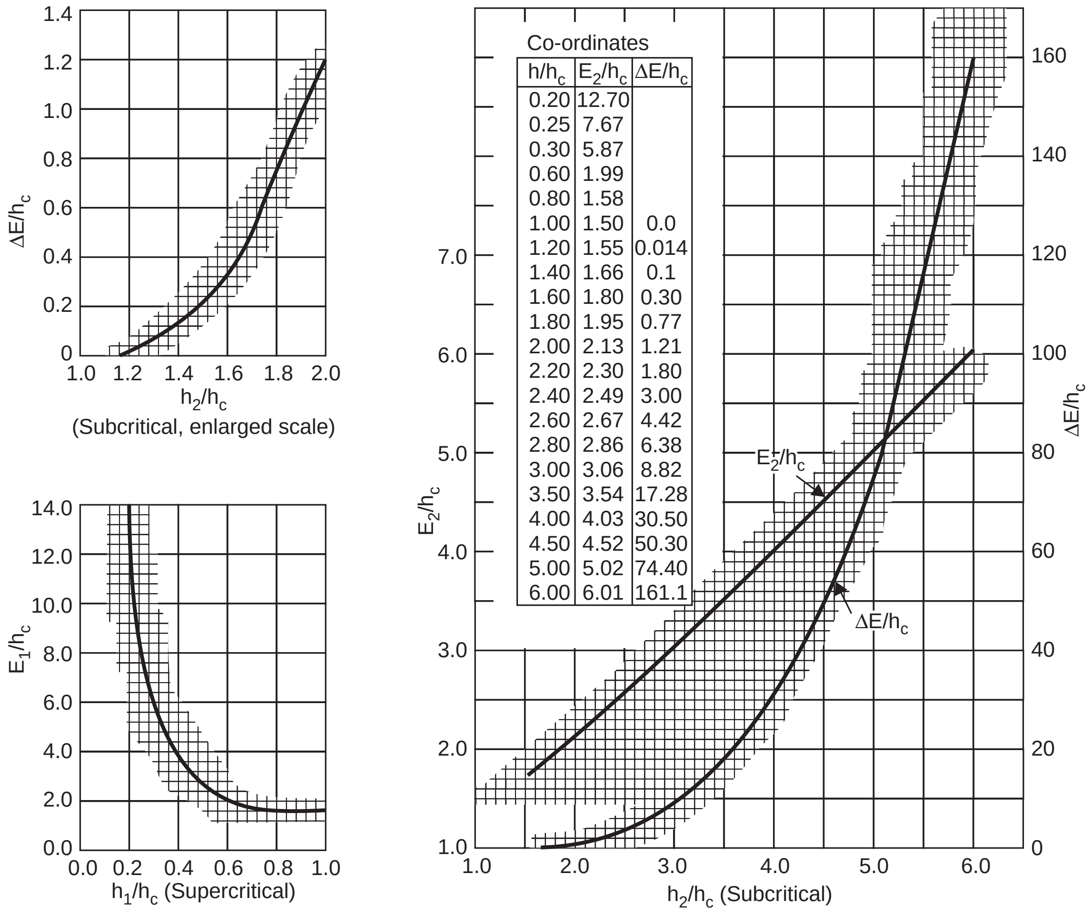

Fig. 9.4 Relationship for Crump's coefficients (4)

One of the two equations (Eqs. (9.27) and (9.28)) can be used for obtaining the value of Y for a specified value of Z. Equations (9.21), (9.22), and (9.23) can then be used for the determination of X, ξ, and η respectively.

Equations (9.25) and (9.26), can be solved to prepare a table for computations of hydraulic jump elements in rectangular channels (Table 9.1).

Table 9.1 Hydraulic jump elements in rectangular channels (5)

Z	X	Y	$\frac{E_1}{h_c}$	$\frac{E_2}{h_c}$	$\frac{Y}{X} = \frac{h_2}{h_1}$	$\frac{\Delta E}{E_1}$
0.01	0.839	1.180	1.549	1.539	1.406	0.006
0.10	0.681	1.407	1.760	1.660	2.067	0.057
0.50	0.516	1.728	2.396	1.896	3.351	0.209
1.00	0.436	1.936	3.069	2.069	2.442	0.326
1.50	0.389	2.082	3.700	2.200	4.356	0.406

(Contd.)...

2.00	0.356	2.199	4.303	2.303	6.179	0.465
2.50	0.331	2.298	4.893	2.393	6.940	0.511
3.00	0.311	2.384	5.472	2.472	7.659	0.548
4.00	0.281	2.531	6.609	2.609	9.002	0.605
4.50	0.269	2.594	7.169	2.669	9.638	0.628
5.00	0.259	2.654	6.725	2.725	10.254	0.647
6.00	0.241	2.761	8.826	2.826	11.439	0.680
7.00	0.227	2.856	9.917	2.917	12.572	0.706
8.00	0.215	2.942	11.000	3.000	13.663	0.727
10.00	0.197	3.094	13.146	3.146	15.744	0.761
15.00	0.165	3.396	18.439	3.439	20.527	0.814
20.00	0.145	3.640	23.878	3.878	25.079	0.846

9.2.6. Hydraulic Jump on Sloping Channels

On a horizontal floor with little friction, location of hydraulic jump varies considerably with a slight change in the depth or velocity of flow. But, on a sloping floor, location of hydraulic jump is relatively stable and can be closely predicted. However, energy dissipation in the case of jumps on a sloping floor is less owing to the vertical component of velocity remaining intact.

Equation (9.1) is theoretically applicable to hydraulic jumps forming on sloping channels. But the solution of the problem is difficult due to the following reasons :

(*i*) The length and shapes of the hydraulic jump are not well-defined and, hence, the term $W \sin \theta$ is poorly computed.

(*ii*) The specific weight of the liquid in the control volume can change considerably due to air entrainment.

(*iii*) The pressure terms cannot be determined accurately.

Figure 9.5 shows several cases of hydraulic jumps on sloping channels. In the studies of hydraulic jump on sloping channels, the end of the surface roller is taken as the end of the jump. This means that the length of roller (measured horizontally) is the length of the jump.

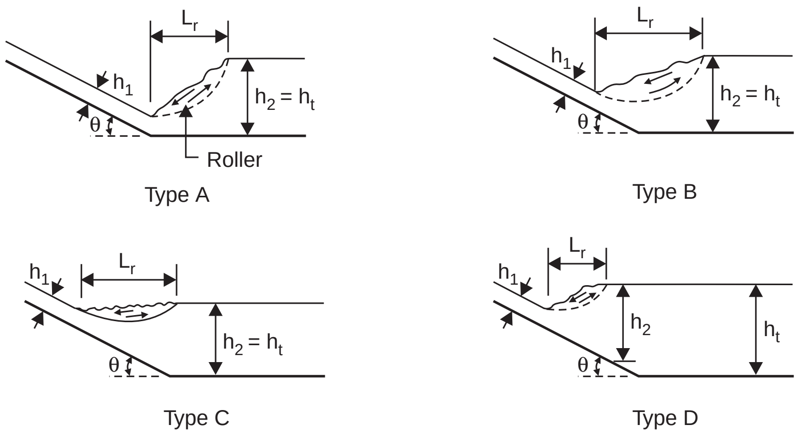

Fig. 9.5 Different types of hydraulic jump which form in sloping channels

When the jump begins at the end of the sloping apron, type *A* jump occurs and $h_2 = h_2^* = h_t$. Here, h_2 is the subcritical sequent depth corresponding to h_1, h_t the tail-water depth, and h_2^* is the subcritical sequent depth h_2 given by Eq. (9.9). Type *A* jump is, obviously, governed by Eqs. (9.9) and (9.10).

When the end of the jump coincides with that of the sloping bed, type *C* jump occurs. For this case, Kindsvater (6) developed the following equation for the sequent depth h_2 :

$$\frac{h_2}{h_1} = \frac{1}{2\cos\theta}\left[\sqrt{1 + 8\,F_1^{\,2}\left(\frac{\cos^3\theta}{1 - 2N\tan\theta}\right)} - 1\right] \tag{9.29}$$

in which, θ is the longitudinal slope angle of the channel, and N is an empirical coefficient dependent on the length of the jump. Equation (9.29) can be rewritten as

$$\frac{h_2}{h_1'} = \frac{1}{2}\,[\sqrt{1 + 8\,G_1^{\,2}} - 1] \tag{9.30}$$

where,
$$h'_1 = h_1/\cos\theta \tag{9.31}$$

and
$$G_1^2 = \frac{\cos^3\theta}{1 - 2N\tan\theta} \times F_1^2 \tag{9.32}$$

Rajaratnam (6) gave the following simple expression :

$$\frac{\cos^3\theta}{1 - 2N\tan\theta} = 10^{0.054\,(\theta)} \tag{9.33}$$

where, θ is in degrees.

When h_t is greater than the sequent depth h_2 required for type *C* jump, then type *D* jump occurs completely on the sloping apron. Bradley and Peterka (7) found that Eqs. (9.30) to (9.33) valid for type *C* jump can be used for the type *D* jump also.

If h_t is less than that required for type *C* jump but greater than h_2^*, the toe of the jump is on the sloping bed, and the end of the jump on the horizontal bed. This jump is classed as type *B* jump. A graphical solution (Fig. 9.6) has been developed for this type of jump (7).

Bradley and Peterka (7) have developed plots for the estimation of the length of the type *D* jump (Fig. 9.7). These plots can also be used to determine the lengths of the types *B* and *C* jumps.

The energy loss for the type *A* jump can be estimated from Eq. (9.14). For *C* and *D* jumps, one can write

$$E_1 = L_j\tan\theta + \frac{h_1}{\cos\theta} + \frac{u_1^2}{2g} \tag{9.34}$$

and
$$E_2 = h_2 + \frac{u_2^2}{2g} \tag{9.35}$$

Thus,

$$\frac{\Delta E}{E_1} = \frac{(1 - h_2/h_1) + (F_1^2/2)\,[1 - \{1/(h_2/h_1)\}^2] + [(L_j/h_2)\,(h_2/h_1)]\tan\theta}{1 + (F_1^2/2) + (L_j/h_2)\,(h_2/h_1)\tan\theta} \tag{9.36}$$

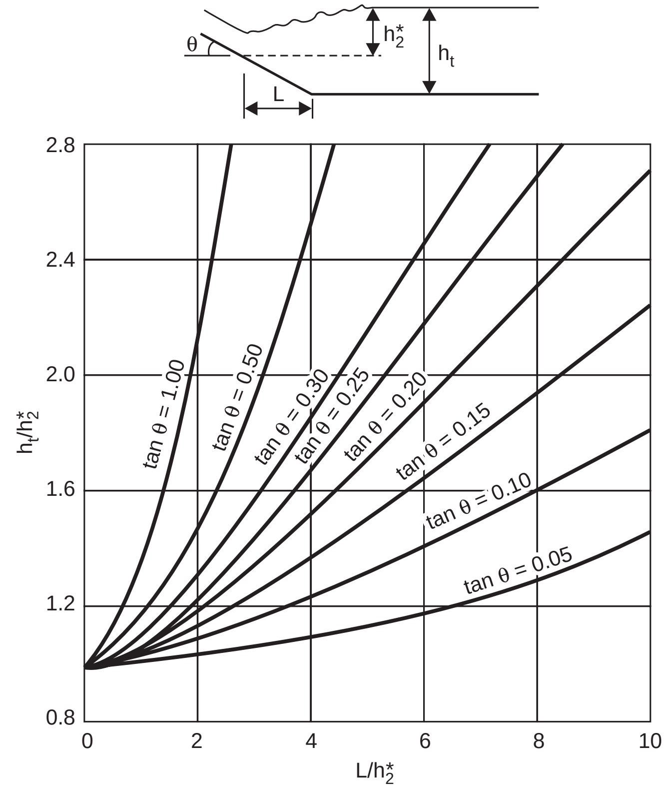

Fig. 9.6 Solution for *B* jump (2)

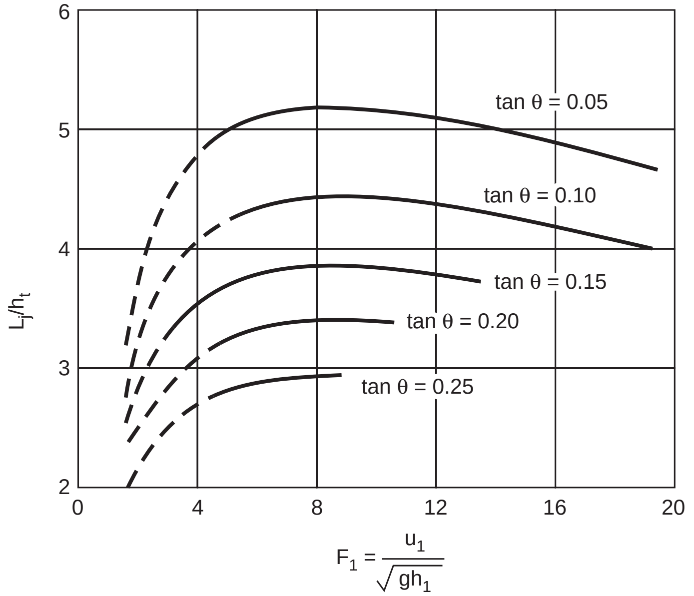

Fig. 9.7 Hydraulic jump length for jump types *B*, *C* and *D* (7)

Here, the bed level at the end of the jump has been chosen as the datum and the potential energy term $h_1/\cos\theta$ has been approximated as h_1. Equation (9.36) should not be used when F_1 is less than 4 as in this range very little is known about L_j/h_2 which affects $\Delta E/E_1$. In order to solve a problem of hydraulic jump on a sloping channel, the first step is to determine the type of jump for given slope, the pre-jump supercritical depth, and the tail-water condition, Figure 9.8 illustrates the procedure for the determination of the type of hydraulic jump.

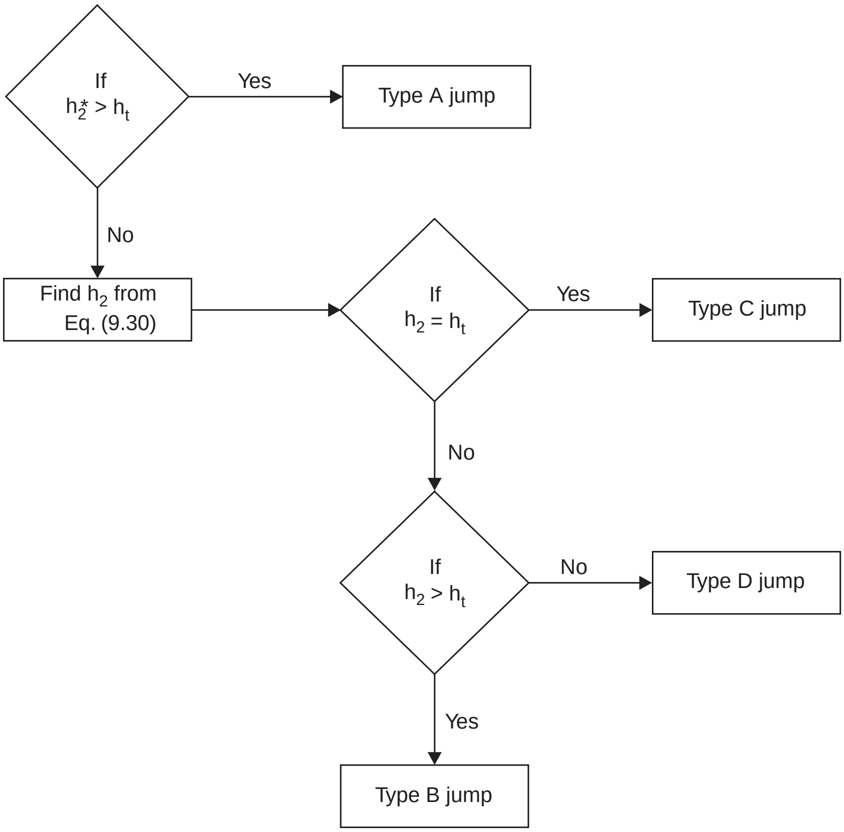

Fig. 9.8 Determination of type of hydraulic jump on sloping channels

Example 9.1 A discharge of 9.0 m^3/s flows in a 6.0 m wide rectangular channel which is inclined at an angle of 3° with the horizontal. Determine the type of jump if h_1 = 0.10 m and h_t = 2.6 m.

Solution:

$$F_1 = \frac{9.0/(6.0\times 0.1)}{\sqrt{9.81\times 0.10}} = 15.15$$

h_2^* (*i.e.,* the sequent depth in a horizontal channel) can be calculated from

$$h_2{}^* = \frac{h_1}{2}\left[\sqrt{1+8F_1{}^2}-1\right] = \frac{0.1}{2}\left[\sqrt{1+8(15.15)^2}-1\right]$$

$$= 2.09 \text{ m}$$

Since $h_t > h_2^*$, the depth h_2 should be calculated from Eqs. (9.30)-(9.33).

$$G_1^2 = 10^{0.054\,(\theta)}\, F_1^2$$

$$= 10^{0.054\,(3)} \times (15.15)^2$$

$$= 333.29$$

$$\therefore \qquad \frac{h_2}{(0.10/\cos 3°)} = \frac{1}{2}\left[\sqrt{1 + 8\, G_1^2} - 1\right]$$

or
$$h_2 = \frac{0.10}{2 \cos (3°)}\left[\sqrt{1 + 8\,(333.29)} - 1\right] = 2.54 \text{ m}$$

Since $h_2 < h_t$, the jump is classified as the type D jump. Using Fig. 9.7, the length of the jump, L_j, can be determined by obtaining L_j / h_t for tan 3° = 0.05 and F_1 = 15.15.

$$\frac{L_j}{h_t} = 4.9$$

$$\therefore \qquad L_j = 4.9 \times 2.6 = 12.74 \text{ m}$$

Now
$$\frac{h_t}{h_1} = \frac{2.6}{0.1} = 26$$

and
$$\frac{h_2}{h_1} = \frac{2.31}{0.1} = 23.1$$

Therefore, relative energy loss can be determined from Eq. (9.36)

$$\frac{\Delta E}{E_1} = \frac{(1 - 23.1) + [(15.15)^2/2]\,[1 - (1/23.1)^2] + \tan (3°)\,[4.9(26)]}{1 + [(15.15)^2/2] + (4.9)\,(26) \tan (3°)} = 0.82$$

9.2.7. Forced Hydraulic Jump

When the tail-water depth h_t is less than the required sequent depth h_2 corresponding to the pre-jump depth h_1, the jump is repelled downstream. However, by introducing devices such as baffle walls or baffle blocks and, thus, increasing the surface friction, the jump can be made to move upstream and form forcibly at the section it would have formed if sufficient tail-water depth (= h_2) were available. Such a jump is called the forced hydraulic jump (Fig. 9.9).

Figure 9.10 shows different types of forced hydraulic jump (8). For small Z and large x_o, type I jump is formed. This is similar to a free jump. When Z is increased and x_o is decreased, the baffle acts like an obstruction placed across the channel having free flow conditions and the jump is of type II*. With increase in tail-water depth, the obstruction is submerged and type II jump forms. On increasing Z and decreasing x_o further, the jump becomes more violent and is of type III. The effect of further increase in Z and decrease in x_o results in jumps of type IV, VI and VI*. *A* type VI* jump is a type VI jump with low tail-water depth. Between types IV and VI, there occurs an unstable transition phenomenon called type V (not shown in Fig. 9.10).

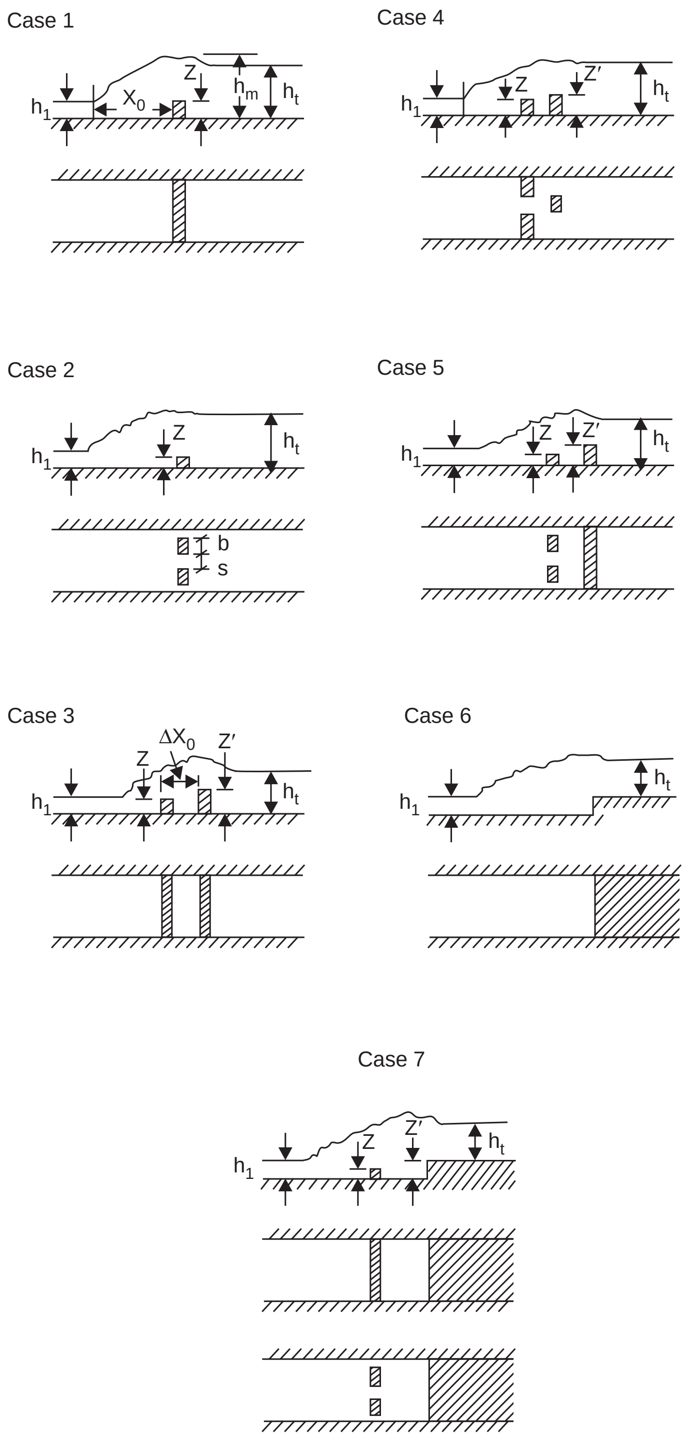

Fig. 9.9 Devices for producing forced hydraulic jump (8)

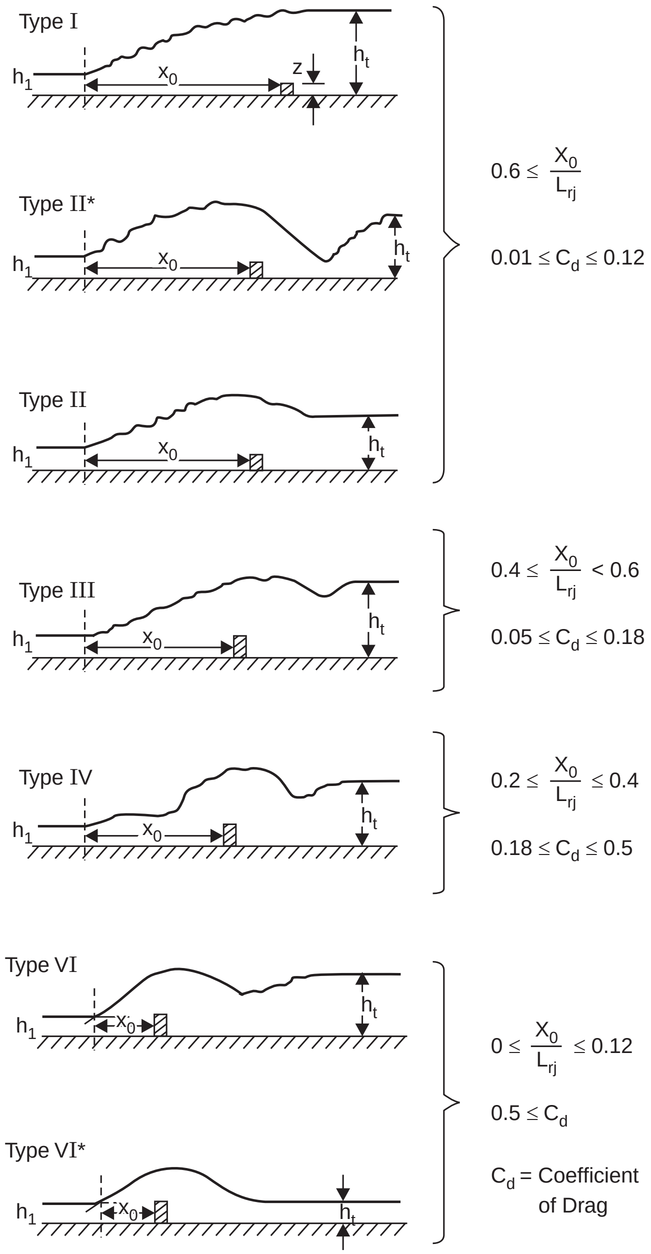

Fig. 9.10 Types of forced hydraulic jump (8)

A large number of experimental studies have been carried out on the forced hydraulic jump as it forms the basic design element of the well-known hydraulic jump type stilling basins. The simplest case of a forced hydraulic jump is the jump forced by a two-dimensional baffle, known as a baffle wall, of height Z kept at a distance x_o from the toe (*i.e.,* the beginning) of the jump (Fig. 9.9, case 1).

Considering unit width of the channel, the momentum equation can be written as

$$\frac{1}{2}\rho\, gh_1^2 - \frac{1}{2}\rho\, gh_t^2 - F_b = \rho\, q\left(\frac{q}{h_t} - \frac{q}{h_1}\right) \tag{9.37}$$

where, $$F_b = C_d(\rho\, u_1^2/2)\, Z.$$

Equation (9.37), on simplifying, gives

$$C_d = \frac{(\alpha - 1)\,[2F_1^2 - \alpha\,(1+\alpha)]}{F_1^2\, \alpha\, \beta} \tag{9.38}$$

where, $\alpha = h_t/h_1$ and $\beta = Z/h_1$. Rajaratnam (8) found experimentally that C_d obtained from Eq. (9.38) is a function of only x_o which is made dimensionless by the length of roller of the classical jump, L_{rj} (Figs. 9.2 and 9.11). Figure 9.11 is based on data from only one source and, therefore, needs further verification. A design chart (Fig. 9.12) has been developed (8), using Eq. (9.38), with Ψ (= h_t/h_2 = $(h_t/h_1)(h_1/h_2)$ = α/ϕ where, $\phi = h_2/h_1$) versus F_1 for various values of $\beta\, C_d$. Choosing a C_d value of 0.4 (a very competitive design) or smaller (for a conservative design), obtain the value of $C_d\, \beta$ (and, hence, β) from Fig. 9.12, for known Ψ and F_1. Now obtain x_o/L_{rj} from Fig. 9.11. Using Fig. 9.2, L_{rj} (and, hence, x_o) can now be determined. From known β and h_1, the depth of baffle wall Z can also be determined.

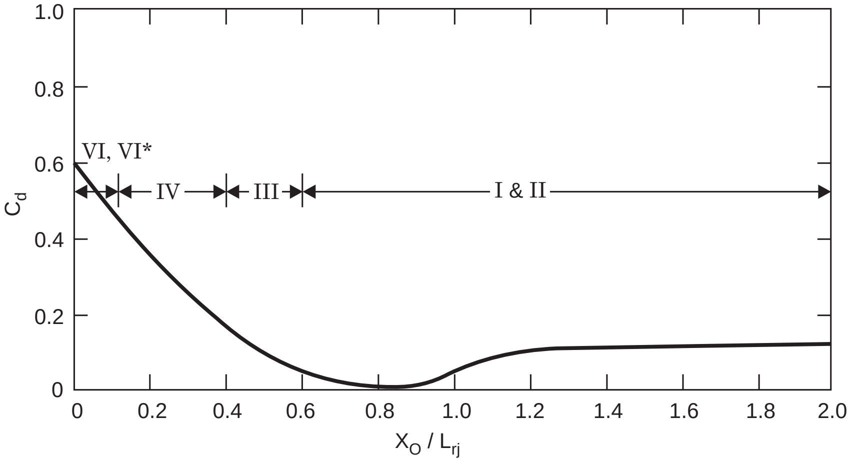

Fig. 9.11 Variation of the drag coefficient for forced hydraulic jump (8)

Baffle block (also known as baffle pier or friction block) is the case of a three-dimensional baffle wall. Baffle blocks are generally trapezoidal in shape and are placed in a single row or in two rows with staggered pattern (Fig. 9.13). The momentum equation [Eq. (9.37)] is applicable for this case too but with a different expression for F_b (the force exerted by the baffle blocks per unit width of the channel) which can be written as

$$\frac{F_b}{F_2} = f\left(\frac{x_o}{h_1}, \beta, F_1, \eta\right) \tag{9.39}$$

Here, $F_2 = \frac{1}{2}\rho\, gh_2^2$ and η (*i.e.,* the blockage ratio) = $W/(W + S)$ (Fig. 9.13). Based on the analysis of data of Basco and Adams (10), Ranga Raju *et al.* (9) found that F_1 is unimportant in Eq. (9.39) and $\Psi_1 \Psi_2 F_b / F_2$ is uniquely related to x_o/h_1 as shown in Fig. 9.14. Ψ_1 and Ψ_2 are empirical correction factors and are functions of Z/h_1 and η, respectively, as shown in Figs. 9.15 and 9.16.

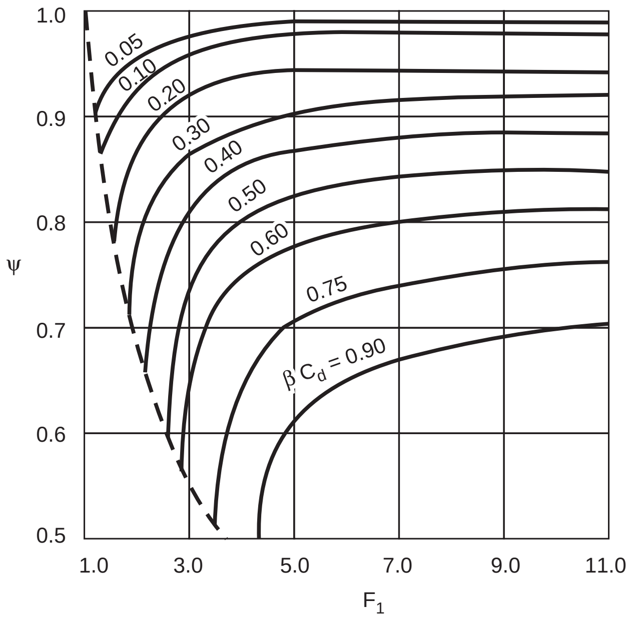

Fig. 9.12 Design chart for baffle wall (9)

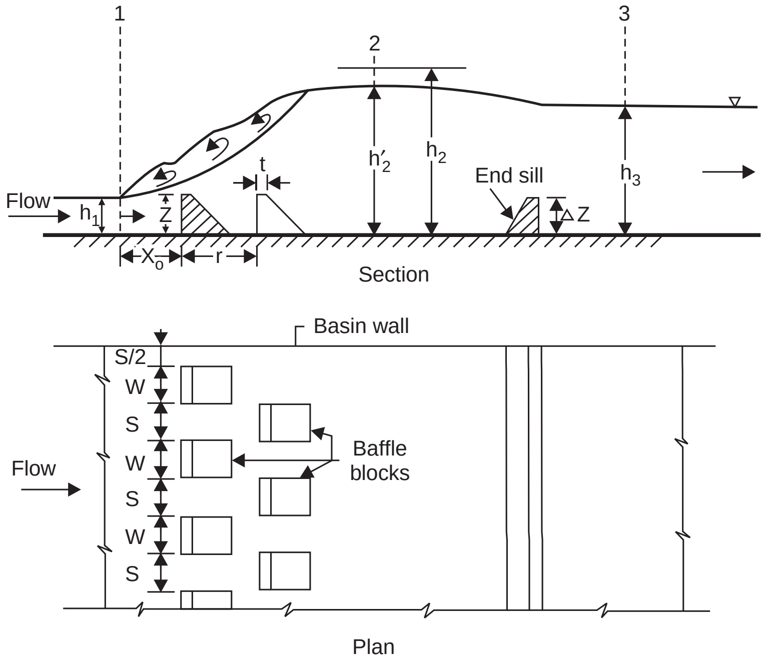

Fig. 9.13 Arrangement of trapezoidal baffle blocks

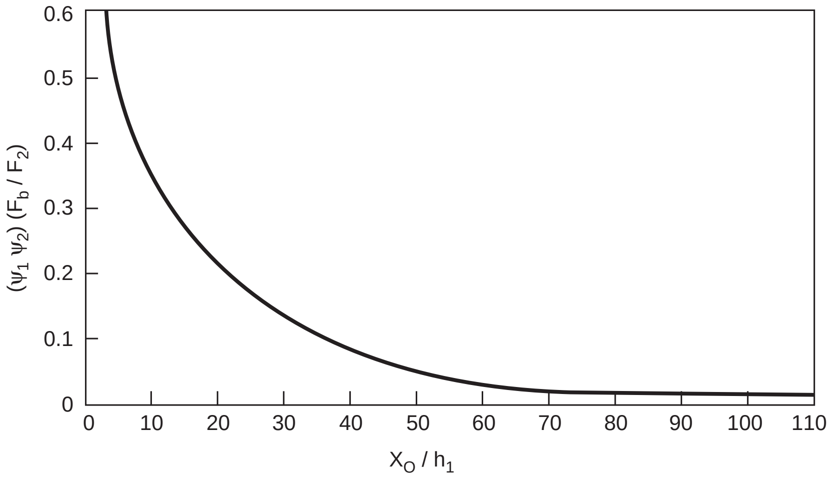

Fig. 9.14 Variation of $(\Psi_1 \Psi_2\, F_b/F_2)$ with x_o/h_1 for trapezoidal blocks (9)

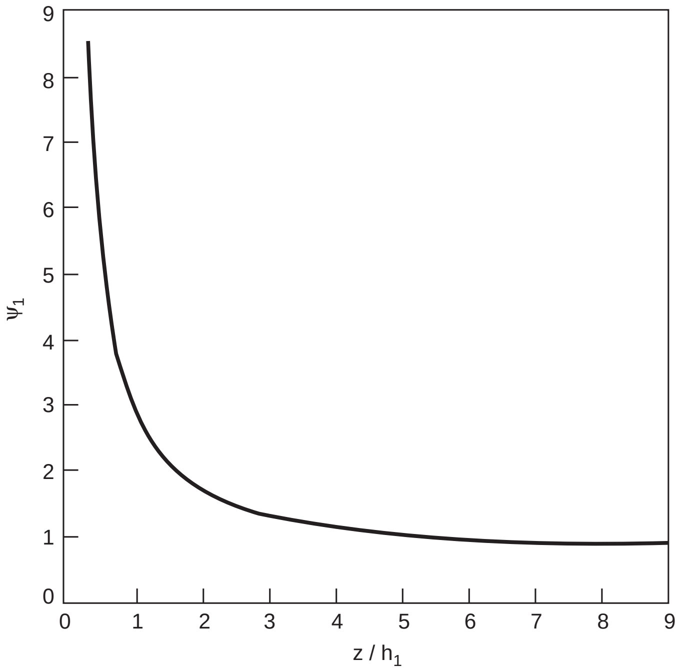

Fig. 9.15 Variation of Ψ_1 with Z/h_1 (9)

These data also indicated no change in the value of F_b when baffle blocks were placed in two rows for the range of r/Z from 2.5 to 5.0. Here, r is the spacing between the two rows of blocks.

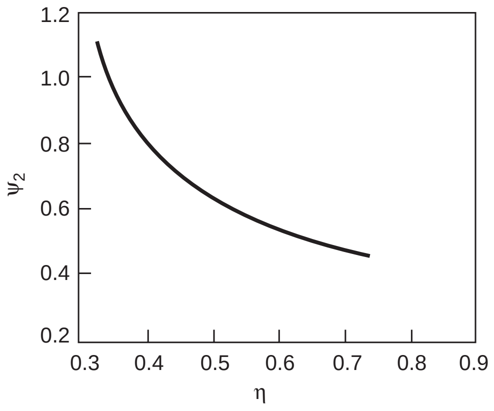

Fig. 9.16 Variation of Ψ_2 with η (9)

9.2.8. Location of Hydraulic Jump on Glacis

It is generally assumed that the jump forms at the junction of the glacis with the horizontal floor and, therefore, the relations for classical hydraulic jump are used for determining the location of the jump on glacis. For known discharge intensity q, and the difference in the levels of the upstream and downstream levels of total energy line, ΔE, one can determine the downstream specific energy, E_2, using a suitable method or Blench's curves (Fig. 9.17). The location of the jump on a glacis can then be obtained by finding the intersection of the glacis with the horizontal plane E_2 below the downstream level of the total energy line.

9.3. SEEPAGE FORCE

All the canal structures constructed in India up to the 19th century were mainly designed on the basis of the experience and intuition of the designer. The canal structures, not founded on solid rock, have the problem of seepage. Henry Darcy (11) was the first to propose an experimental relationship [Eq. (4.1)] for flow of water through porous medium. In 1895, following the damage to the Khanki weir on Chenab river a test program was initiated (12). These experiments proved the validity of Darcy's law. The experiments and observations on the Narora weir confirmed that the piping effect and uplift pressure due to seepage endanger the stability of structure founded on a permeable foundation.

Bligh (13) gave a simple method to calculate uplift pressure below a masonry or concrete structure. He stated that the length of the path of flow had the same effectiveness in reducing uplift pressures irrespective of the direction of flow. The length of the flow path along the bottom profile of the structure, *i.e.*, length of the uppermost flow line, was termed creep length. Thus, Bligh made no distinction between horizontal creep and vertical creep. According to Bligh's theory, the creep length (*i.e.*, the length of flow path) L in an idealised weir profile (Fig. 9.18) is equal to $2d_1 + b_1 + 2d_2 + b_2 + 2d_3$. Thus, the loss of head per unit creep length is H/L which is the average hydraulic gradient. Here, H is the seepage head. Bligh termed the average hydraulic gradient as the percolation coefficient C and assigned safe values to it for different types of soil. For coarse-grained soil, the value of C is taken as 1/12 while for sand mixed with boulder and gravel, and for loam soil, the value of C may vary from 1/5 to 1/9. For fine micaceous sand of north Indian rivers, the value of C is 1/15. According to Bligh, if the average hydraulic gradient is less than the assigned safe value of C, there will be no danger of piping.

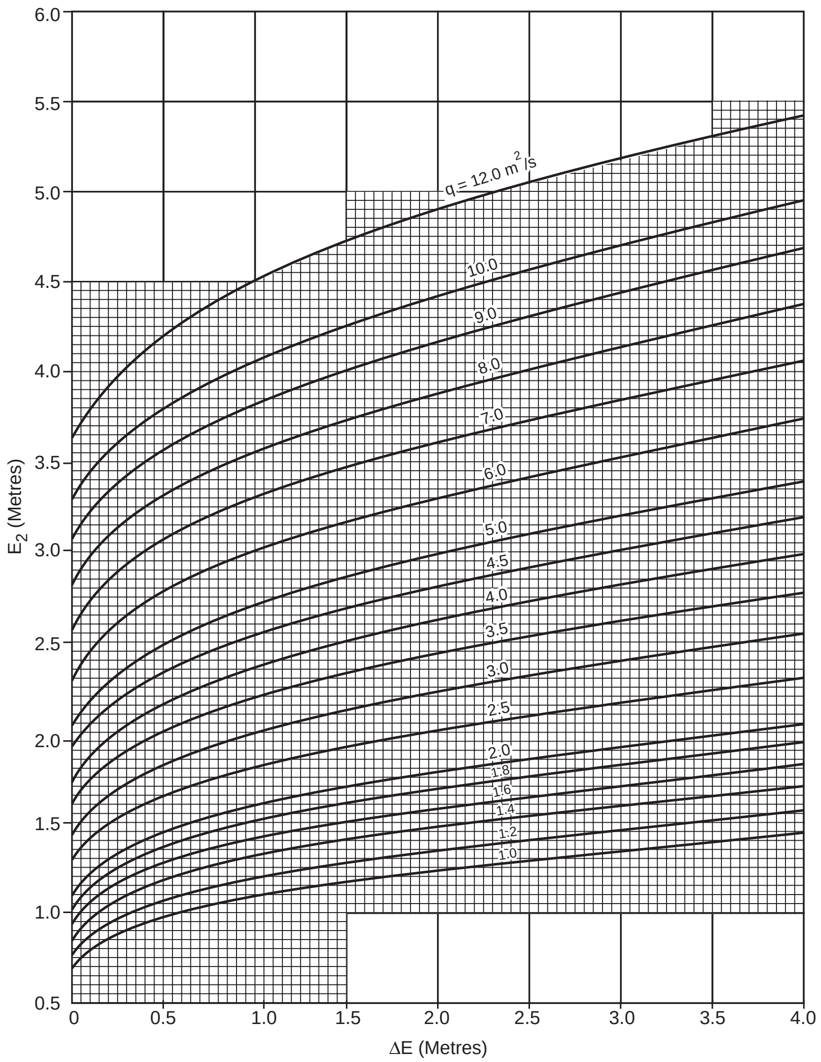

Fig. 9.17 Blench's curves

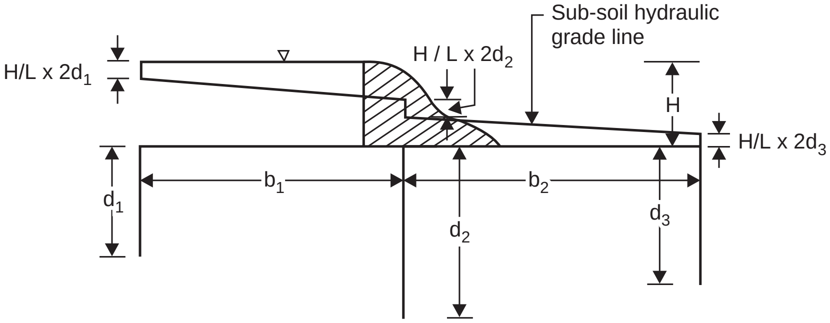

Fig. 9.18 Subsoil hydraulic grade line

It should be noted that the seepage head H is the difference between water levels upstream and downstream of the weir. The worst condition will occur when water is held up to the highest possible level on the upstream side with no flow. The downstream level is then taken as the downstream bed level.

The elevation of the subsoil hydraulic gradient line above the bottom of the floor at any point measures the uplift pressure at that point. If h' is the height of the subsoil hydraulic gradient line above the bottom of the floor, the uplift pressure of subsoil water exerted on the floor is $\rho gh'$. Assuming the mass density of the floor material as ρ_s and the floor thickness as t, the downward force per unit area due to the weight of the floor is $t\rho_s g$. For equilibrium, these two should be equal.

$$\therefore \qquad \rho gh' = t\,\rho_s g \qquad (9.40)$$

or

$$t = \frac{h'}{(\rho_s/\rho)}$$

The surface profile of the floor is determined from the surface flow considerations and is known. But h', measured from the bottom of the floor, can be known only if the thickness of the floor t is known. Equation (9.40) is, therefore, rewritten as

$$h' = t(\rho_s/\rho)$$

Subtracting t from both sides,

$$h' - t = t(\rho_s/\rho) - t$$

$$= t[(\rho_s/\rho) - 1]$$

$$\therefore \qquad t = \frac{h' - t}{(\rho_s/\rho) - 1} \qquad (9.41)$$

$(h' - t)$ is the height of subsoil hydraulic gradient line measured above the top surface of the floor and, hence, is known. The thickness of the floor t can, therefore, be directly obtained.

On the upstream side of the barrier, the weight of water causing the seepage is more than sufficient to counterbalance the uplift pressure. In the absence of water upstream, there will be no seepage and, hence, no uplift pressure. Therefore, the upstream floor thickness may be kept equal to minimum practical thickness to resist wear and development of cracks. The floor downstream of the barrier must be in accordance with Eq. (9.41). This also suggests that it would be economical to keep as much floor length upstream as possible. A minimum floor length downstream of the barrier is, however, always required to resist the action of fast-flowing water. Shifting of the floor upstream also reduces the uplift pressure on the floor downstream of the barrier (Fig. 9.19) because a larger portion of the total loss of head has occurred up to the barrier. Further, an upstream cutoff reduces uplift pressure (Fig. 9.20) while a downstream cutoff increases uplift pressure all along the floor.

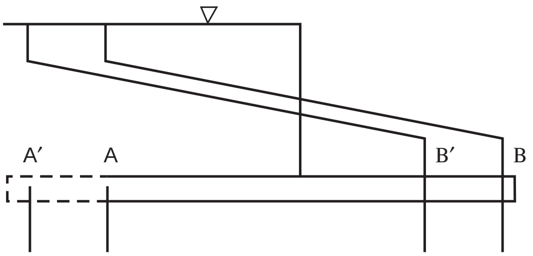

Fig. 9.19 Effect of shifting floor upstream on subsoil H.G.L.

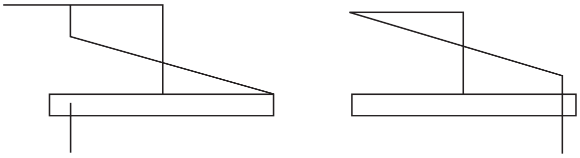

Fig. 9.20 Effect of U/S and D/S cutoffs on subsoil H.G.L

In 1932, Lane (14), after analysing 290 weirs and dams, evolved what is now known as weighted creep theory which, in effect, is Bligh's creep theory corrected for vertical creep. When the coefficient of horizontal permeability is three times the coefficient of vertical permeability, Lane suggested a weight of three for the vertical creep and one for the horizontal creep. Thus, for the case of Fig. 9.18, according to Lane's method, the creep length is $[3(2d_1 + 2d_2 + 2d_3) + (b_1 + b_2)]$. Inclined floors may be treated as vertical if its slope exceeds 45° and horizontal if the slope is less than 45°. Alternatively, for inclined floors, the weight may be taken as equal to $3[1 + (2\theta/90)]$ where θ is the angle (in degrees) of inclination of the floor with the horizontal. Because of their simplicity, Bligh's and Lane's methods are still useful for preliminary dimensioning of the floor. Bligh's method is, obviously, more conservative.

Following the appearance of cracks at the upstream and downstream ends due to the undermining of soil at the upper Chenab canal structures in 1926-27, Khosla, *et al*. (12) carried out some studies. These studies disclosed that the measured pressures were not equal to those calculated from Bligh's theory. Other notable findings of these studies (12) were as follows :

(*i*) The outer faces of the end sheet piles were more effective than the inner ones and the horizontal length of floor.

(*ii*) The intermediate piles, if smaller in length than the outer ones, were ineffective except for local redistribution of pressures.

(*iii*) Erosion of foundation soil below the structure started from the tail end. If the hydraulic gradient at the exit was more than the critical gradient for the soil below the structure, the soil particles would move with the flow of water thus causing progressive degradation of the soil and resulting in cavities and ultimate failure.

(*iv*) It was absolutely essential to have a reasonably deep vertical cutoff at the downstream end to prevent undermining.

In 1929, Terzaghi (15), based on his laboratory studies stated that failure occurred due to undermining if the hydraulic gradient at the exit was more than the floatation gradient. The floatation gradient is similar to the term 'critical gradient' used by Khosla, *et al*. (12). The floatation gradient implied a state of floatation of the soil at the toe of the work if the exit gradient exceeded the limit of 1 : 1 at which condition the upward force due to the flow of water was almost exactly counterbalanced by the weight of the soil.

The above-mentioned methods of determining seepage effects do not have any theoretical justification. Now that considerable knowledge of the theory of seepage is available, the equation governing the seepage flow has been solved to obtain the uplift pressures and the exit gradient.

9.3.1. Theory of Seepage

According to Darcy's law [Eq. (4.4)], the apparent velocities of flow of liquid through porous media are

$$u = -k\frac{\partial h}{\partial x}, \quad v = -k\frac{\partial h}{\partial y}, \quad \text{and} \quad w = -k\frac{\partial h}{\partial z} \tag{9.42}$$

where, k is the coefficient of permeability, h is the hydraulic head causing the flow, and u, v, and w, respectively, are the x-, y-, and z-components of velocity.

Substituting these in the continuity equation,

$$\frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} + \frac{\partial w}{\partial z} = 0 \tag{9.43}$$

and assuming that k is a constant, one obtains

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2} = 0 \tag{9.44}$$

This is the well-known Laplace equation which governs the flow of liquid through porous medium. This equation implicily assumes that,

(*i*) The soil is homogeneous and isotropic,

(*ii*) The voids are completely filled with water,

(*iii*) No consolidation or expansion of the soil takes place,

(*iv*) The soil and water are incompressible, and

(*v*) The flow obeys Darcy's law and is steady.

For two-dimensional flow, Eq. (9.44) reduces to

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = 0 \tag{9.45}$$

Equation (9.45) can be solved by graphical, analytical, numerical or some other suitable method, such as analogue method.

9.3.2. Graphical Solution of Seepage Equation

A graphical solution of Laplace equation results in two families of curves which intersect at right angle and form a pattern of '*square*' figures known as flownet (Fig. 9.21). One set of lines is called the streamlines or flow lines along which water can flow through a cross-section. The other set of lines comprise what are called equipotential lines which are lines of equal head or energy level. Any flownet must satisfy the following basic requirements:

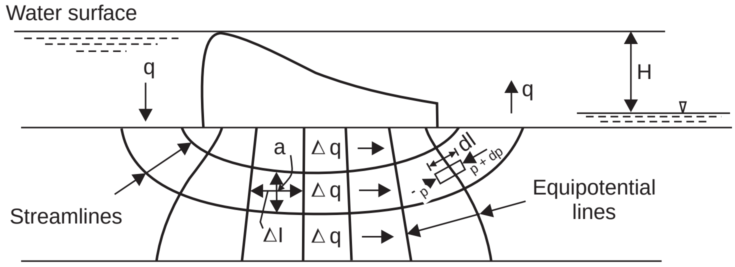

Fig. 9.21 Flownet for seepage under a weir

(*i*) Flow lines and equipotential lines must intersect at right angle to form areas which are approximately squares. Most flownets are composed of curves and not straight lines. As such, the "*square*" figures are not true squares but curvilinear squares.The requirement of square figures is met, if the average width of any squire is equal to its average length.

(*ii*) Certain entrance and exit requirements must be satisfied.

(*iii*) All pairs of adjacent equipotential lines must have equal head losses between them.

(*iv*) The same amout of seepage flows between all pairs of adjacent flow lines.

Using Darcy's law, a simple expression for seepage discharge can be obtained. Referring to Fig. 9.21, let the number of flow channels be N_f, the number of equipotential drops be N_d, and the seepage quantity flowing between any two adjacent flow lines be equal to Δq. Thus, the total seepage quantity,

$$q = N_f \Delta q$$

or
$$q = N_f k \frac{\Delta h}{\Delta l} a$$

or
$$q = N_f k \Delta h$$

as $\Delta l = a$, since the flownet is composed of squares.

Since
$$\Delta h = H/N_d,$$

$$q = N_f k \frac{H}{N_d} = kH \frac{N_f}{N_d} \tag{9.46}$$

Equation (9.46) enables computation of the seepage quantity.The ratio N_f/N_d is called the *shape factor*. The seepage quantity is, therefore, the product of the coefficient of permeability, the net head, and the shape factor.

Considering an elementary cylindrical element of soil of cross-sectional area dA and length dl along any flow line, shown in Fig. 9.21, one can formulate the expression for the net seepage force dF in the direction of flow as

$$dF = pdA - (p + dp)\, dA$$

or
$$dF = -\, dp\, dA$$

Therefore, seepage force per unit volume of soil,

$$\frac{dF}{dA\, dl} = -\frac{dp}{dl} = -\rho\, g \frac{dH}{dl} \tag{9.47}$$

As the head H decreases in the direction of flow, dH/dl is negative and, hence, the seepage force is positive in the flow direction.

Obviously, at the exit end (Fig. 9.21), the seepage force is vertical and may cause the lifting of soil particles resulting in piping failure. Hence, to provide safety against piping failure, the seepage force at the exit end must be less than the submerged weight of the soil particles. At the critical condition, the two forces will just balance each other. Thus,

$$-\rho\, g \frac{dH}{dl} = (1 - n)(\rho_s - \rho) g \tag{9.48}$$

Here, n is the porosity of the soil and $(\rho_s - \rho)g$ is the submerged weight of unit volume of soil particles. Dividing Eq. (9.48) by $\rho\, g$, one gets

$$-\frac{dH}{dl} = (1 - n)\left(\frac{\rho_s}{\rho} - 1\right) \tag{9.49}$$

or
$$-\frac{dH}{dl} = (1 - n)\,(G - 1) \tag{9.50}$$

where, G is the relative density of soil.

The quantity dH/dl represents the hydraulic gradient at the exit (or, simply, the exit gradient) which is negative. The value of dH/dl given by Eq. (9.50) is termed the critical gradient which should not be exceeded in order to prevent failure by piping.

Assuming $G = 2.65$ (true for most of the river sands) and $n = 0.4$, the value of the critical gradient is approximately 1.0. In practice, however, the actual gradient at the exit is kept around 1/4 to 1/6 depending on the safety requirements.

9.3.3. Method for Determination of Seepage Pressure

The seepage equation (or the Laplace equation), Eq. (9.45), cannot be solved exactly for usual canal structures having complex boundary conditions. Khosla, *et al*. (12) used the method of independent variables and obtained solutions of Laplace equation for a number of simple profiles. This solution is commonly known as Khosla's solution. The following forms of these simple profiles (Fig. 9.22) are very useful in the design of weirs and barrages on permeable foundations:

(*i*) A straight horizontal floor of negligible thickness with a sheet pile at either end [Fig. 9.22 (*i*) and (*ii*)].

(*ii*) A straight horizontal floor of negligible thickness with an intermediate sheet pile [Fig. 9.22 (*iii*)].

(*iii*) A straight horizontal floor depressed below the bed but with no vertical cutoff [Fig. 9.22 (*iv*)]. This arrangement is useful for very small structures where no cutoff is provided.

The solutions for these simple profiles have been obtained (12) in terms of the pressure head ratio ϕ at '*key*' points. These key points are the junction points of sheet piles with floor, *i.e.*, E, C, D, E_1, C_1, and D_1 in case of floors of negligible thickness and D' and D_1' in case of depressed floor. The pressure head ratio ϕ at any key point, say E, is thus H_E/H. Here, H_E is the uplift pressure head at E. The mathematical expressions for the pressure head ratio ϕ at the key points are as follows:

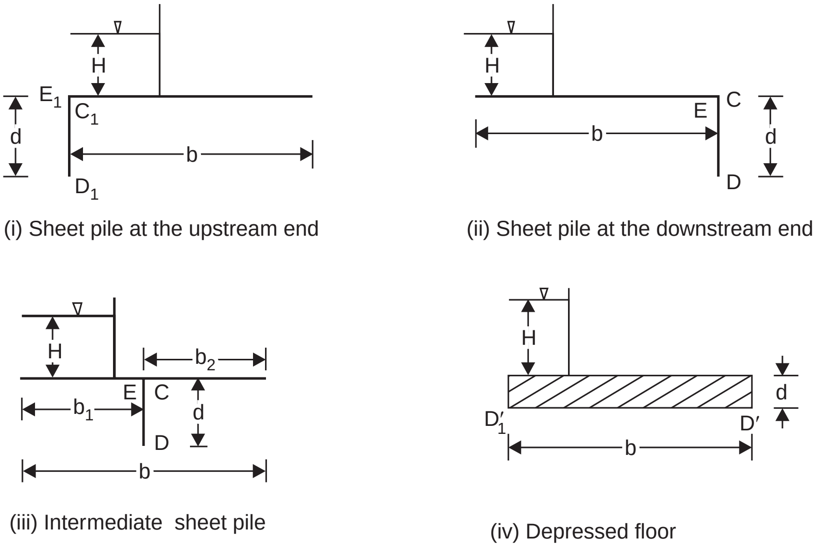

Fig. 9.22 Simple standard profiles of weir floors.

(*i*) For sheet piles at either the upstream end [Fig. 9.22 (*i*)] or the downstream end [Fig. 9.22 (*ii*)],

$$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right) \tag{9.51}$$

$$\phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 1}{\lambda}\right) \tag{9.52}$$

$$\phi_{C_1} = 100 - \phi_E \tag{9.53}$$

$$\phi_{D_1} = 100 - \phi_D \tag{9.54}$$

where,
$$\lambda = \frac{1}{2}[1 + \sqrt{1 + \alpha^2}] \tag{9.55}$$

and
$$\alpha = \frac{b}{d}$$

(*ii*) For sheet piles at the intermediate point [Fig. 9.22 (*iii*)],

$$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 - 1}{\lambda_2}\right) \tag{9.56}$$

$$\phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1}{\lambda_2}\right) \tag{9.57}$$

$$\phi_C = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 + 1}{\lambda_2}\right) \tag{9.58}$$

Here,
$$\lambda_1 = \frac{1}{2}[\sqrt{1 + \alpha_1^2} - \sqrt{1 + \alpha_2^2}] \tag{9.59}$$

$$\lambda_2 = \frac{1}{2}[\sqrt{1 + \alpha_1^2} + \sqrt{1 + \alpha_2^2}] \tag{9.60}$$

where,
$$\alpha_1 = \frac{b_1}{d} \quad \text{and} \quad \alpha_2 = \frac{b_2}{d}$$

(*iii*) In the case of a depressed floor [Fig. 9.22 (*iv*)],

$$\phi_{D'} = \phi_D - \frac{2}{3}(\phi_E - \phi_D) + \frac{3}{\alpha^2} \tag{9.61}$$

$$\phi_{D_1'} = 100 - \phi_{D'} \tag{9.62}$$

where, ϕ_D and ϕ_E are given by Eqs. (9.52) and (9.51), respectively, and $\alpha = \dfrac{b}{d}$.

The above solutions have also been presented in the form of curves of Fig. 9.23 which too can be used for determining the pressure head ratio ϕ at the key points. One can directly obtain, from Fig. 9.23, the values of ϕ_C, and ϕ_D (for $b_1/b > 0.5$) for known values of α (= b/d) and b_1/b of intermediate pile [Fig. 9.22 (*iii*)]. The value of ϕ_E for given values of α (=b/d) and b_1/b of intermediate pile is obtained by subtracting the value of ϕ_C (for $1 - b_1/b$ and given α) from 100. For example, ϕ_E [for $b_1/b = 0.4$ and $\alpha = 4$] = $100 - \phi_C$ (for $b_1/b = 0.6$ and $\alpha = 4$) = 100 – 29.1 = 70.9. Likewise, to obtain ϕ_D (for $b_1/b < 0.5$ and given α), subtract ϕ_D (for $1 - b_1/b$ and given α) from 100. For example, ϕ_D [for $b_1/b = 0.4$ and $\alpha = 4$] = $100 - \phi_D$ (for $b_1/b = 0.6$ and $\alpha = 4$) = 100 – 44.8 = 55.2.

Similarly, while ϕ_E and ϕ_D for the sheet pile at the downstream end (Fig. 9.22 (*ii*) can be read from Fig. 9.23, the values of ϕ_C and ϕ_D for the sheet pile at the upstream end (Fig. 9.22 (*i*)) are obtained as follows:

$$\phi_{C_1} = 100 - \phi_E \quad \text{(for the sheet pile at the downstream end)}$$

$$\phi_{D_1} = 100 - \phi_D \quad \text{(for the sheet pile at the downstream end)}$$

And for the depressed floor (Fig. 9.22 (*iv*)),

$$\phi_{D_1'} = 100 - \phi_{D'}$$

The uplift pressures obtained for the simple forms from either the above-mentioned mathematical expressions or the curves of Fig. 9.23 are corrected for (*i*) the floor thickness, (*ii*) mutual interference of sheet piles, and (*iii*) the slope of the floor.

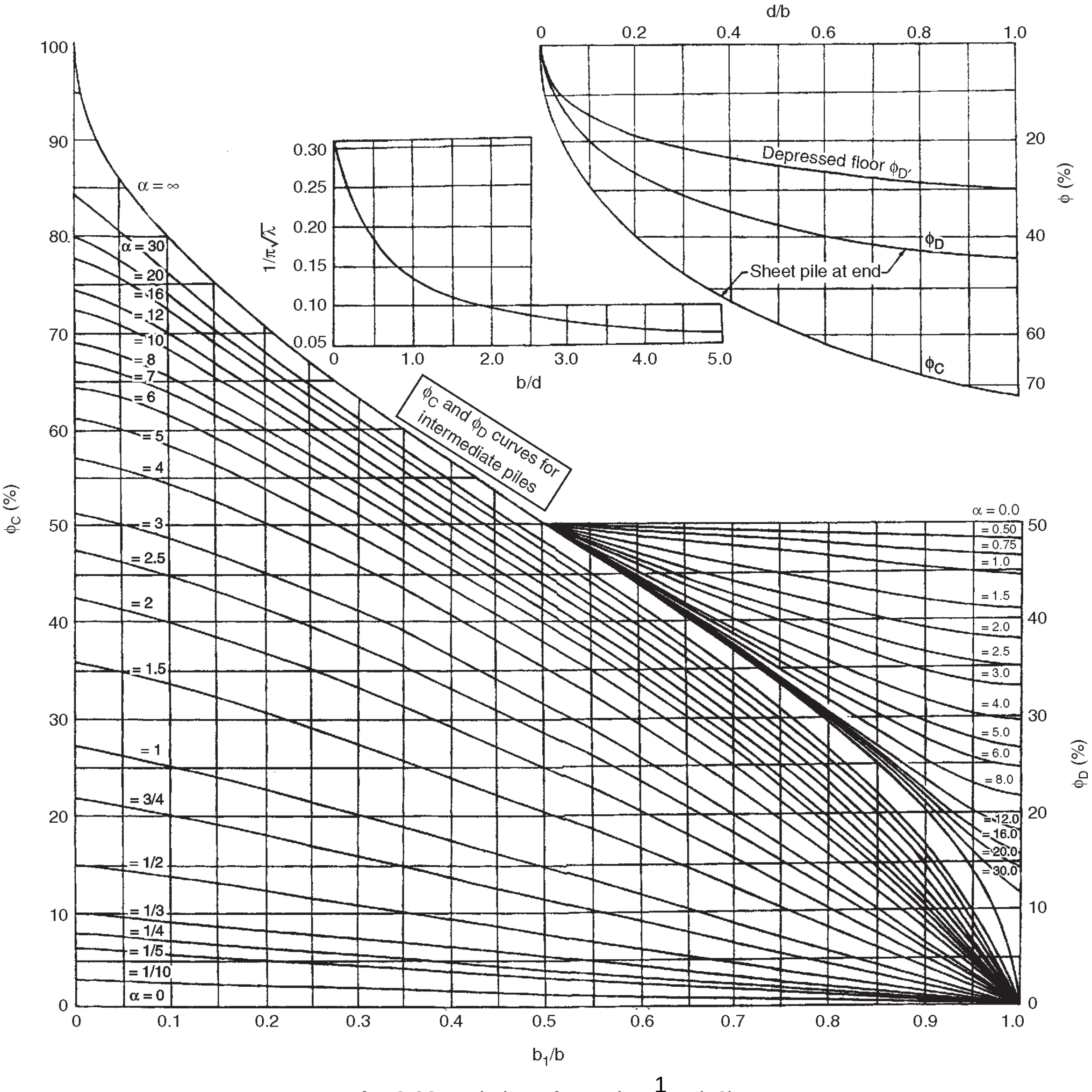

Fig. 9.23 Variation of ϕ and $\dfrac{1}{\pi\sqrt{\lambda}}$ (12)

The corrected values of the pressures are valid for actual profiles. The pressures at intermediate points between the adjacent key points are assumed to vary linearly. This assumption causes only negligible error.

Correction for Floor Thickness

The key points E (or E_1) and C (or C_1) correspond to the level at the top of the floor. The values of pressure at points E' (or E_1') and C' (or C_1') (Fig. 9.24) are interpolated assuming linear variation of pressure between the key points. Thus,

$$\phi_{E'} = \phi_E - \frac{\phi_E - \phi_D}{d + t}\, t \tag{9.63}$$

$$\phi_{C'} = \phi_C + \frac{\phi_D - \phi_C}{d + t}\, t \tag{9.64}$$

Here, t is the thickness of the floor.

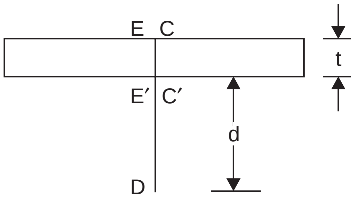

Fig. 9.24 Sketch for correction for floor thickness

Correction for Mutual Interference of Sheet Piles

Referring to Fig. 9.25, the amount of correction C (in per cent) for interference of sheet pile B for the key point C_{1a} of pile A is given as

$$C_i = 19\sqrt{d_1/b'}\ \frac{d_1 + d}{b} \tag{9.65}$$

where, b' is the distance between the two pile lines, d_1 the depth of the interfering pile (*i.e.,* the pile whose influence is to be determined on the neighbouring pile of depth d) measured below the level at which the interference is desired, and b is the total floor length.

The correction C_i is additive for upstream (in relation to the interfering pile) points and negative for points downstream of the interfering pile. Equation (9.65) is not applicable for determining the effect of an outer pile on an intermediate pile when the latter is equal to or smaller than the former and is at a distance less than or equal to twice the length of the outer pile. The correction for interference of a pile is calculated only for the key points of the adjacent pile towards the interfering pile. In Fig. 9.25, pile B interferes with the downstream face (*i.e.,* C_{1a}) of pile A and the upstream face (*i.e.,* E_c) of pile C.

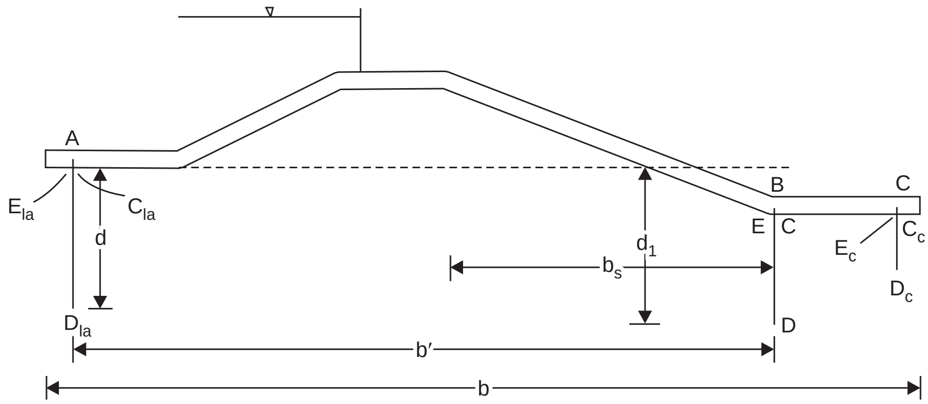

Fig. 9.25 Sketch for mutual interference of sheet piles and for the slope of the floor

Correction for the Slope of the Floor

The correction for the slope of the floor is applied to the pressures of the key point (on the side of the sloping floor) of that pile line which is fixed at either the beginning or the end of the slope. The correction is additive for positive slope (*i.e.,* level of the floor is decreasing in the direction of flow) and is negative for the negative slope. In Fig. 9.25, the correction for slope is applicable only to the pressure at E of pile B and is positive. If b_s is the horizontal length of the sloping floor and b' is the distance between the two pile lines between which the sloping floor is located, then the amount of slope correction is equal to $C_s(b_s/b')$. The value of C_s depends on the slope of the sloping floor and is as given in Table 9.2.

Table 9.2 Values of C_s

Slope ($V : H$)	1 : 1	1 : 2	1 : 3	1 : 4	1 : 5	1 : 6	1 : 7	1 : 8
Correction C_s (% of pressure)	11.2	6.5	4.5	3.3	2.8	2.5	2.3	2.0

9.3.4. Method for Determination of Exit Gradient

For the simple profile of the downstream sheet pile, [Fig. 9.22 (*ii*)], the exit gradient G_E, as obtained by Khosla, *et al*. (12), is given as

$$G_E = \frac{H}{d} \frac{1}{\pi\sqrt{\lambda}} \tag{9.66}$$

Equation (9.66) gives G_E equal to infinity for no sheet pile at the downstream end of the floor (*i.e.,* $d = 0$). It is, therefore, necessary that a vertical cutoff (*i.e.,* sheet pile) is always provided at the downstream end of the floor. To prevent piping, the exit gradient must not be allowed to exceed the critical value of the exit gradient which depends on the type of soil. The value of the critical exit gradient for sand varies from 1/5 to 1/7. One can also obtain the values of $1/\pi\sqrt{\lambda}$ from Fig. 9.23 and thus obtain the exit gradient.

Example 9.2 Using Khosla's method, obtain the residual seepage pressures at the 'key' points for the weir profile shown in Fig. 9.26. Also calculate the value of the exit gradient. Consider the case of no flow at pond level.

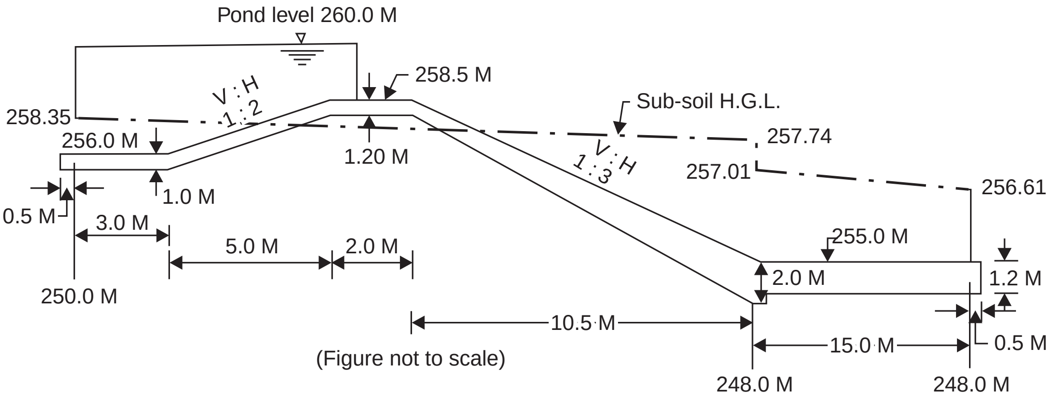

Fig. 9.26 Figure for Example 9.2

Solution: **Upstream Pile :**

Depth of pile $\quad d = 256.00 - 250.00 = 6.0$ m

Total floor length $\quad b = 36.5$ m

Using Eqs. (9.51) to (9.55),

$$\alpha = \frac{b}{d} = \frac{36.5}{6.0} = 6.083$$

$$\lambda = \frac{1}{2}[1 + \sqrt{1+\alpha^2}] = 3.083$$

$$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-2}{\lambda}\right) = 0.386$$

$$\therefore \quad \phi_{C_1} = 1 - 0.386 = 0.614 = 61.4\%$$

$$\phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-1}{\lambda}\right) = 0.264$$

$$\therefore \quad \phi_{D_1} = 1 - 0.264 = 0.736 = 73.6\%$$

Using Fig. 9.23 with $\frac{1}{\alpha} = \frac{d}{b} = \frac{6.0}{36.5} = 0.164$

$$\phi_{C_1} = 100 - \phi_E = 100 - 36 = 64\%$$

and

$$\phi_{D_1} = 100 - \phi_D = 100 - 25 = 75\%$$

It may be noted that there is slight difference between the values calculated from mathematical expressions and those read from Fig. 9.23. Corrections have been calculated and applied to the values obtained from mathematical expressions.

Thickness correction for $\phi_{C_1} = \frac{73.6 - 61.4}{6.0} \times 1.0 = 2.03\%\ (+)$

Correction for interference of intermediate pile on ϕ_{C_1} of upstream pile

$$= 19\sqrt{\frac{(255-248)}{20.5}}\ \frac{[(255-248)+5]}{36.5} = 3.65\%\ (+)$$

$\therefore \quad \phi_{C_1}$ (corrected) = 61.4 + 2.03 + 3.65 = 67.08%

Intermediate Pile:

Using Eqs. (9.56) to (9.60),

$$\alpha_1 = \frac{b_1}{d} = \frac{21.0}{7.0} = 3.0$$

$$\alpha_2 = \frac{b_2}{d} = \frac{15.5}{7} = 2.2143$$

$$\lambda_2 = \frac{1}{2}\left[\sqrt{1+\alpha_1^{\,2}} + \sqrt{1+\alpha_2^{\,2}}\right] = 2.796$$

$$\lambda_1 = \frac{1}{2}\left[\sqrt{1+\alpha_1^{\,2}} - \sqrt{1+\alpha_2^{\,2}}\right] = 0.366$$

$$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 - 1}{\lambda_2}\right) = \frac{1}{\pi}\cos^{-1}\left(\frac{-0.634}{2.796}\right) = 0.5728 = 57.28\%$$

$$\phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1}{\lambda_2}\right) = \frac{1}{\pi}\cos^{-1}\left(\frac{0.366}{2.796}\right) = 0.4582 = 45.82\%$$

$$\phi_C = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 + 1}{\lambda_2}\right) = \frac{1}{\pi}\cos^{-1}\left(\frac{1.366}{2.796}\right) = 0.3375 = 33.75\%$$

Using Fig. 9.23 with

b = 36.5 m, b_1 = 21.0 m, d = 255 – 248 = 7.0 m, and b_2 = 15.5 m

$$b_1/b = \frac{21.0}{36.5} = 0.575$$

$\therefore$ $$1 - (b_1/b) = 0.425$$

and $$\alpha = \frac{36.5}{7.0} = 5.21$$

$\therefore$ $$\phi_C = 34\%$$

$$\phi_E = 100 - 42 = 58\%$$

and $$\phi_D = 46\%$$

The values of ϕ obtained from Eqs. (9.56) to (9.60) have been corrected as follows:

Thickness correction for $\phi_E = \dfrac{57.28 - 45.82}{7} \times 2$ = 3.28% (–)

Slope correction for $\phi_E = C_s\,(b_s/b') = 4.5 \times \left(\dfrac{10.5}{20.5}\right)$

= 2.31% (+)

Correction for interference of the upstream pile (for ϕ_E)

$$= 19\sqrt{\frac{(253-250)}{20.5}} \times \frac{[(253-250)+5]}{36.5}$$

= 1.59% (–)

ϕ_E(corrected) = 57.28 – 3.28 + 2.31 – 1.59 = 54.72%

Thickness correction for $\phi_C = \frac{45.82 - 33.75}{7} \times 2 = 3.45\%\ (+)$

Correction for interference of the downstream pile (for ϕ_C)

$$= 19\sqrt{\frac{(253-248)}{15}} \times \frac{(253-248)+5}{36.5} = 3.01\%\ (+)$$

$\therefore$ ϕ_C (corrected) = 33.75 + 3.45 + 3.01 = 40.21%

Downstream pile:

$$d = 255 - 248 = 7.0 \text{ m and } b = 36.5 \text{ m}$$

$$\therefore \qquad \alpha = \frac{36.5}{7.0} = 5.21$$

Using Eqs. (9.55), (9.51) and (9.52),

$$\lambda = \frac{1}{2}\left[1+\sqrt{1+\alpha^2}\right] = \frac{1}{2}\left[1+\sqrt{1+(5.21)^2}\right] = 3.15$$

$$\therefore \qquad \phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-2}{\lambda}\right) = \frac{1}{\pi}\cos^{-1}\left(\frac{1.15}{3.15}\right) = 0.3810 = 38.10\%$$

$$\therefore \qquad \phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-1}{\lambda}\right) = \frac{1}{\pi}\cos^{-1}\left(\frac{2.15}{3.15}\right) = 0.2609 = 26.09\%$$

Using Fig. 9.23 with $\frac{1}{\alpha} = \frac{d}{b} = \frac{7.0}{36.5} = 0.192$,

$$\phi_E = 38\%$$

and $\qquad \phi_D = 26\%$

Using the values of ϕ_D and ϕ_E obtained from Eqs. (9.51) and (9.52),

Thickness correction for $\phi_E = \frac{38.1 - 26.09}{7} \times 1.2 = 2.06\%\ (-)$

Correction for interference of intermediate pile (for ϕ_E)

$$= 19\sqrt{\frac{(253.8-248)}{15}} \times \frac{(5.8+5.8)}{36.5}$$

$$= 3.75\%\ (-)$$

ϕ_E (corrected) = 38.1 – 2.06 – 3.75 = 32.29%

Based on the above calculations, subsoil hydraulic gradient line has been plotted in Fig. 9.26

Exit Gradient:

$$\alpha = \frac{36.5}{7.0} = 5.21\ ;\ H = 260 - 255 = 5.0 \text{ m}$$

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = 3.15$$

$$G_E = (H/d)\frac{1}{\pi\sqrt{\lambda}} = \frac{5.0}{7.0} \times \frac{1}{3.14\sqrt{3.15}} = 0.128 = 1 \text{ in } 7.8$$

9.4. UPLIFT FORCE ON THE FLOOR OF CANAL STRUCTURE

Seepage below the canal structure causes uplift force on the floor of the structure. This force is likely to be maximum when the water is ponded up to the highest level on the upstream side with no discharge on the downstream side. The subsoil hydraulic gradient line for this case has been shown in Fig. 9.27 and the net uplift pressure head at any section on the downstream

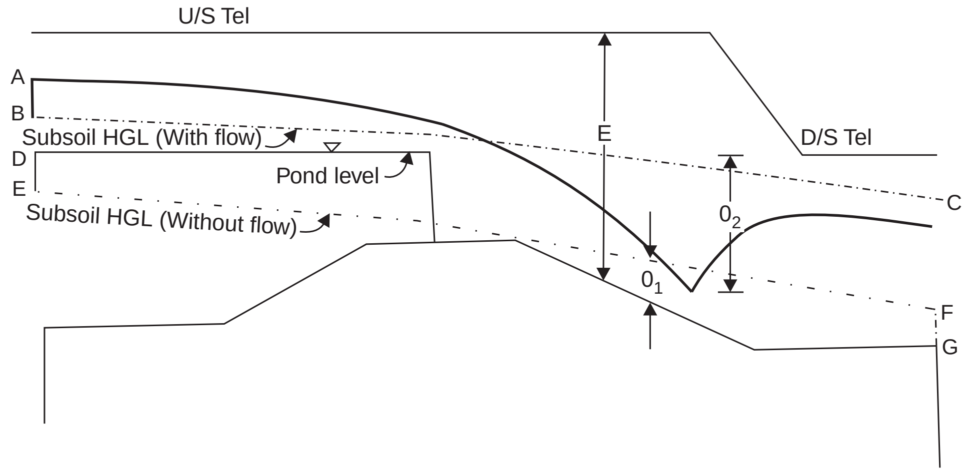

Fig. 9.27 Determination of uplift forces

of the barrier is the difference between the hydraulic gradient line and the floor level. This has been shown as o_1 in the figure. When there is flow, hydraulic jump forms and corresponding subsoil hydraulic gradient line for this case is also shown in Fig. 9.27. The net uplift pressure head for this condition will be obtained by measuring the distance of the water surface from the subsoil hydraulic gradient line at the desired section. This has been shown as o_2 in the figure. For most of the downstream sections, o_2 will be smaller than o_1. However, in the vicinity of the jump trough, o_2 may be greater than o_1. The floor thickness on the downstream side should, obviously, be based on the larger of the two values of o_1 and o_2.

For determining the uplift pressures at any section upstream of the jump trough the water surface profile needs to be determined. For this purpose, measure the difference between the levels of the total energy line and the floor at the section. This difference E is the value of specific energy at that section. Using Montague's curves (Fig. 9.28) or the specific energy equation, $E = h + (q^2/2gh^2)$, one can determine the supercritical depth of flow h for known E and q. In this way, one can obtain the water surface profile upstream of the jump. The jump profile can be obtained as explained in Sec. 9.2.4. The downstream sub-critical depth can also be obtained by solving the specific energy equation or using Montague's curves.

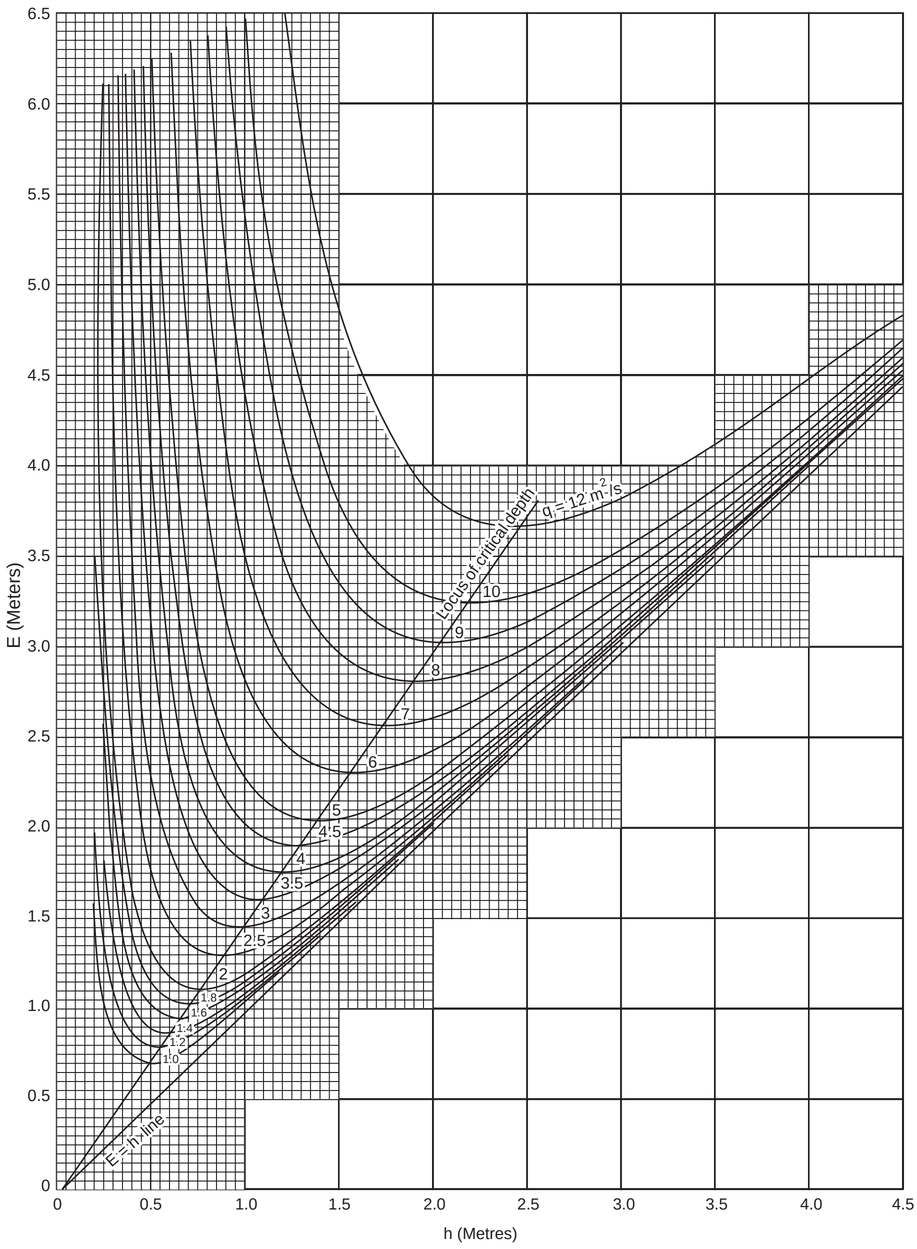

Fig. 9.28 Montague's curves

EXERCISES

9.1 How does seepage endanger safety of a structure on permeable foundation ?

9.2 Differentiate between Bligh's creep theory and Khosla's method for the analysis of seepage below hydraulic structures.

9.3 Sketch the hydraulic gradient line for the weir profile, shown in Fig. 9.29, considering the case of no flow at pond level. Slope correction for the slope (2 : 1) is 6.5 per cent. Also compute the value of the exit gradient.

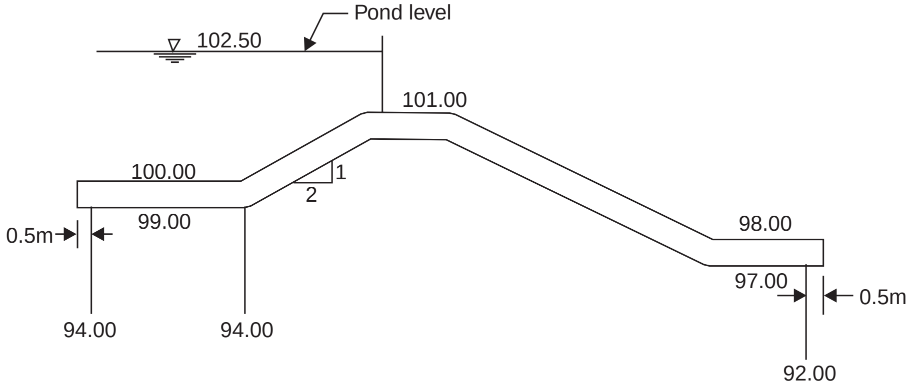

Fig. 9.29 (Exercise 9.3)

9.4 Using Khosla's method, check the safety of the weir profile, shown in Fig. 9.30, against piping and uplift (at point A). Safe exit gradient may be assumed as 1 in 5.

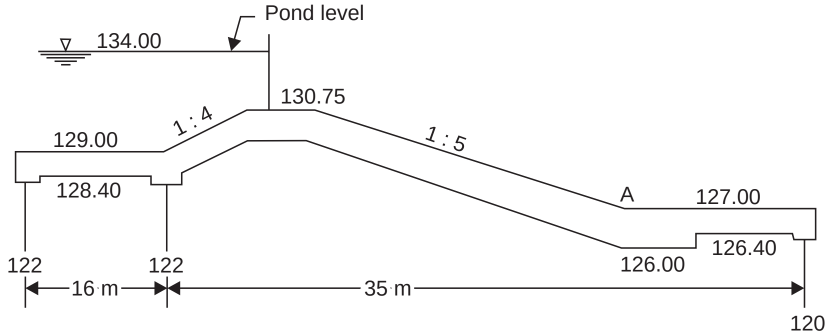

Fig. 9.30 (Exercise 9.4)

9.5 For the diversion structure, shown in Fig. 9.31, it was observed that wide cracks have developed immediately, downstream of the intermediate sheet pile, allowing the entire seepage water to escape through the cracks. Check the safety of the cracked structure, against uplift at B and also piping. The safe exit gradient is 0.20, correction for slope A is 4.5 per cent, and for slope B is 1.0 per cent.

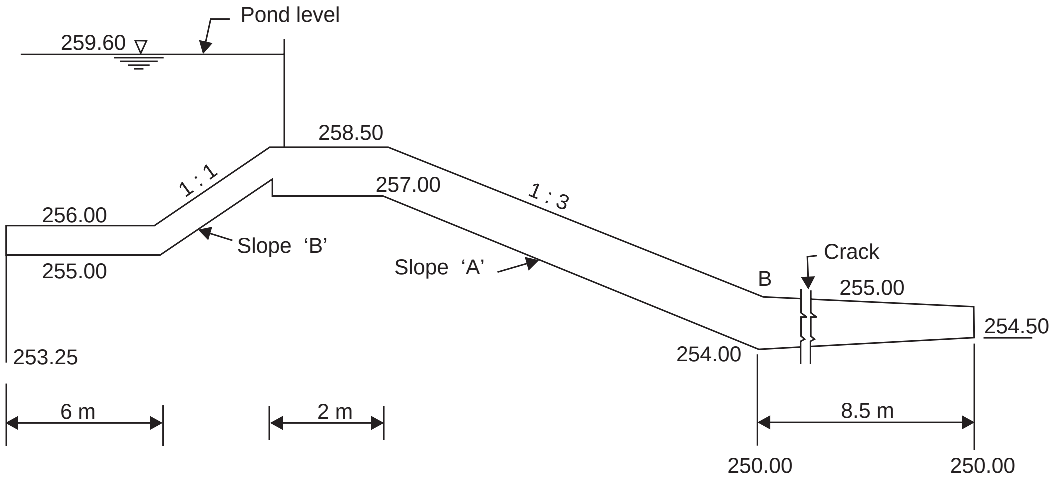

Fig. 9.31 (Exercise 9.5)

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10

CANAL REGULATION STRUCTURES

10.1. GENERAL

Canal regulation structures are hydraulic structures which are constructed to regulate the discharge, flow velocity, or supply level in an irrigation channel. These structures are necessary for efficient working as well as for the safety of an irrigation channel. Canal regulation structures can be classified as follows:

(*i*) *Canal fall:* The canal fall (or, simply, the 'fall' or 'drop') regulates the supply level in a canal by negotiating the change in its bed elevation necessitated by the difference in ground slope and canal slope.

(*ii*) *Distributary head regulator:* This controls the supply to an offtaking channel from the parent channel.

(*iii*) *Cross regulator:* This structure controls the water level of a channel and the discharge downstream of another hydraulic structure.

(*iv*) *Canal escape:* Canal escape disposes of extra supplies when the safety of a canal is endangered due to heavy rains or closure of outlets by farmers.

10.2. CANAL FALL

A canal fall is a hydraulic structure constructed across a canal to lower its water level. This is achieved by negotiating the change in bed elevation of the canal necessitated by the difference in ground slope and canal slope. The necessity of a fall arises because the available ground slope usually exceeds the designed bed slope of a canal. Thus, an irrigation channel which is in cutting in its head reach soon meets a condition when it has to be entirely in filling. An irrigation channel in embankment has the disadvantages of: (*i*) higher construction and maintenance cost, (*ii*) higher seepage and percolation losses, (*iii*) adjacent area being flooded due to any possible breach in the embankment, and (*iv*) difficulties in irrigation operations. Hence, an irrigation channel should not be located on high embankments. Falls are, therefore, introduced at appropriate places to lower the supply level of an irrigation channel. The canal water immediately downstream of the fall structure possesses excessive kinetic energy which, if not dissipated, may scour the bed and banks of the canal downstream of the fall. This would also endanger the safety of the fall structure. Therefore, a canal fall is always provided with measures to dissipate surplus energy which, obviously, is the consequence of constructing the fall.

The location of a fall is primarily influenced by the topography of the area and the desirability of combining a fall with other masonry structures such as bridges, regulators, and so on. In case of main canals, economy in the cost of excavation is to be considered. Besides, the relative economy of providing a large number of smaller falls (achieving balanced earth work and ease in construction) compared to that of a smaller number of larger falls (resulting in reduced construction cost and increased power production) is also worked out. In case of channels which irrigate the command area directly, a fall should be provided before the bed of the channel comes into filling. The full supply level of a channel can be kept below the ground level for a distance of up to about 500 metres downstream of the fall as the command area in this reach can be irrigated by the channels offtaking from upstream of the fall.

10.3. HISTORICAL DEVELOPMENT OF FALLS

There was no theory or established practice for the design and construction of falls in the nineteenth century. Falls were usually avoided by providing sinuous curves in the canal alignment. This alternative increased the length of the canal. Obviously, this approach was uneconomical and resulted in an inefficient irrigation system.

The ogee fall (Fig. 10.1) was first constructed by Cautley on the Upper Ganga canal with a view to providing a smooth transition between the upstream and the downstream bed levels so that flow disturbances could be reduced as far as practicable. The smooth transition of the ogee fall preserved the kinetic energy and also resulted in large drawdown which caused heavy erosion of bed and banks on the downstream as well as upstream of the fall. Later, this type of fall was converted into a vertical impact type so as to cause more energy dissipation downstream of the fall.

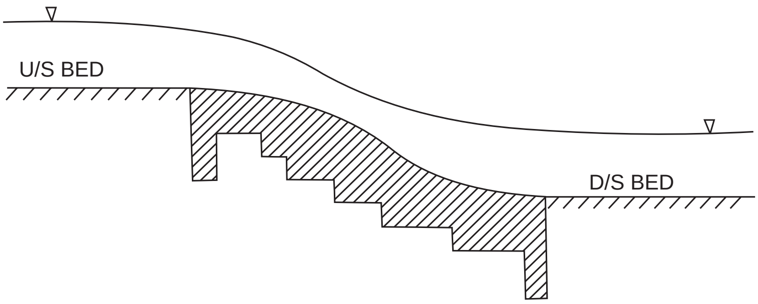


Fig. 10.1 Ogee fall

The falls on the Western Yamuna canal were in the form of 'rapids' (Fig. 10.2) which were gently sloping floors constructed at a slope of about 1 in 10 to 1 in 20.These rapids worked satisfactorily and permitted the movement of timber logs also. But, these structures were very expensive due to their longer length.

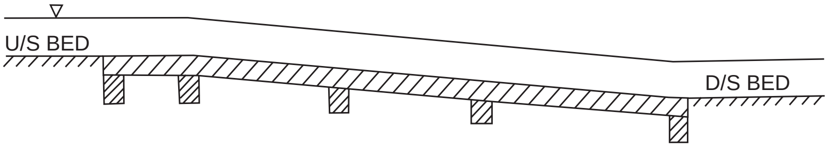
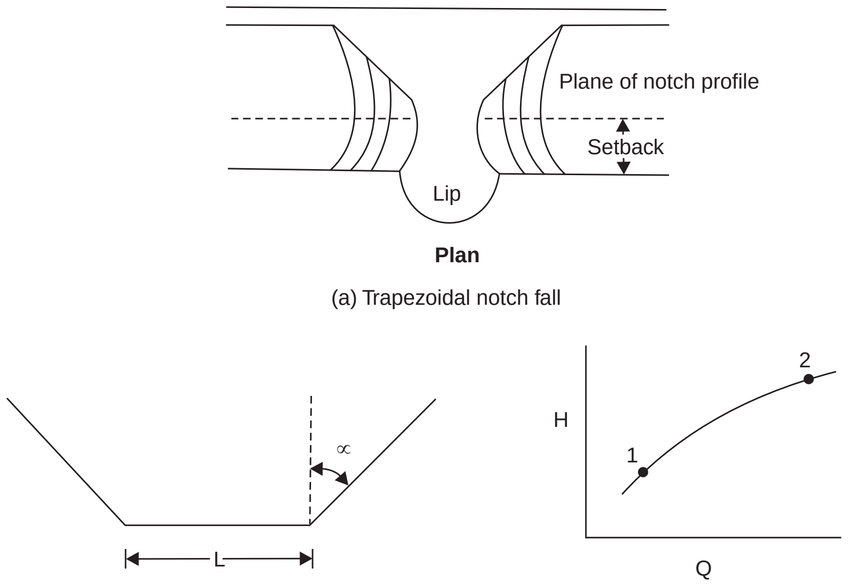


Fig. 10.2 Rapids

The realisation of the importance of a raised crest wall at the location of fall in reducing the drawdown resulted in the design of trapezoidal notch fall (Fig. 10.3). The fall structure had a number of trapezoidal notches in a high breast wall constructed across the channel having smooth entrance and a flat lip projecting downstream to spread out the falling jet. Such falls were very popular till simpler and economical falls were developed.



(b) Dimensions of trapezoidal notch fall

(c) Depth-discharge relation

Fig. 10.3 Trapezoidal notch fall

After World War I, the Sarda and glacis falls were developed in UP and Punjab, respectively. Of these two, the former proved to be more successful. The glacis fall gave trouble because of the resulting increase in discharge per unit width on account of fluming and, hence, the increased amount of kinetic energy to be dissipated downstream of the fall.

10.4. TYPES OF CANAL FALL

Canal falls are generally one of the following types:

(*i*) *Canal falls which nearly maintain the normal depth-discharge relationship:* Notch falls—trapezoidal or rectangular in shape—are of this type. The rectangular notch or low weir is not able to maintain accurately the normal depth-discharge relationship, but is economical and more suitable for discharge measurement. In a trapezoidal notch fall, a number of trapezoidal notches are made in a high breast wall across the channel. This arrangement provides an opening for flow right up to the bed level and thus eliminates silting in the channel upstream of the fall. The shape of the trapezoidal notch is decided on the basis of full supply and half supply conditions (1).

(*ii*) *Canal falls which nearly maintain a fixed water surface level in the upstream channel:* When either a subsidiary channel takes off upstream of a fall, or the fall is combined with a hydro-electric plant, it is desirable that the water surface in the parent channel be maintained at a fixed level as far as possible. Siphon falls and high-crested weir falls fulfil this requirement. However, siphon falls, although very efficient, are too expensive and, hence, used only as siphon spillways in dams and not used as canal falls. High-crested weir falls are usually not flumed so as to keep the discharge per metre length of fall, q, small. The smaller the discharge intensity, the smaller is the head required and, hence, the water level upstream of the fall can be maintained at a relatively fixed level to a considerable extent. A smaller value of q also makes energy dissipation easier. Such falls are, therefore, relatively cheaper.

Generally, the length of a fall is limited to the width of the channel but, can be increased by providing an expansion followed by contraction in the channel. However, this type of provision would increase the construction cost of the fall. Depending upon the type of the weir crest, whether broad or narrow, and the flow condition, whether free or submerged, one can use these falls as metering devices after suitable calibration.

A raised-crest fall with vertical impact was first used on the Sarda canal system in UP. The amount of the drop at any fall in this canal system does not exceed 1.80 m. A large number of smaller falls were necessitated on this system because of the stratum of pure sand lying below the thin stratum of clay sand. Therefore, the depth of excavation for channel construction had to be kept low so as to keep the seepage losses to the minimum.

(*iii*) *Canal falls which permit variation of water level upstream of the fall:* The necessity of such falls arise when the subsidiary channel upstream of the fall has to be fed with minimum supply level in the parent channel. Such falls consist of either trapezoidal or rectangular notches. But, trapezoidal notches are relatively expensive and render the operation of regulators, such as stop logs and vertical strips, more difficult. In general, falls of this category, therefore, consist of rectangular notches combined with one of the following three types of regulators:

(*a*) Sluice gate—Raising or lowering the gate helps in controlling the upstream level.

(*b*) Horizontal stop logs inserted into grooves—Their removal or insertion causes the required change in the upstream level.

(*c*) Vertical strips (or needles)—These change the effective width (*i.e.,* the width of opening) of the channel and do not cause silting.

(*iv*) In addition to the above main types of falls, there may be falls designed to meet specific requirement. Cylindrical falls (also known as well falls), pipe falls, chutes, etc. are falls designed for specific requirements. For example, a cylindrical fall (Fig. 10.4) is suitable for low discharge and high drop whereas chutes or rapids may have to be constructed if the canal is to carry timber logs as well.

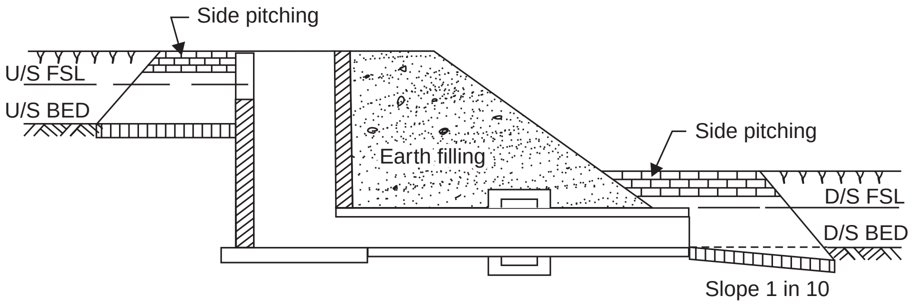


Fig. 10.4 Cylindrical fall

Canal falls can, alternatively, be divided on the basis of their capability to measure discharge. Accordingly, they may be either meter falls or non-meter falls.

10.5. CISTERN ELEMENT

As a result of the flow passing over a fall, the potential energy of the flow gets converted into kinetic energy. This excess kinetic energy, if not dissipated properly, will result in undesirable scour of the bed and sides of the downstream channel. Hence, provision of means to dissipate the surplus kinetic energy is essential in all types of canal falls and is provided in a portion of the canal fall known as cistern element.

The cistern element or, simply, cistern, Fig. 10.5, located at the downstream of the crest of a fall structure, forms an important part of any canal fall. The *cistern element* is defined as that portion of the fall structure in which the surplus energy of the water leaving the crest is dissipated and the subsequent turmoil stilled, before the water passes into the lower level channel (1). The cistern element includes glacis, if any, devices for ensuring the formation of hydraulic jump and deflecting the residual high velocity jets, roughening devices, and the pool of water in which hydraulic impact takes place. In other words, the cistern element includes the complete structure from the downstream end of the crest to the upstream end of the lower channel.

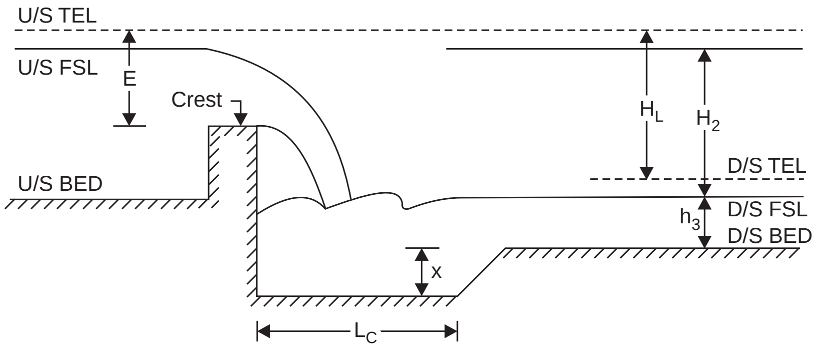


Fig. 10.5 Cistern element (vertical impact cistern)

Most of the cisterns employ hydraulic impact of the supercritical stream of the falling water with the subcritical stream of the lower channel for the dissipation of surplus energy. Depending upon the type of impact, the cisterns are divided into four categories discussed in the following paragraphs.

10.5.1. Vertical-impact Cisterns

In these cisterns (Fig. 10.5) there is an impact of a stream of water falling freely. The path of such a stream is, obviously, parabolic. This type of cistern is very efficient for the dissipation of surplus energy when the drop is sufficient so that the falling stream becomes almost vertical. The dimensions of the cistern should be such that it serves the purpose of stilling and combing out the residual eddies and disturbances. The length L_c and depth x of the cistern can be determined only by empirical expressions given by different investigators. Some of these are as follows (1):

Dyas' formula $$x = \sqrt{H_L h_3}^{1/3} \tag{10.1}$$

Glass' formula $$x + h_3 = 1.85\, E^{1/2} H_L^{1/3} \tag{10.2}$$

$$L_c = 5\,(x + h_3) \tag{10.3}$$

Etchevery formula $$L_c = 3\sqrt{EH_L} \tag{10.4}$$

$$x = \frac{L_c}{6} \tag{10.5}$$

UPIRI formula $$x = \frac{1}{4}(EH_L)^{2/3} \tag{10.6}$$

$$L_c = 5\sqrt{EH_L} \tag{10.7}$$

The symbols used in the above relations have been explained in Fig. 10.5 and their values are in metres.

In most of the vertical-impact cisterns, roughening is not provided. Instead, a short length of cistern is provided for stilling and such a provision has been found to be satisfactory. To prevent the falling nappe from adhering to the masonry face of the fall, aeration of the nappe is necessary and is provided by aeration pipes embedded in the wing walls just downstream of the crest. The exit from the cistern should be smooth so that the flow is streamlined before it enters the downstream channel.

Considerations such as combining the fall with a bridge may require contracting the width of the channel at the site of the fall. This increases the discharge per unit width and also the surplus energy to be dissipated. The design of an efficient cistern may then be difficult and expensive. In such situations, one may have to design some other type of cistern even if the available drop is, otherwise, large enough and suitable for a vertical-impact cistern.

10.5.2. Horizontal-impact Cisterns

In this type of cistern (Fig. 10.6), water, after passing over the crest, flows on a glacis whose reverse curve at the downstream end turns the inclined supercritical flow to horizontal supercritical flow before it strikes the subcritical flow of the downstream channel resulting in the formation of a hydraulic jump. However, the position of a hydraulic jump on a horizontal floor is very sensitive to the variations in the depth or velocity of the downstream flow. As a result, surging (movement of the jump) is very likely to occur on the horizontal floor of this type of cistern. In the absence of a perfect jump, the energy dissipation may not be proper. Therefore, it is usual to depress the cistern downstream of the jump location until the depth in the cistern increases by about 25 per cent of the tail-water depth (1). Although this provision is generally adequate for energy dissipation, it should, however, be noted that when the jump forms on sloping floor, the impact is no longer horizontal. Inglis has, instead, suggested a low baffle for holding the jump on a horizontal floor (1).

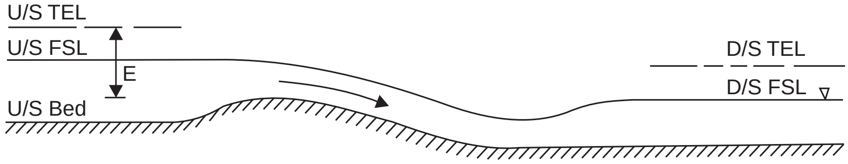


Fig. 10.6 Horizontal-impact cistern

For known discharge intensity q and the drop H_L, the total energy downstream of the jump E_2 can be calculated. The level of the cistern bed is then fixed at 1.25 E_2 below the downstream total energy line. Thus, the level of the bed of the cistern is independent of the level of the downstream channel. The length of the cistern is usually kept equal to 5 to 6 times E_2 to keep the jump within the cistern. Roughening devices, if required, may be provided starting from a section which is at a distance of half the height of the jump from the toe of the jump.

The design for such a cistern becomes more complicated when the supercritical jet is to be splayed. In such situations, the discharge intensity will be greater in the central part of the channel resulting in a 'bowed' jump. The expansion, therefore, must be very gradual. Although the horizontal-impact is an efficient energy dissipator (so long as the hydraulic jump continues to form), this type of provision requires expensive devices to hold the jump. Hence, provision of this type of cistern is usually made for dam spillways rather than for canal falls.

10.5.3. Inclined-impact Cisterns

For such cisterns, the glacis is carried straight down into the cistern and reliance is placed upon the effectiveness of the jump forming on the glacis for dissipation of surplus energy. However, the vertical component of the supercritical jet is not affected by the impact and, hence, energy dissipation is inefficient.

The dimensions of such cisterns are also decided in the same way as in the case of horizontal-impact cisterns. But, because of imperfect energy dissipation, the cistern has to accommodate the roughening devices and, hence, the cistern length for this class of cisterns is more than that for the preceding classes of cisterns.

10.5.4. No-impact Cisterns

Hydraulic impact is possible only when a supercritical flow meets a subcritical flow. In low falls and falls with large submergence, hydraulic impact may not be possible. Hence, other means of energy dissipation are adopted. A properly designed baffle wall along with suitable roughening devices are useful means of energy dissipation for such cases. The depth of no-impact cisterns cannot be calculated from theoretical considerations. However, it is useful to depress the cistern floor below the bed of the downstream channel as much as is economically feasible (1). This results in a larger cistern volume and, hence, permits retention of water for a longer period. Besides, it necessitates the construction of an upward slope (at the exit of the cistern) which helps in suppressing large-scale turbulence.

10.6. ROUGHENING MEASURES FOR ENERGY DISSIPATION

Hydraulic impact is the best means of energy dissipation in most hydraulic structures including canal falls. Of the three possible types of impact cisterns, *viz.,* vertical-impact, horizontal-impact, and inclined-impact cisterns, the vertical-impact cistern is the most effective while the inclined-impact cistern is the least effective. However, even in the case of an efficient vertical-impact cistern, the high turbulence persists downstream of the cistern and means for dissipating the residual energy are essential. In case of cisterns with no hydraulic impact, the roughening devices (provided on the cistern floor) are the only means to dissipate the surplus kinetic energy. Artificial roughness increases the actual wetted area which, in turn, increases the boundary friction. Besides, if correctly shaped and placed, the roughness increases the internal friction by increasing the interaction between high speed layers of the stream.

Provision of grids in a canal is impossible because of the presence of debris in the stream. As such, roughening has to be in the form of projections from the bed and sides of the channels. Projections are the most effective means of energy dissipation. The following roughening devices are generally used.

10.6.1. Friction Blocks

Rectangular concrete blocks properly anchored into the cistern floor and projecting up to one-fourth the full supply depth are simple, effective and commonly used devices for dissipating surplus kinetic energy in hydraulic structures. The spacing between the blocks in a row is kept about twice the height of the blocks. Depending upon the need, two or more staggered rows of these friction blocks may be provided (Fig. 10.7).

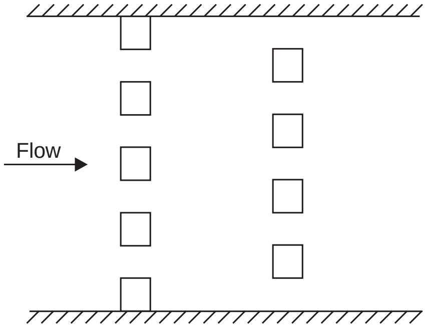


Fig. 10.7 Staggered friction blocks

The 'arrows' are specially shaped friction blocks (Fig. 10.8). The plan form of these arrows is approximately an equilateral triangle with rounded corners. The back face of arrows is vertical. The top of arrows is sloped from the front rounded corner to the back edge to give an upward deflection to stream filaments.

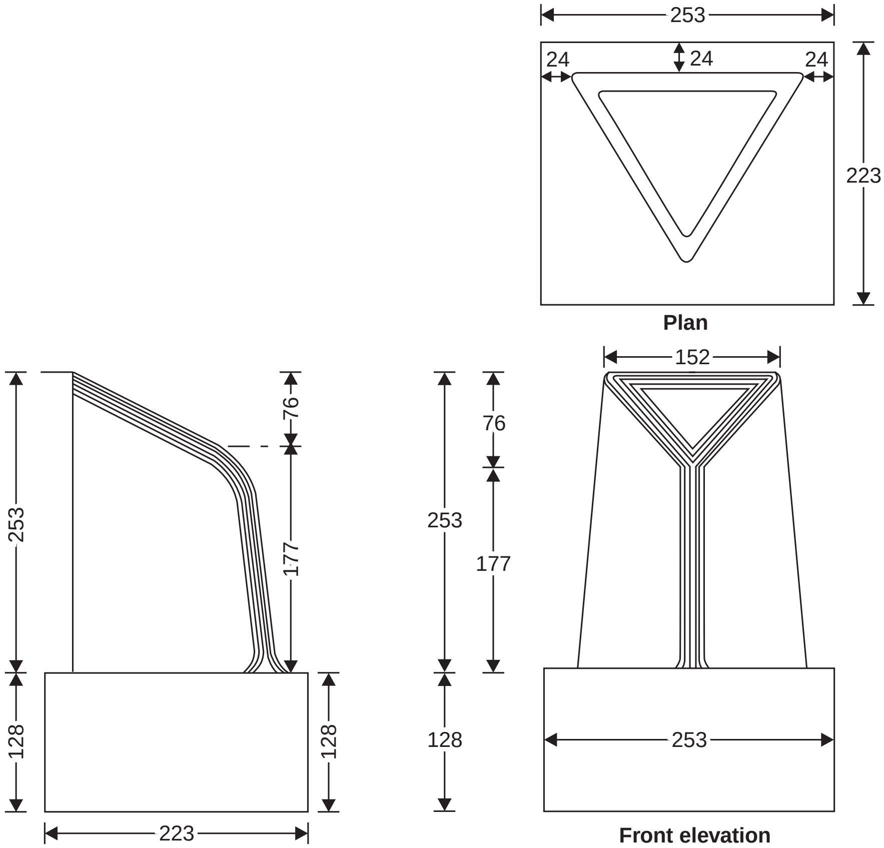


Fig. 10.8 Arrows

The length of cistern required to be roughened is equal to (1)

$$c\,\frac{h_3^{3/2} H_L^{1/2}}{D}$$

where, D is the depth of water in the cistern, h_3 the depth of flow in the downstream channel, H_L the drop height and c is a coefficient whose value depends on the type of impact as follows:

Vertical impact, $c = 1$

Horizontal impact, $c = 3$

Inclined impact with baffle, $c = 4$

Inclined impact without baffle, $c = 6$

No impact, $c = 8$ to 10

Where hydraulic impact occurs, the roughening should start downstream of the jump at a distance about half the height of the hydraulic jump. Downstream of the roughened length, a smooth cistern (without roughening devices) about half as long as the roughened cistern should be provided (1).

10.6.2. Ribbed Pitching

Projections on the sides of the channel for the purpose of dissipating surplus energy of the flow can be provided in the form of ribbed pitching which consists of bricks laid flat and on edge alternately (Fig. 10.9). Bricks laid on edge project into the stream and thus increase boundary friction and dissipate the surplus energy.

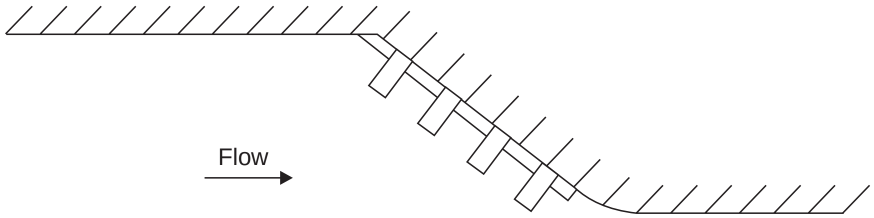


Fig. 10.9 Ribbed pitching

10.6.3. Provisions at the Downstream End of Cistern

If high velocity stream continues up to the end of the cistern, a baffle wall or a deflector [Fig. 10.10 (*a*)], or a dentated cill [Fig. 10.10 (*b*)] or a biff wall [Fig. 10.10 (*c*)] may be provided

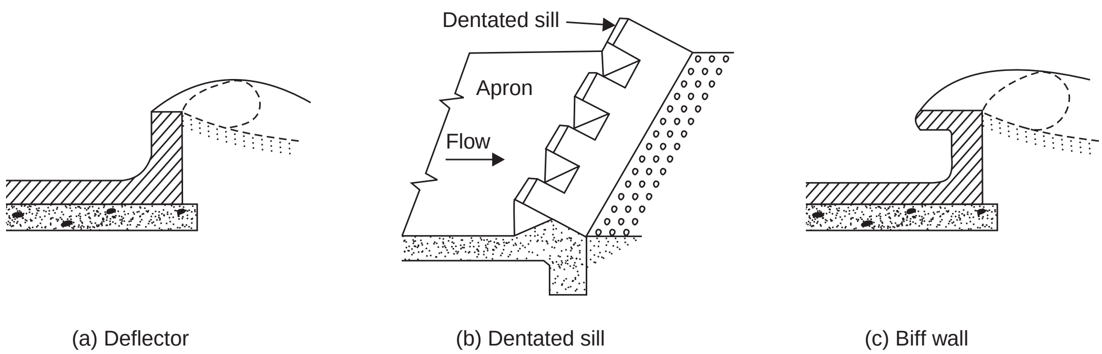


Fig. 10.10 Roughening devices at the downstream of cistern

at the downstream end of the cistern. The baffle wall provides a deep pool of water upstream of itself in the cistern. This pool of water is helpful in the dissipation of residual energy. Other devices (*i.e.,* deflector, dentated cill, biff wall) produce a reverse roller which results in a limited scour away from the toe and piles up material against the toe of the structure. A dentated cill, in addition, breaks up the stream jet.

10.7. TRAPEZOIDAL NOTCH FALL

A trapezoidal notch fall can be designed [*i.e.,* determine α and L, Fig. 10.3 (*b*)] to maintain the normal depth in the upstream channel for extreme values [say 1 and 2, Fig. 10.3 (*c*)] of a specified range of discharge, Q, using the following discharge equation for free flow condition:

$$Q = \frac{2}{3} C\sqrt{2g}\left(LH^{3/2} + \frac{4}{5} H^{5/2} \tan\alpha\right) \tag{10.8}$$

Here, H is the depth of water above the notch cill up to the normal water surface and is measured upstream of the fall where the streamlines are relatively straight. The value of the coefficient of discharge C may be taken as 0.78 for canal notches, and 0.70 for distributary notches (2). For two values of discharge, Q_1 and Q_2, and corresponding values of H_1 and H_2, one can obtain the following two equations for the determination of two unknowns L and α:

$$Q_1 = \frac{2}{3} C\sqrt{2g}\left(LH_1^{3/2} + \frac{4}{5} H_1^{5/2} \tan\alpha\right) \tag{10.9}$$

$$Q_2 = \frac{2}{3} C\sqrt{2g}\left(LH_2^{3/2} + \frac{4}{5} H_2^{5/2} \tan\alpha\right) \tag{10.10}$$

On solving Eqs. (10.9) and (10.10), one obtains

$$\tan\alpha = \frac{15}{8} \frac{Q_2 H_1^{3/2} - Q_1 H_2^{3/2}}{C\sqrt{2g}\, H_1^{3/2} H_2^{3/2} (H_2 - H_1)} \tag{10.11}$$

and

$$L = \frac{Q_1}{(2/3)\, C\sqrt{2g}\, H_1^{3/2}} - \frac{4}{5} H_1 \tan\alpha \tag{10.12}$$

Trapezoidal notch falls are designed for the full supply discharge and half of the full supply discharge (1).

Similarly, using the following equation for the submerged flow condition, the unknowns L and α can be determined for two sets of known values of Q, H and h_d (*i.e.,* the submergence head) for the two stages of the channel (2):

$$Q = \frac{2}{3} C\sqrt{2g}(H - h_d)^{3/2}\{(L + 2h_d \tan\alpha) + 0.8 \tan\alpha\,(H - h_d)\}$$
$$+ C\sqrt{2g}\,(H - h_d)^{1/2}(L + h_d \tan\alpha)\, h_d \tag{10.13}$$

The number of notches is so adjusted that the top width of the flow in the notch lies between 3/4th to full water depth above the cill of the notch. The minimum thickness of notch piers is half the depth and can be more if the piers have to support a heavy superstructure.

10.8. SARDA FALL

It is a raised-crest fall with a vertical-impact cistern. For discharges of less than 14 m^3/s, a rectangular crest with both faces vertical is adopted. If the canal discharge exceeds 14 m^3/s, a trapezoidal crest with sloping downstream and upstream faces is selected. The slopes of the downstream and upstream faces are 1 in 8 and 1 in 3, respectively. Both types of crests (Fig. 10.11) have a narrow and flat top with rounded corners. In Sarda fall, the length of the crest L is generally kept the same as the channel width. However, for future and other specific requirements, the crest length may exceed the bed width of the channel by an amount equal to the depth of flow in the channel.

For a rectangular-crest type Sarda fall, the discharge Q under free flow condition is expressed as (2)

$$Q = 0.415 \sqrt{2g}\ LH^{3/2}\left(\frac{H}{B}\right)^{1/6} \qquad (10.14)$$

Here, H is the head over the crest, and B is the width of crest which is related to the height of the crest above the downstream bed d as follows:

$$B = 0.55\ \sqrt{d} \qquad (10.15)$$

Obviously, $H + d = h_1 + D$

and, therefore, $d = h_1 + D - H$ (10.16)

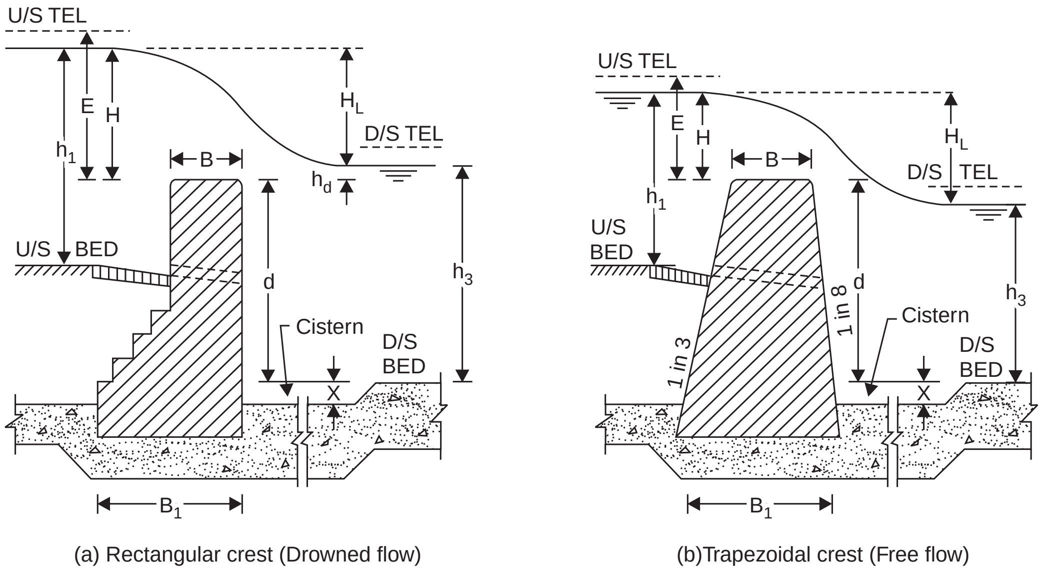


Fig. 10.11 Types of cross-sections of crest for Sarda fall

Here, D is the drop in the bed level, and h_1 is the upstream depth of flow. Equations (10.14) and (10.15) are solved by trial for obtaining the values of B and H for the rectangular-crest type Sarda fall.

For the trapezoidal-crest type Sarda fall, the discharge Q under free flow condition is expressed as (2)

$$Q = 0.45\sqrt{2g}\,LH^{3/2}\left(\frac{H}{B}\right)^{1/6} \tag{10.17}$$

Here, $B = 0.55\sqrt{H + d}$

$$\therefore \quad B = 0.55\sqrt{h_1 + D} \tag{10.18}$$

For known h_1 and D, one can determine B using Eq. (10.18). The head H is obtained from Eq. (10.17).

For submerged flow conditions, one should use the following discharge equation (2):

$$Q = C_d\sqrt{2g}L\left(\frac{2}{3}H_L^{3/2} + h_d\sqrt{H_L}\right) \tag{10.19}$$

Here, H_L is the difference between the upstream and the downstream water levels, and h_d is the submergence head as shown in Fig. 10.11 (*a*). The coefficient of discharge C_d is usually assigned an average value of 0.65. Equation (10.19) ignores the effect of the approach velocity. For given conditions, one can determine h_d from Eq. (10.19) and, hence, the height of the crest above the upstream bed which equals $[h_1 - (H_L + h_d)]$. Alternatively, one can use the following equation for the submerged flow (3):

$$Q = C_d\sqrt{2g}\,L\left[\frac{2}{3}\{(H_L + h_a)^{3/2} - h_a^{3/2}\} + h_d\{H_L + h_a\}^{1/2}\right] \tag{10.20}$$

Here, h_a is the approach velocity head.

The base width of a fall is decided by the requirement of concrete cover for stability and the slopes of the upstream and downstream faces of the fall.

A depressed cistern having suitable length and depression is provided immediately downstream of the crest and below the downstream bed level. The cistern and the downstream floor are usually lined with bricks laid on edge so that repair work would only involve brick surfacing and, hence, be relatively simple.

The depths of the cutoffs are as follows (3):

$$\text{Depth of upstream cutoff} = \left(\frac{h_1}{3} + 0.6\right)\text{m}$$

with a minimum of 0.8 m

$$\text{Depth of downstream cutoff} = \left(\frac{h_1}{2} + 0.6\right)\text{m}$$

with a minimum of 1.0 m

Here, h_1 is the upstream depth of flow in metres. The thickness of the cutoff is generally kept 40 cm.

The length of impervious floor is decided on the basis of either Bligh's theory or method proposed by Khosla *et al.* (Sec. 9.3.3). For all major works, the method of Khosla *et al.* should always be used. The most critical condition with respect to seepage would occur when water is up to the crest level with no overflow. The minimum floor length l_d, which must be provided downstream of a fall, is given by (2)

$$l_d = 2h_1 + H_L + 2.4$$

A thickness of 0.30 m is usually sufficient for the upstream floor. The thickness of the downstream floor is calculated from considerations of uplift pressure subject to a minimum of 0.45 to 0.60 m for larger falls and 0.3 to 0.45 m for smaller falls.

Brick pitching is provided on the channel bed immediately upstream of the fall structure. It is laid at a slope of 1 : 10 for a distance equal to the upstream depth of flow h_1. Few drain holes of diameter about 15 to 30 cm are provided in the raised-crest wall at the bed level to drain out the upstream bed when the channel is closed for maintenance or other purposes.

Radius of curvature of the upstream wing walls is around 5 to 6 times H. These walls subtend an angle of 60° at the centre and extend into earthen banks such that these are embedded in the channel banks by a minimum of 1 m. The wing walls should continue up to the end of the upstream impervious floor and for this purpose, if necessary, the walls may run along straight banks tangential to the wall segment.

Downstream of the fall structure, the wing walls are lowered down to the levels of the downstream wing walls through a series of steps. The downstream wing walls are kept vertical for a distance of about 5 to 8 times $\sqrt{EH_L}$ from the fall structure. These walls are then flared so that their slope changes from vertical to the side slope of the downstream channel. The wing walls are designed as the retaining walls to resist the earth pressures when the channel is not running.

Downstream of the warped wing walls, pitching protection is provided on the bed and sides of the channel. Pitching is either brick work or stones laid dry on a surface without the use of mortar. Pitching is, therefore, pervious. It is provided for a length equal to about three times the downstream depth of flow (3). Alternatively, Table 10.1 can be used for deciding the length of bed pitching. Bed pitching is kept horizontal up to the end of wing walls and, thereafter, it slopes at 1 in 10. The pitching on the side slopes of the channel is provided up to the line originating from the end of the bed pitching and inclined at 45° towards the upstream (Fig. 10.12). A toe wall at the junction of the bed and the side pitching is necessary for providing firm support to the side pitching.

Table 10.1 Length of bed pitching (2)

Head over crest, H in metres	*Total length of pitching in metres*
Less than 0.30	3
0.30 to 0.45	3 + 2D
0.45 to 0.60	4.5 + 2D
0.60 to 0.75	6.0 + 2D
0.75 to 0.90	9.0 + 2D
0.90 to 1.05	13.5 + 2D
1.05 to 1.20	18.0 + 2D
1.20 to 1.50	22.5 + 2D
D = Drop in bed level.	

Example 10.1 Design a 1.2 m Sarda fall for a channel carrying 25 m^3/s of water at a depth of flow equal to 1.8 m. The bed width of the channel is 20 m.

Solution: Since the discharge exceeds 14 m^3/s, the cross-section of the crest of Sarda fall is chosen as trapezoidal with sloping downstream (1 in 8) and upstream (1 in 3) faces.

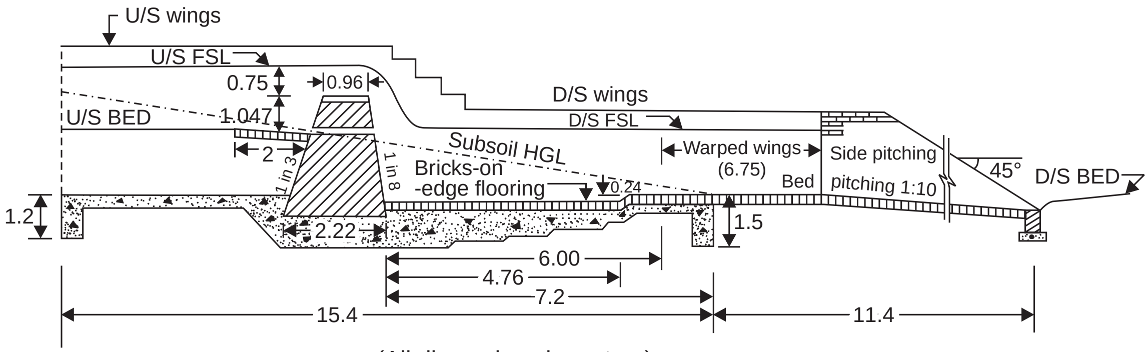


Fig. 10.12 Longitudinal section of Sarda fall (Example 10.1) (not to scale)

Dimensions of crest: Using Eq. (10.18),

Width of crest, $B = 0.55\sqrt{h_1 + D}$

$= 0.55\sqrt{1.8 + 1.2}$

$= 0.953$ m

$= 0.96$ m (say)

Length of crest = Bed width of channel = 20.0 m

From Eq. (10.17),

$$H = \left[\frac{QB^{1/6}}{0.45\sqrt{2gL}}\right]^{0.6}$$

$$= \left[\frac{25 \times (0.96)^{1/6}}{0.45 \times \sqrt{19.62} \times 20}\right]^{0.6} = 0.753 \text{ m}$$

Height of crest above the upstream bed $= h_1 - H$

$= 1.8 - 0.753$

$= 1.047$ m

Height of crest above the downstream bed, $d = h_1 + D - H$

$= 1.8 + 1.2 - 0.753$

$= 2.247$ m

The base of the fall should be at least 0.5 m below the downstream bed level. Accordingly,

Base width of fall $= (1/3)(d + 0.5) + B + (1/8)(d + 0.5)$

$= (11/24)(d + 0.5) + B$

$= (11/24)(2.247 + 0.5) + 0.96$

$= 2.220$ m

Cistern: Using Eq. (10.7),

Length of cistern, $L_c = 5\sqrt{EH_L}$

Assuming $E \cong H$, $L_c = 5\sqrt{0.753 \times 1.2}$

$$= 4.753 \text{ m}$$
$$= 4.76 \text{ m (say)}$$

From Eq. (10.6),

Depth of cistern, $x = (1/4)\,(EH_L)^{2/3}$

$$= (1/4)\,(0.753 \times 1.2)^{2/3}$$
$$= 0.234 \text{ m}$$
$$= 0.24 \text{ m (say)}$$

Upstream and downstream cutoffs:

Depth of the upstream cutoff $= \dfrac{h_1}{3} + 0.60$

$$= (1.8/3) + 0.60$$
$$= 1.2 \text{ m}$$

Depth of the downstream cutoff $= \dfrac{h_1}{2} + 0.60$

$$= (1.8/2) + 0.60$$
$$= 1.5 \text{ m}$$

Thickness of these cutoffs may be kept equal to 0.4 m.

Length of impervious floor: Assuming safe exit gradient to be equal to 1/5, one can write

$$G_E = \frac{1}{\pi\sqrt{\lambda}}\left(\frac{H}{d}\right)$$

Here, H = Head for no flow condition

= height of the crest above the downstream bed = 2.247 m

d = the depth of the downstream cutoff = 1.5 m

$$\therefore \qquad \frac{1}{\pi\sqrt{\lambda}} = G_E\,\frac{d}{H} = \frac{1}{5} \times \frac{1.5}{2.247} = 0.134$$

$$\therefore \qquad \lambda = 5.643$$

$$\alpha = \frac{b}{d_1} = 10.24$$

$$\therefore \qquad b = 10 \times 1.5$$

$\therefore$ Total floor length = 15.36 m $\cong$ 15.4 m (say)

Minimum floor length required on the downstream $= 2h_1 + H_L + 2.4$

$$= 2 \times 1.8 + 1.2 + 2.4 = 7.2 \text{ m}$$

So provide the downstream floor length equal to 7.2 m and the balance 8.2 m long impervious floor on the upstream side. Thickness of the concrete floor at various sections is decided as illustrated in Example 10.2.

Upstream protection:

Radius of curvature of the upstream wing walls = 5 to 6 times H

$$= 5 \times 0.753 \text{ to } 6 \times 0.753$$
$$= 3.765 \text{ m to } 4.518 \text{ m}$$
$$= 4.0 \text{ m (say)}$$

Brick pitching on the upstream bed is provided at a slope of 1 : 10 for a distance equal to the upstream depth of flow. Drain holes of 20 cm diameter are provided at an interval of 4 m.

Downstream protection: The downstream wing walls are to be kept vertical for a distance of about 5 to 8 $\sqrt{EH_L}$

$$= 5\sqrt{0.753 \times 1.2} \text{ to } 8\sqrt{0.753 \times 1.2}$$

$$= 4.75 \text{ m to } 7.60 \text{ m}$$

$$= 6.0 \text{ m (say)}$$

Thereafter, the wing walls should be warped from the vertical to the side slopes of the channel. The top of the wing walls is given an average splay of about 1 : 2.5 to 1 : 4. The difference in the surface widths of flow in the rectangular and trapezoidal section is 1.8 × 1.5 × 2 = 5.4 m. For providing a splay of 1 : 2.5 in the warped wings, the length of the warped wings along the channel axis is equal to (5.4/2) × 2.5 = 6.75 m.

In accordance with Table 10.1, total length of bed pitching on the downstream side is equal to 9 + 2D

$$= 9 + 2.4$$

$$= 11.4 \text{ m}$$

The bed pitching should be horizontal up to the end of the downstream wing walls and, thereafter, it should be laid at a slope of 1 in 10. A toe wall of thickness equal to 40 cm and depth equal to 1.0 m should be provided at the end of the bed pitching.

Side pitching is provided downstream of the wing walls up to the end of bed pitching. The side pitching may be suitably curtailed. Longitudinal section based on these computations has been shown in Fig. 10.12.

10.9. GLACIS FALL

In case of a glacis fall, the energy is dissipated through the hydraulic jump which forms at the toe of the glacis. Ideally, the profile of the glacis should be such that the maximum horizontal acceleration is imparted to the falling stream of water in a given length of the structure to ensure maximum dissipation of energy. It is obvious that in case of a free fall under gravity, there will be only vertical acceleration and no horizontal acceleration. Theoretically speaking, there will be some horizontal acceleration on a horizontal floor at the crest level because of the formation of an H_2 profile. This acceleration, however, would be relatively small. Hence, there should be a glacis profile, in between the horizontal and vertical, which would result in the maximum horizontal acceleration. Neglecting the acceleration on a horizontal floor at the crest level, Montague obtained the following profile which would yield maximum horizontal acceleration (1):

$$x = 2U\sqrt{y/g} + y \tag{10.21}$$

where, U is the initial horizontal velocity of water at the crest, and x and y are, respectively, the horizontal and vertical distances from the crest to any point along the glacis. The parabolic glacis profile [Eq. (10.21)] is, however, difficult and costly to construct. As such, a straight glacis with a slope of 2(H) : 1 (V) is commonly used.

A straight glacis fall may be provided with a baffle platform at the toe of the glacis and a baffle wall at the end of the platform in order to hold the jump at the toe of the glacis. The baffle platform is followed by a cistern downstream of the baffle wall. Such a fall was first developed by Inglis (1) and is called an Inglis fall; this too is obsolete now.

The present practice is to use a fall (Fig. 10.13) with straight glacis of slope 2 (*H*) : 1(*V*) at the toe of which is provided a cistern followed by friction blocks and other suitable measures to hold the jump at the toe of the glacis. Such straight glacis falls may or may not be flumed and can be either meter or non-meter falls. For known specific energy E at the downstream and the energy loss H_L in the jump, one can determine the discharge intensity q using Blench curves (Fig. 9.17). The crest length L is obtained by dividing the total canal discharge Q by the discharge intensity q. While finalising the crest length, the following limits with regard to permissible fluming should be kept in mind (3):

Drop	*Permissible minimum value of the ratio of crest length to the width of canal*
Up to 1 m	0.66
Between 1 and 3 m	0.75
Above 3 m	0.85

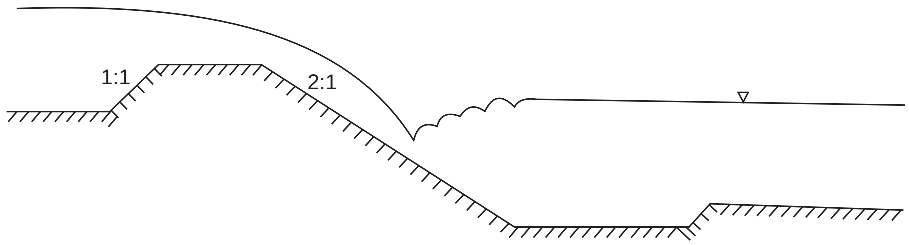


Fig. 10.13 Straight glacis fall

The level of the crest is H below the upstream total energy line. Here, H is the head over the crest up to the total energy line and is obtained by the discharge equation of a broad-crested weir,

$$Q = 1.71\, LH^{3/2} \tag{10.22}$$

Here, Q is expressed in m^3/s, and L and H are in metres. For meter falls, the width of the crest should at least be $2.5H$ so that the crest behaves as a broad-crested weir, and the coefficient of discharge is fairly constant at different discharges.

The upstream face of the crest is kept inclined at a slope of 1 : 1. The downstream face of the crest, *i.e.*, the glacis, is kept straight and inclined at a slope of 2 (*H*) : 1(*V*). The glacis is usually carried below the bed level of the downstream channel so that the toe of the glacis (or the level of the cistern) is 1.25 E_2 below the downstream total energy line. The depressed cistern increases the post-jump depth by about 25 per cent of the tail-water depth. This increased depth ensures formation of the jump at the toe of the glacis. The length of the cistern should not be less than 2 E_2. The cistern joins the downstream concrete floor through an upward slope of 5 (*H*) : 1 (*V*). The length of the downstream concrete floor (inclusive of the cistern) should be about five to six times the height of the jump. A concrete floor is also provided in the upstream canal immediately upstream of the crest for a length of about 2.0 m. In addition, a vertical cutoff is also provided at the downstream end of the concrete floor. The length and thickness of the concrete floor are checked against uplift force and exit gradient.

In case of flumed falls, suitable expansion has to be provided starting from the downstream end of the glacis. The expansion can be either a hyperbolic type or, simply, a 1 : 3

straight expansion. The vertical side walls at the end of the cistern are so warped that at the end of the expansion, the slope of the side walls is equal to the side slopes of the downstream channel. It should be noted here that while deciding the width of the flumed portion, the presence of the downstream expansion was completely ignored. For the prevalent subcritical flow conditions, the tail-water depth at the upstream end of the expansion will be smaller than the tail-water depth in the downstream channel after the expansion. To ensure formation of the jump at the toe of the glacis, friction blocks and end cill (both of around 0.5 m height) are additionally provided on the concrete floor. These provisions along with the depressed cistern ensure formation of the jump at the toe of the glacis. These design specifications have stood the test of time and are, accordingly, used in practice.

Downstream of the concrete floor, brick pitching is provided on the bed up to the end of the expansion. Brick pitching is also provided on the upstream canal bed just upstream of the concrete floor. Toe walls are suitably provided to support the brick pitching.

For meter falls, the side walls immediately upstream of the crest are made curved with radius of curvature of five to six times the drop and subtending an angle of 60° at the centre. These curved wing walls are joined to the upstream canal banks. For non-meter falls, however, the side walls may only be splayed at an angle of 45° from the upstream edge of the crest and carried into the banks for about 1.0 m.

Example 10.2 Design a straight glacis fall for a drop of 2.25 m in the water surface level of an irrigation channel carrying water at the rate of 60 m^3/s. The bed width and depth of flow in the channel are 30 m and 2.20 m, respectively.

Solution:

$$\text{Area of flow cross-section} = 30 \times 2.20 + \frac{1}{2}(2.20)^2 = 68.42 \text{ m}^2$$

$$\therefore \text{ Mean velocity of flow in the channel} = \frac{60}{68.42} = 0.877 \text{ m/s}$$

$$\therefore \quad \text{Velocity head} = (0.877)^2/(2 \times 9.81) = 0.039 \text{ m}$$

Post-jump specific energy in the downstream (lower level) channel,

$$E_2 = 2.20 + 0.039 = 2.239 \text{ m}$$

If the jump forms at the bed level of the downstream channel,

$$\text{pre-jump specific energy} = \text{specific energy in the upstream channel} + \text{drop}$$
$$= 2.239 + 2.25 = 4.489 \text{ m}$$

$\therefore$ Energy loss in jump, $H_L = 4.489 - 2.239 = 2.25$ m

Using Blench's curves (Fig. 9.17) for $H_L = 2.25$ m and $E_2 = 2.239$ m,

$$q = 2.85 \text{ m}^3\text{/s/m}$$

Thus,

$$\text{Length of crest} = \frac{60}{2.85} = 21.05 \text{ m}$$

This length is less than the permissible flumed width of channel which is 75% of 30 m, *i.e.,* 22.5 m.

Therefore, provide a crest of length = 22.5 m

so that $$q = 60/22.5 = 2.67 \text{ m}^3\text{/s/m}$$

From $$Q = 1.71\ LH^{3/2}$$

$$H = (Q/1.71\ L)^{2/3}$$

$$= \left(\frac{60}{1.71 \times 22.5}\right)^{2/3} = 1.345 \text{ m}$$

Hence, height of crest above the upstream channel bed

$$= 2.239 \text{ m} - 1.345 \text{ m} = 0.894 \text{ m}$$

For a meter fall, the width of crest should not be less than

$$2.5\ H \text{ i.e., } 2.5 \times 1.345 = 3.363 \text{ m}$$

Therefore, provide the crest width as 3.70 m.

The level of cistern is kept 1.25 E_2 below the level of the downstream total energy line, *i.e.,* 1.25 × 2.239 = 2.799 m. This means that the cistern is depressed below the downstream channel bed by an amount x = 2.799 – 2.239 = 0.56 m

(The value of E_2 corresponding to q = 2.67 m^3/s/m and H_L = 2.25 m is 2.16 m).

Thus, 1.25 E_2 = 2.7 m. The lowering of the cistern level below the downstream bed would, therefore, be 2.7 – 2.16 = 0.54 m which is less than 0.56 m. Hence, the cistern is depressed by 0.56 m.

Length of cistern $L_c = 2\ E_2 = 2 \times 2.239 = 4.478 \cong 4.48$ m

Length of the downstream concrete floor (inclusive of cistern length) should be sufficient to accommodate the jump within itself.

Post-jump depth = 2.20 m

The pre-jump depth for q = 2.67 m^3/s/m and specific energy = E_2 + drop = 2.16 + 2.25 = 4.41 m is obtained from Montague's curves (Fig. 9.27) as 0.30 m.

∴ Height of jump = 2.20 – 0.30 = 1.90 m

Hence,

Length of downstream floor = 5 × 1.90 = 9.50 m

On providing 2.4 m length of concrete floor upstream of the crest and the slopes of the upstream face of the crest and the glacis as 1 : 1 and 2 (H) : 1 (V), the total length of the impervious floor works out as

$$2.4 + 0.894 + 3.7 + 2\ (0.894 + 2.25 + 0.56) + 9.50 = 23.902 \text{ m}$$

Let the depth of the downstream cutoff be 1.5 m

$$\therefore \quad \alpha = \frac{b}{d} = \frac{23.902}{1.5} = 15.935$$

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = \frac{1+\sqrt{1+(15.935)^2}}{2} = 8.483$$

The exit gradient is calculated using the equation

$$G_E = \frac{H}{d} \times \frac{1}{\pi\sqrt{\lambda}}$$

The value of H for the condition of no flow with water on the upstream up to the crest level is (2.25 + 0.894) m, *i.e.,* 3.144 m. The corresponding value of H, when full supply discharge is flowing, works out to only 2.25 m. Therefore, the former condition is the most critical condition for the exit gradient. Accordingly,

$$G_E = \frac{(2.25 + 0.894)}{1.5} \times \frac{1}{3.14\sqrt{8.483}} = 0.229$$

which is less than 0.25 and may be considered satisfactory. Using Eqs. (9.51) and (9.52),

$$\phi_E = \frac{1}{\pi} \cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right) = \frac{1}{\pi} \cos^{-1}\left(\frac{8.483 - 2}{8.483}\right) = 0.223$$

$$\phi_D = \frac{1}{\pi} \cos^{-1}\left(\frac{\lambda - 1}{\lambda}\right) = \frac{1}{\pi} \cos^{-1}\left(\frac{8.483 - 1}{8.483}\right) = 0.156$$

Assuming a floor thickness of 0.5 m at the downstream end,

Correction for thickness (for ϕ_E) $= \dfrac{0.223 - 0.156}{1.5} \times 0.5 = 0.022$ (negative)

$\therefore$ Corrected value of $\phi_E = 0.223 - 0.022 = 0.201$

For no flow condition, the uplift pressure at the downstream end is equal to 0.201 × (2.25 + 0.894) = 0.632 m (w.r.t. the downstream bed). Ignoring the effect of the upstream cutoff (conservative approach), one can now draw the subsoil hydraulic gradient line by joining the uplift ordinate (0.632 m w.r.t. the downstream bed) at the downstream end of the impervious floor with the uplift ordinate (3.144 m w.r.t. the downstream bed, *i.e.,* 0.894 m above the upstream end of the concrete floor) at the upstream end of the impervious floor by a straight line.

Similarly, for full supply flow condition the uplift pressure at the downstream end would be equal to 0.201 × (2.25) = 0.45 m above the downstream full supply level. The subsoil hydraulic gradient line for this case too has been shown in Fig. 10.14. Calculations for jump profile are given below:

Pre-jump Froude number, $F_1 = \dfrac{2.67 / 0.32}{\sqrt{9.81 \times 0.32}} = 4.7$

Therefore, from Eq. (9.20), $\overline{X} = (5.08 \times 4.7 - 7.82) \times 0.32 = 5.14$ m

Values of distance from the trough of the jump, x, and the corresponding height of the water surface profile above the trough of the jump have been computed using Fig. 9.3.

$x(m)$	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0
$x/\overline{X}$	0.195	0.39	0.58	0.78	0.97	1.17	1.36	1.56	1.75	1.95
$y(m)$	0.395	0.677	0.917	1.13	1.34	1.48	1.62	1.75	1.80	1.83

The jump profile has been plotted in Fig. 10.14.

The thickness of floor at different locations can now be computed using Eq. (9.41) on the basis of the larger of the two uplift pressures and assuming a value of $G = 2.30$.

Radius of curvature of upstream wing walls = 5 × 25 = 11.25 m

Provide brick pitching for a length of about 5 m upstream of the concrete floor. Providing a splay of 1 in 3 for the straight downstream expansion, the length of the downstream expansion will be 11.25 m. Provide brick pitching from the downstream end of the concrete floor to the downstream end of the expansion.

The longitudinal section of the glacis fall based on these computations is shown in Fig. 10.14.

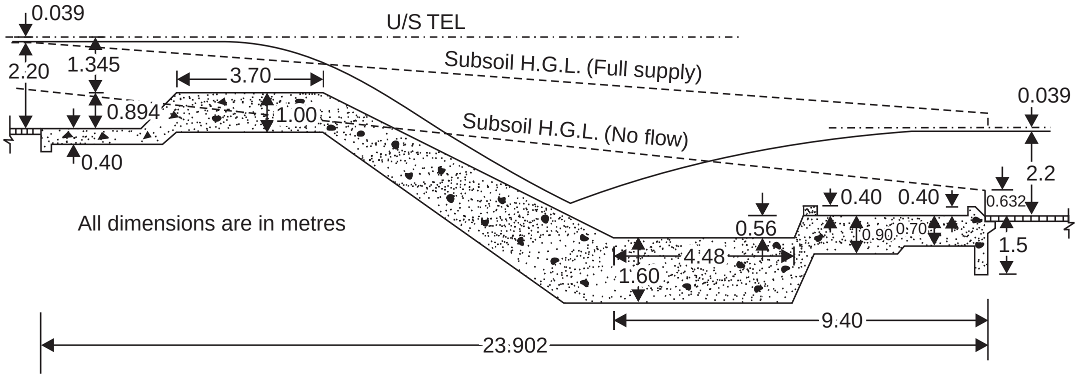


Fig. 10.14 Longitudinal section of glacis fall (Example 10.2) (not to scale)

10.10. DISTRIBUTARY HEAD REGULATOR

The distributary head regulator is constructed at the upstream end (*i.e.*, the head) of a channel where it takes off from the main canal or a branch canal or a major distributary. The distributary head regulator should be distinguished from the canal head regulator which is provided at the canal headworks where a canal takes its supplies from a river source. The distributary head regulator serves to (*i*) divert and regulate the supplies into the distributary from the parent channel, (*ii*) control silt entering the distributary from the parent channel, and (*iii*) measure the discharge entering the distributary.

For the purpose of regulating the supplies entering the offtaking channel from the parent channel, abutments on either side of the regulator crest are provided. Piers are placed along the regulator crest at regular intervals. These abutments and piers have grooves (at the crest section) for the purpose of placing planks or gates. The supplies into the offtaking channel are controlled by means of these planks or gates. The planks are used for small channels in which case manual handling is possible. The span of hand-operated gates is also limited to 6 to 8 m. Mechanically-operated gates can, however, be as wide as 20 m.

An offtaking channel tends to draw excessive quantity of sediment due to the combined effects of the following:

(*i*) Because of their smaller velocities, lower layers of water are more easily diverted into the offtaking channels in comparison to the upper layers of water.

(*ii*) Sediment concentration is generally much higher near the bed.

(*iii*) Sediment concentration near the banks is usually higher because of the tendency of the bottom water to move towards the banks due to difference in central and near-bank velocities of flow.

As such, if suitable steps are not taken to check the entry of excessive sediment into the offtaking channel, the offtaking channel will soon be silted up and would require repeated sediment removal.

Sediment entry into the offtaking channel can be controlled by causing the sediment to concentrate in the lower layers of water (*i.e.,* near the bed of the parent channel upstream of the offtaking point) and then letting only the upper layers of water enter the offtaking channel. Concentration of sediment in lower layers can be increased by providing smooth bed in the parent channel upstream of the offtaking point. The smooth channel bed reduces turbulence which keeps sediment particles in suspension. In addition, steps which accelerate the flow velocity near the banks would also be useful. It should also be noted that the alignment of the offtaking channel also affects the sediment withdrawal by the offtaking channel. Hence, the alignment of the offtaking distributary channel with respect to the parent channel needs careful consideration. The angle of offtake may be kept between 60° and 80° to prevent excessive sediment withdrawal by the offtaking channel. For all important works, the alignment of offtaking channels should be fixed on the basis of model studies.

For the purpose of regulating the discharge in the distributary, it is essential to measure the discharge for which one can use gauge-discharge relationship of the distributary. However, this relationship is likely to change with the change in the channel regime. Hence, it is advantageous to use head regulator as a metering structure too.

10.11. CROSS REGULATOR

A cross regulator is a structure constructed across a canal to regulate the water level in the canal upstream of itself and the discharge passing downstream of it for one or more of the following purposes (4):

(*i*) To feed offtaking canals located upstream of the cross regulator.

(*ii*) To help water escape from canals in conjunction with escapes.

(*iii*) To control water surface slopes in conjunction with falls for bringing the canal to regime slope and section.

(*iv*) To control discharge at an outfall of a canal into another canal or lake.

A cross regulator is generally provided downstream of an offtaking channel so that the water level upstream of the regulator can be raised, whenever necessary, to enable the offtaking channel draw its required supply even if the main channel is carrying low supply. The need of a cross regulator is essential for all irrigation systems which supply water to distributaries and field channels by rotation and, therefore, require to provide full supplies to the distributaries even if the parent channel is carrying low supplies.

Cross regulators may be combined with bridges and falls for economic and other special considerations.

10.12. DESIGN CRITERIA FOR DISTRIBUTARY HEAD REGULATOR AND CROSS REGULATOR

The effective waterway of a head regulator should not be less than 60 per cent of the width of the offtaking canal. It should be fixed such that the mean velocity of flow at full supply condition does not exceed 2.5 m/s. The overall waterway should at least be 70 per cent of the normal channel width (at mid-depth) of the offtaking channel at the downstream of the head regulator. For cross regulators, waterway should be decided so that the resulting afflux does not exceed 0.15 m.

The crest level of a head regulator should be such that the full supply discharge of the offtaking channel can be passed even when the parent channel is running with low supplies of the order of two-thirds of the fully supply discharge of the parent channel. It should be possible to maintain full supply level in the parent channel downstream of the offtake by means of a cross regulator. The water level at the location of offtake should be computed using back water computations. The level of the crest of the head regulator is obtained by subtracting the amount of head required from the computed water level at the offtake. In any case, the crest level should not be lower than the bed level of the offtaking channel. Usually, the crest level of the head regulator is 0.3 to 0.6 m higher than the crest level of the cross regulator. The amount of head over the crest of the head regulator, H is computed from the equation (3),

$$Q = CB_e H^{3/2} \tag{10.23}$$

where, Q is the full supply discharge of the offtaking channel, and B_e is the effective width of waterway and is given as

$$B_e = B_t - 2\,(NK_p + K_a)\,H \tag{10.24}$$

Here, B_t is the overall width of the waterway (*i.e.*, the length of the crest), N the number of piers, and K_p and K_a are contraction coefficients for piers and abutments, respectively. Values of K_p range from 0.005 to 0.02 while those of K_a range from 0.1 to 0.2.

C is a suitable discharge coefficient whose value can be taken as 1.84 for sharp-crested weirs (the crest width being less than 2/3 H) and 1.705 for broad-crested weirs (the crest width being greater than 2.5 H) for free flow conditions (3). If the flow is submerged, the values of C will have to be suitably modified depending upon the submergence ratio.

The crest of the cross regulator should be at least 0.15 m above the bed of the canal but should not be higher than 0.4 times the normal depth of the upstream canal (4). The crest width should be greater than 2/3 H and should be sufficient to accommodate the gate cill. The upstream and downstream slopes of the crest can be at slopes of 2 (H) : 1 (V). Impervious floor and cutoffs will be designed from considerations of hydraulic jump, uplift pressures, safe exit gradient, and scour depth. Similarly 'flexible' protection works at the upstream and downstream ends of the impervious floor will be provided in the form of block protection, inverted filters, and launching apron.

10.13. CONTROL OF SEDIMENT ENTRY INTO AN OFFTAKING CHANNEL

When water is withdrawn by an offtaking channel from the parent canal carrying sediment-laden water, it is essential that the offtaking channel also withdraw sediment in proportion to its water discharge. For achieving proportionate distribution of sediment between the offtaking channel and the parent canal, measures such as silt vanes, groyne walls, and skimming platform are constructed.

10.13.1. Silt Vanes

Silt vanes (also known as King's vanes) are thin, vertical, and curved walls made of plain or reinforced concrete. The recommended dimensions of silt vanes are shown in Table 10.2 and Fig. 10.15. The dimensions are intended only as rough guides and can be used for skew offtakes also. The height of the vanes may be about one-fourth to one-third of the depth of flow in the parent canal (5). The thickness of the vanes should be as small as possible. Faces of vanes should be smooth. The spacing between the vanes may be kept about 1.5 times the vane height.

Table 10.2 Dimensions (in metres) of silt vanes (5)

W	0.60	1.2	1.8	2.4	3.0	3.6	4.6	6.0	7.6	9.0	10.6	12.0
X	1.2	1.5	2.1	2.4	3.0	3.6	4.6	5.4	6.0	7.0	7.8	8.5
Y	0.6	1.2	1.5	1.8	2.4	2.7	3.0	4.0	5.2	6.0	6.6	7.6
Z	1.2	1.2	1.5	1.8	2.4	2.7	3.0	3.6	4.2	5.2	5.8	6.6
R	9.0	9.0	10.0	12.0	18.0	21.0	24.0	30.0	35.0	44.0	50.0	57.0

Note: See Fig. 10.15 for meaning of symbols used in this table.

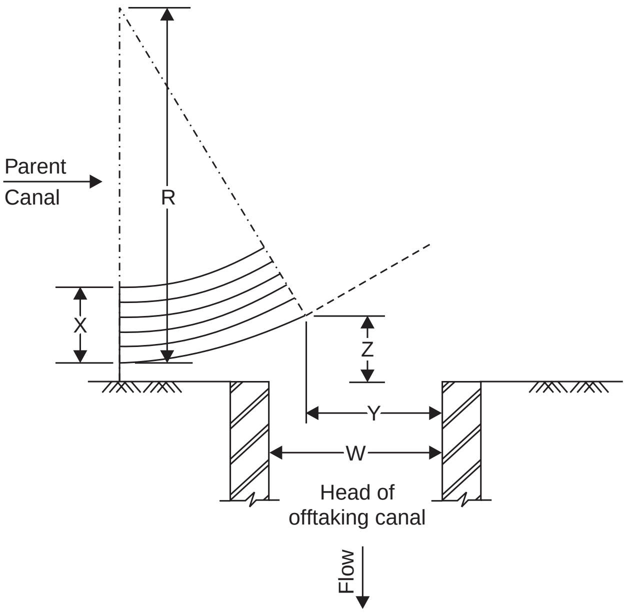


Fig. 10.15 Silt vanes (5)

Reverse vanes (Fig. 10.16) may have to be provided in cases in which the width of the parent canal is small and the sediment deflected towards the edge of the parent canal is likely to be deposited there due to low velocities.

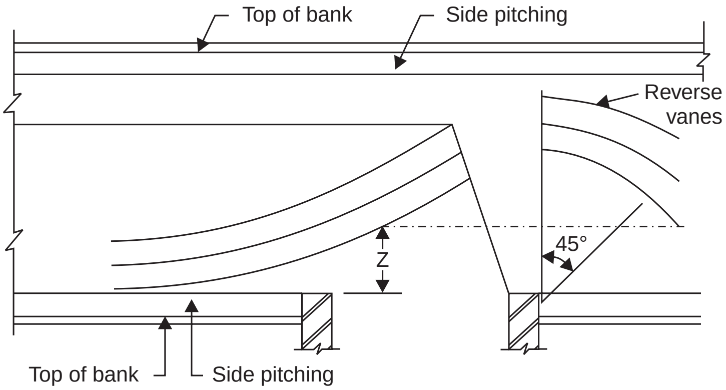


Fig. 10.16 Reverse vanes (5)

10.13.2. Groyne Wall (Curved Wing)

A curved vertical wall (also known as Gibb's groyne wall) extending from the downstream abutment of the offtaking channel into the parent channel (Fig. 10.17) causes the offtaking channel to draw its share of sediment load. The wall should extend at least up to 3/4 of the width of the offtake. It may, however, preferably extend up to the upstream abutment of the offtaking canal. The nose of the wall should be pointed and vertical, and thickness of the wall should increase gradually. The top of the wall is kept at least 30 cm above the full supply level of the parent channel (6). If the offtaking channel has a slope milder than that of the parent channel, the curved wall may not be useful to the desired extent, and has to be employed in conjunction with sediment vanes (6).

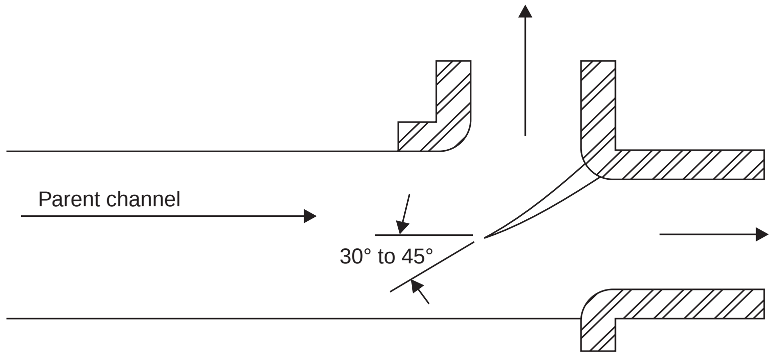


Fig. 10.17 Curved wing wall (6)

10.13.3. Skimming Platform

A skimming platform consists of an RC slab placed horizontally in the parent channel in front of the offtake (Fig. 10.18). It works on the principle of sediment excluder (Sec. 13.11) and is suitable only where the parent channel is deep (say 2 m or more) and the offtake is comparatively small (7).

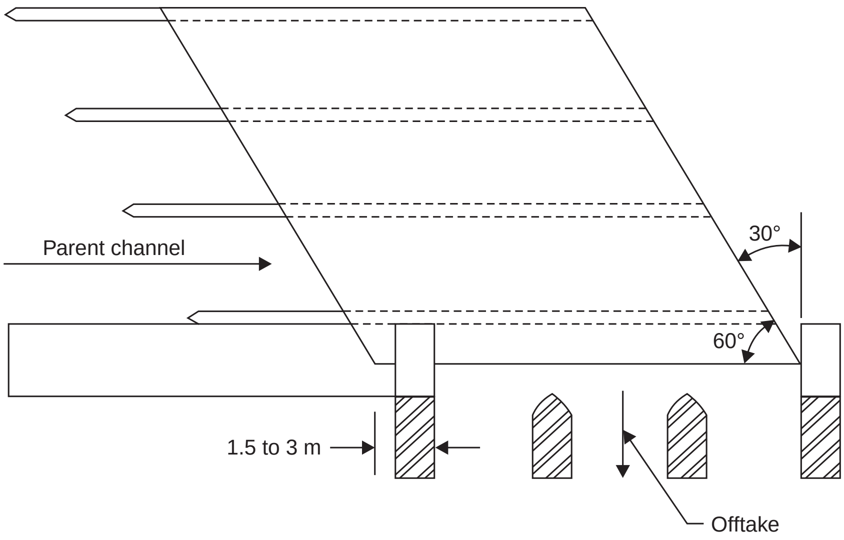


Fig. 10.18 Skimming platform (7)

10.14. CANAL ESCAPES

A canal escape is a structure to dispose of surplus or excess water from a canal. A canal escape essentially serves as a safety valve for the canal system. It provides protection of the canal against possible damage due to excess supplies which may be on account of either a mistake in releasing water at headworks, or a heavy rainfall due to which there may be sudden reduction in demand, making the cultivators close their outlets. The excess supply makes the canal banks vulnerable to breaches or dangerous leaks and, hence, provision for disposing of excess supply in the form of canal escapes at suitable intervals along the canal is desirable. Besides, emptying the canal for repairs and maintenance and removing a part of sediment deposited in the canal can also be accomplished with the help of the canal escapes. The escapes are usually of the following types (8):

(*i*) *Weir or surface escape:* These are weirs or flush escapes constructed either in masonry or concrete with or without crest shutters which are capable of disposing of surplus water from the canal.

(*ii*) *Sluice escapes:* Sluices are also used as surplus escapes. These sluices can empty the canal quickly for repair and maintenance and, in some cases, act as scouring sluices to facilitate removal of sediment.

Location of escape depends on the availability of suitable drains, depressions or rivers with their bed level at or below the canal bed level for disposing of surplus water through the escapes, directly or through an escape channel.

EXERCISES

10.1 Why does the need of a canal fall arise ? Describe different types of fall.

10.2 What are the functions of a distributary head regulator and a cross regulator ?

10.3 A Sarda fall is to be designed for a drop of 1.5 m in a channel 20 m wide and carrying 20 m^3/s of water discharge at a depth of 1.5 m. Determine: (*i*) the crest dimensions, (*ii*) the minimum length of floor to be provided on downstream, and (*iii*) the length and depth of the cistern.

10.4 Design 1.25 m Sarda fall on a channel carrying 22 m^3/s of water with bed width and water depth of 16.0 m and 1.75 m respectively. The channel cross-section is trapezoidal with side slopes as 1.5 (H) : 1 (V). Draw the plan and longitudinal section of the fall.

10.5 Design a 2.5 m straight glacis fall on a channel carrying 35 m^3/s and having a bed width of 25 m and full supply depth of 1.8 m. The slope of the glacis is to be kept 2.5 (H) : 1 (V). Safe exit gradient may be taken as 1/6.

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11

CROSS-DRAINAGE STRUCTURES

11.1. NEED OF CROSS-DRAINAGE STRUCTURES

Aligning a canal on the watershed of an area is necessary so that water from the canal can flow by gravity to fields on both sides of the canal. However, a canal taking off from a river at *A* (Fig. 11.1) has to necessarily cross some streams or drainages (such as at *a, b, c,* and *d* in the figure) before it can mount the watershed of the area at *B*. In order to carry a canal across the streams, major cross-drainage structures have to be constructed. Once the canal is on the watershed at *B*, usually no cross-drainage structure is required except in situations when the canal has to leave a looping watershed (such as *DEF* in Fig. 11.1) for a short distance between *D* and *F*, and may cross tributaries (as at *e* and *f*). Cross-drainage structures are constructed to negotiate an aligned channel over, below, or at the same level of a stream (1).

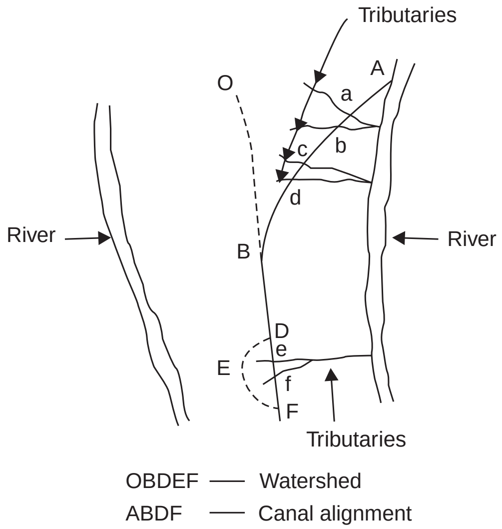

Fig. 11.1 Canal alignment between offtake and watershed

11.2. TYPES OF CROSS-DRAINAGE STRUCTURE

The cross-drainage structures can be classified under three broad categories depending on whether the structure is built to negotiate a carrier channel over, below, or at the same level as the stream channel.

11.2.1. Structures for a Carrier Channel Over a Natural Stream

The structures falling under this category are aqueducts and siphon aqueducts. Maintenance of such structures is relatively easy as these are above ground and can be easily inspected. When the full supply level (FSL) of a canal is much higher than the high flood level (HFL) of a stream which, in turn, is lower than the bottom of the canal trough, the canal is carried over the stream by means of a bridge-like structure, which is called an *aqueduct*. The stream water passes through the space below the canal such that the HFL is lower than the underside of the canal trough, Fig. 11.2

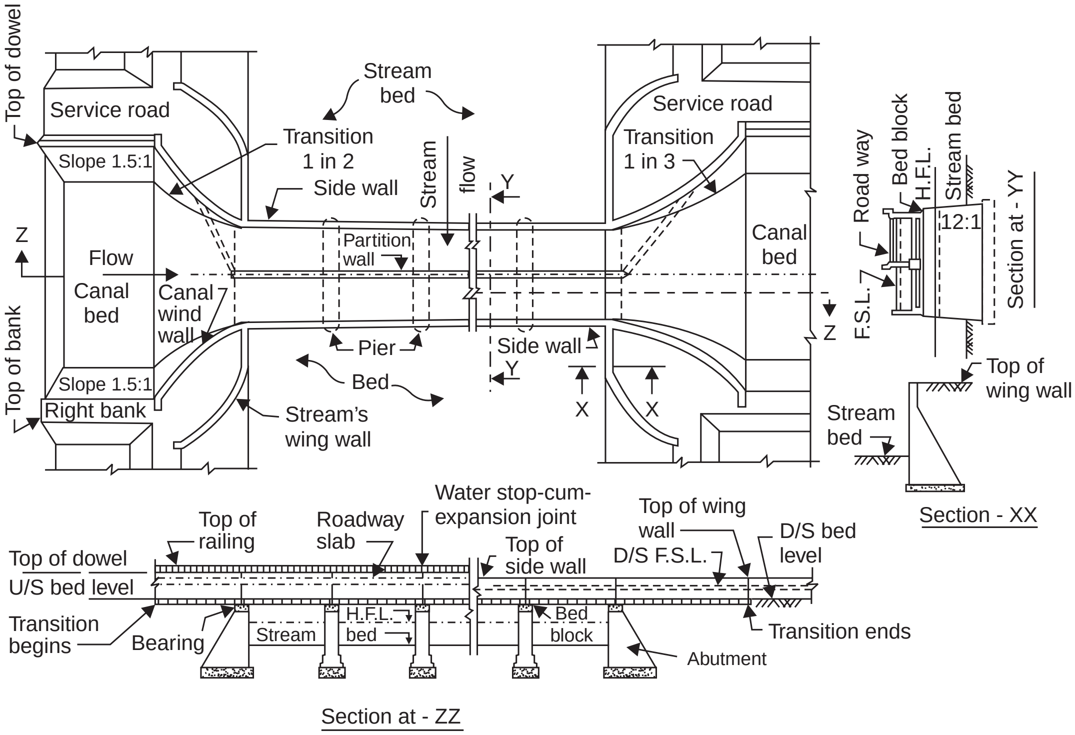

Fig. 11.2 Typical plan and section of an aqueduct (1)

Siphon aqueduct (Fig. 11.3) is an aqueduct in which the bed of the stream is depressed when it passes under the canal trough, and the stream water flows under pressure below the canal. In such aqueducts, the stream bed is usually provided with a concrete or masonry floor.

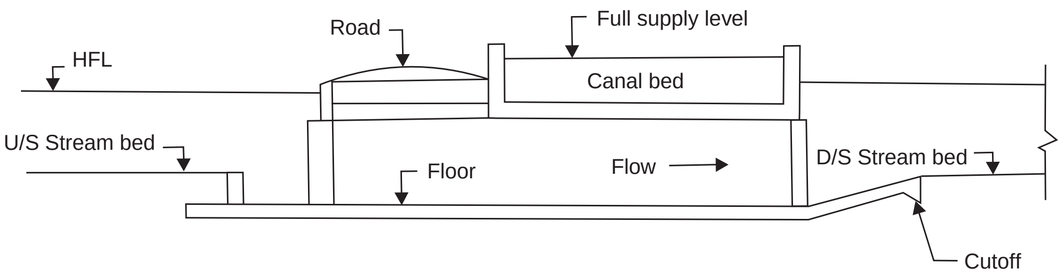

Fig. 11.3 Siphon aqueduct

Aqueducts and siphon aqueducts are further classified into the following types:

Type I: In this type of structure, the earthen canal banks are carried as such and, hence, the culvert length (*i.e.,* the length of barrels through which the stream water is passed under the canal) has to be long enough to support the water section as well as the earthen banks of the canal [Fig. 11.4 (*a*)]. In this type of structure, the canal section is not flumed and remains unaltered. Hence, the width (across the canal) of the structure is maximum. This type of structure, obviously, saves on canal wings and banks connections and is justified only for small streams so that the length (along the canal) of the structure is small. An extreme example of such a structure would be when the stream is carried by means of a pipe laid under the bed of the canal.

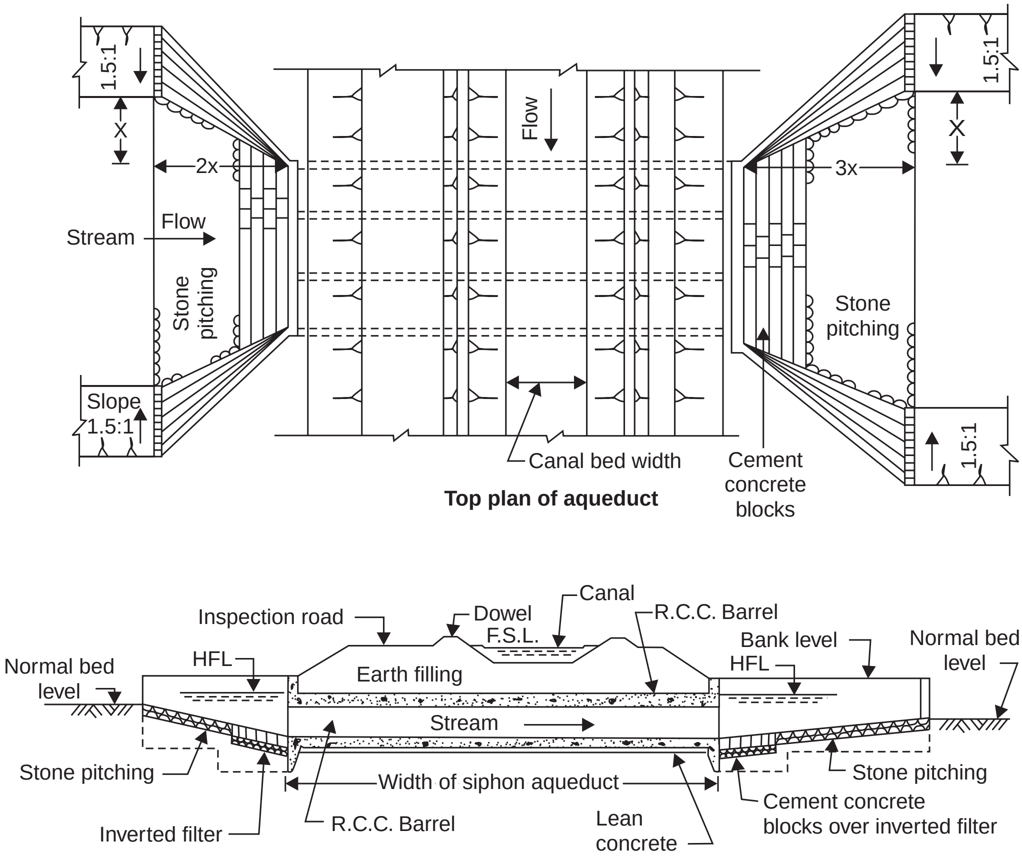

Fig. 11.4 (*a*) Typical plan and section of a siphon aqueduct (Type I) (1)

Type II: This type of structure is similar to the Type I with a provision of retaining walls to retain the outer slopes of the earthen canal banks [Fig. 11.4 (*b*)]. This reduces the length of the culvert. This type of construction can be considered suitable for streams of intermediate size.

Type III: In this type of structure, the earthen canal banks are discontinued through the aqueduct and the canal water is carried in a trough which may be of either masonry or concrete [Fig. 11.4 (*c*)]. The earthen canal banks are connected to the respective trough walls on their sides by means of wing walls. The width of the canal is also reduced over the crossing. In this type of structure, the width of the structure is minimum and, hence, the structure is suitable for large streams requiring considerable length of aqueduct between the abutments.

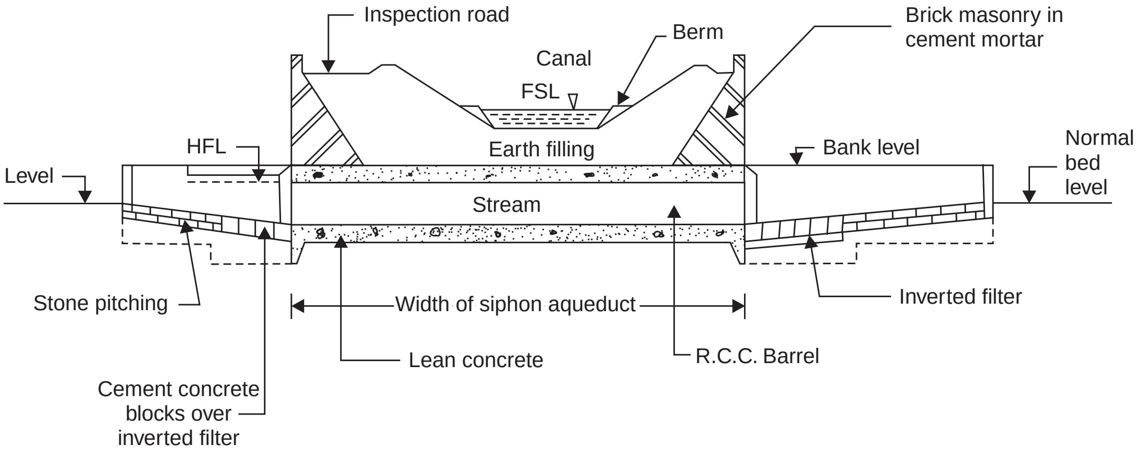

Fig. 11.4 (*b*) Typical section of a siphon aqueduct (Type II) (1)

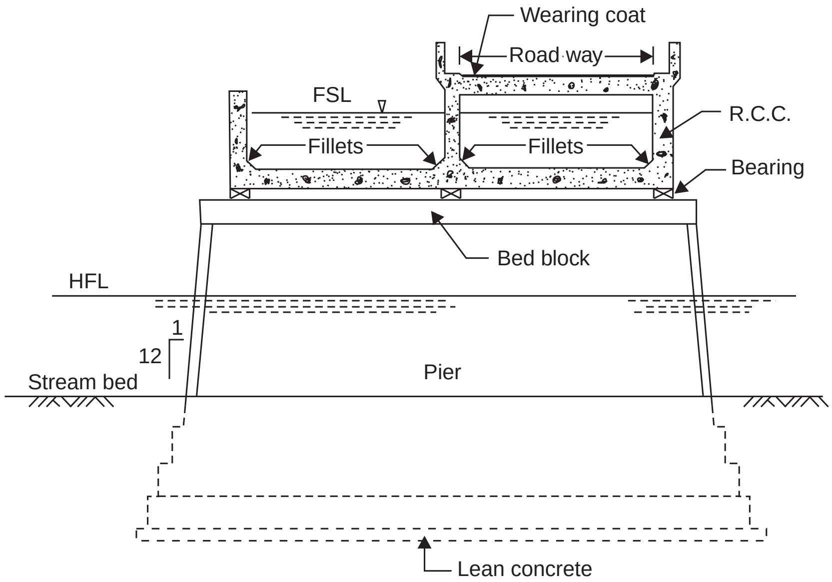

Fig. 11.4 (*c*) Typical section of an aqueduct (Type III) (1)

11.2.2. Structures for a Carrier Channel Underneath a Natural Stream

The structures falling under this category are superpassages and siphons. The maintenance of such structures is relatively difficult as these are not easily accessible.

A superpassage [Fig. 11.5] is like an aqueduct, but carries the stream over the canal. The canal FSL is lower than the underside of the stream trough and, hence, the canal water flows with a free surface.

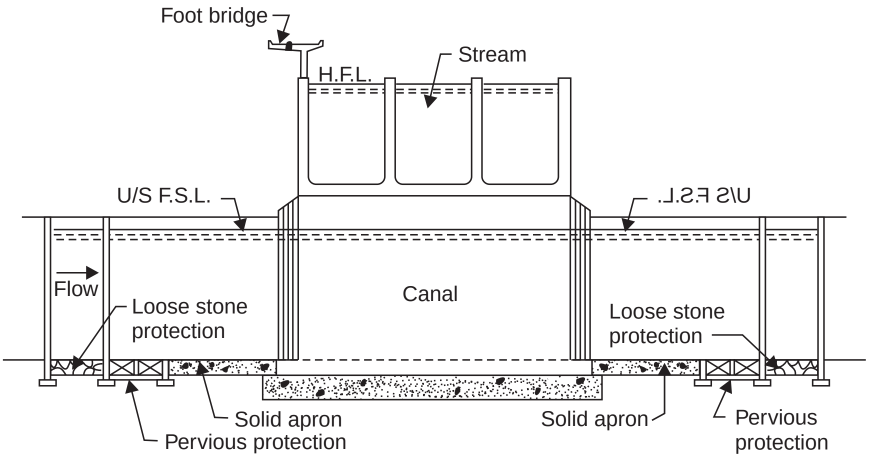

Fig. 11.5 Typical cross-section of a superpassage (1)

A siphon [Fig. 11.6] carries the canal water under pressure through barrels below the stream trough. For siphoning small discharges, precast RCC pipes will be economical. For siphoning higher discharges, horse-shoe-shaped rectangular or circular barrels, either single or multiple, are adopted. Roofs of rectangular barrels are, at times, arch-shaped for economy. For discharges under high pressures, circular or horse-shoe-shaped barrels are more suitable.

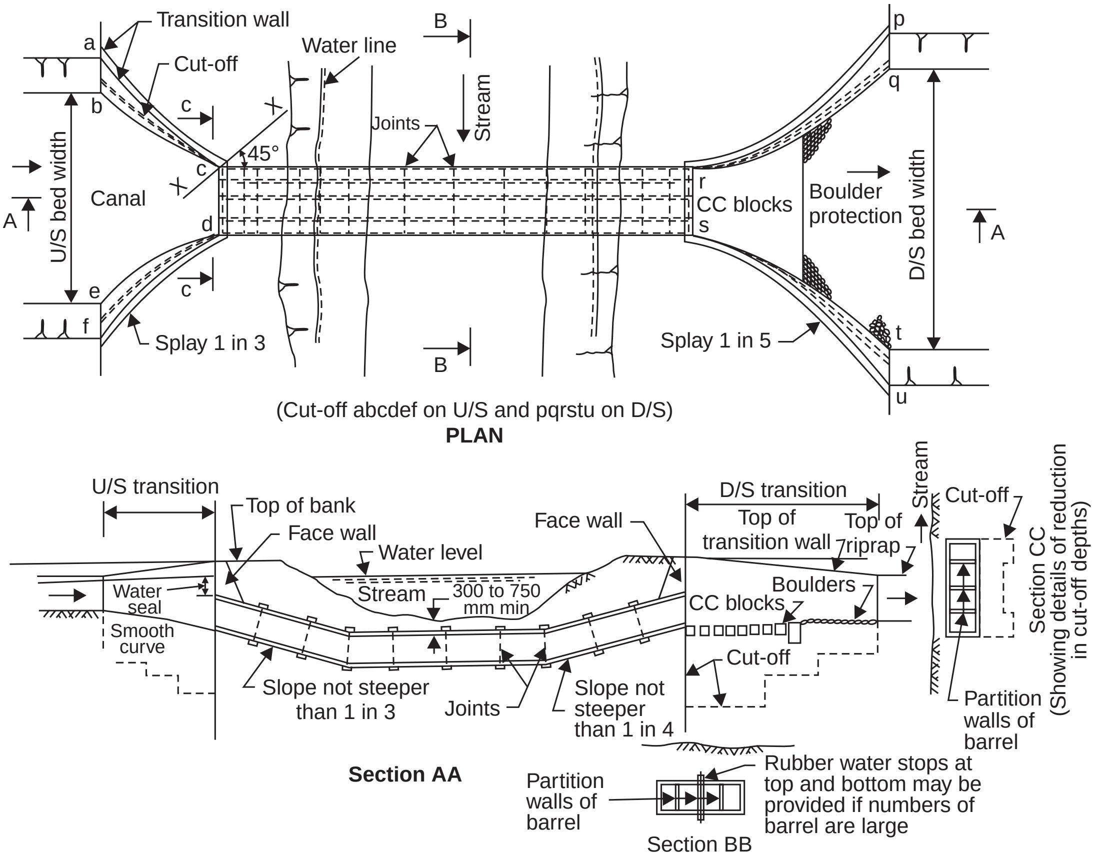

Fig. 11.6 Profile of a typical canal siphon (1)

11.2.3. Structures for Carrier Channel Crossing a Natural Stream at the Same Level

Structures falling under this category are level crossings and inlets. Inlets are, at times, combined with escapes. When the canal and the stream meet each other at practically the same level, a level crossing [Fig. 11.7] is provided. Level crossings involve intermixing of the canal and stream waters. They are usually provided when a large-sized canal crosses a large stream which carries a large discharge during high floods, and when siphoning of either of the two is prohibitive on considerations of economy and non-permissibility of head loss through siphon barrels (1). A barrier with its top at the canal FSL is constructed across the stream and at the upstream end of the junction. The regulators are provided across the stream and canal at the downstream junctions of the level crossing. These regulators control the flow into the canal and stream downstream of the crossing. This type of arrangement is also useful in augmenting the canal supplies with the stream discharge.

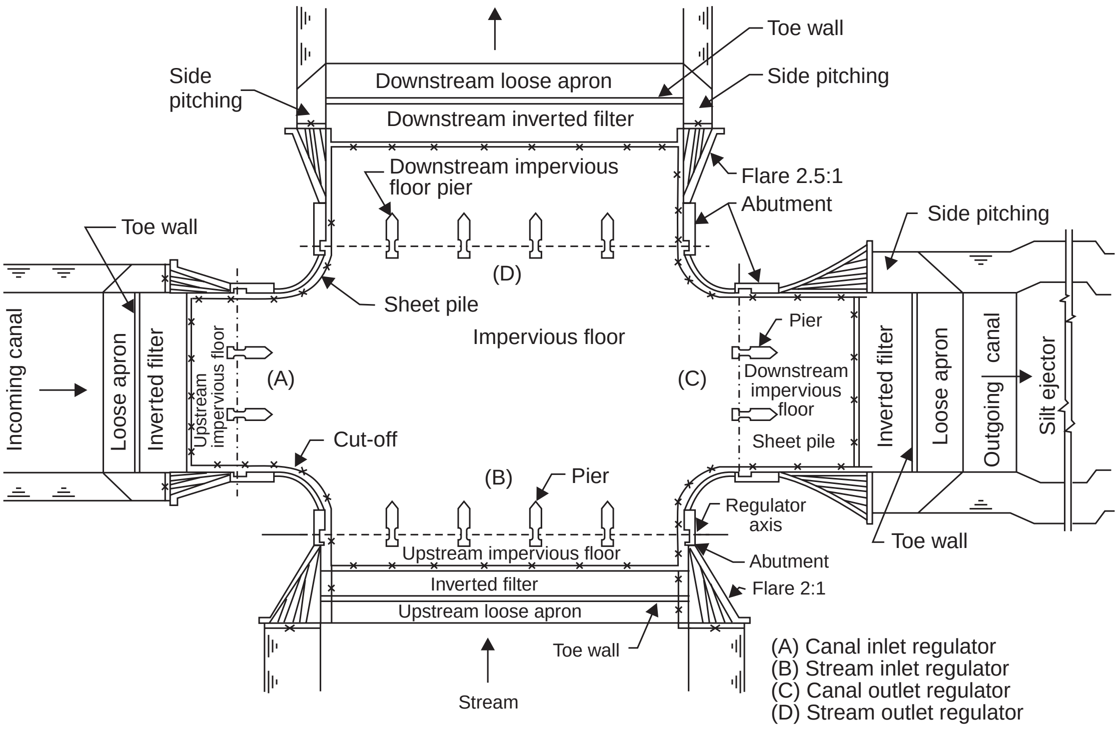

Fig. 11.7 Typical layout plan of a level crossing (1)

When the stream is dry, the stream regulator is kept closed and the canal regulator is opened so that the canal water flows in the canal itself without interruption. When the stream is bringing water, it mixes with the canal water and the stream regulator is used to dispose of that part of the stream water which is not used to augment the canal supply.

If a small and relatively sediment-free stream meets the canal at practically the same level (or its bed level is higher than the canal FSL) then an inlet is provided. An inlet is a structure consisting of an opening in a canal bank, suitably protected, to admit upland stream water into the canal. Inlets are constructed only if the stream discharge is too small and does not carry large quantity of sediment. Inlets do not have a regulator and, hence, the stream bed should be higher than the canal FSL. An inlet consists of a fall or a pitched slope confined within wing walls to guide the stream water into the canal. An inlet simply allows the stream water to be taken into the canal. If the stream water so taken into the canal is appreciable in quantity, it is allowed to flow out at a suitable site downstream (along the canal) of the inlet. The outlet is generally combined with some other structure for economic reasons, but at times only an inlet (and no outlet) is provided.

Sometimes a small stream is diverted into a larger stream and a cross-drainage structure for the combined discharge is provided at a suitable site.

11.3. SELECTION OF SUITABLE CROSS-DRAINAGE STRUCTURE

Relative differences in bed and water levels of a canal with those of the crossing stream as well as their discharges are the main factors for deciding the type of cross-drainage structure at a site. By suitably changing the alignment of the canal between offtaking point *A* and the watershed (Fig. 11.8) the relative difference between the bed levels of the tributaries and the canal at the crossing site can be altered. Consider three possible alignments *ABC, ADE,* and *AFG* of a canal taking off from a river at *A* and intersecting a tributary *HBDFI* at *B*, *D,* and *F* before mounting the watershed at *C, E,* and *G*, respectively (Fig. 11.8). The distances *AB, AD,* and *AF* are almost the same and, hence, the canal reaches the crossing site with its bed more or less at the same level. But, the bed levels of the tributary at *B, D,* and *F* are significantly different due to the slope of the tributary. Obviously, the bed level of the tributary is the highest at *B*, and the lowest at *F* in the reach *BDF*. Thus, if *F* is a suitable crossing site for aqueduct, site *D* may necessitate the construction of a siphon aqueduct or level crossing and site *B* may require the construction of a siphon or a superpassage. Thus, the type of cross-drainage structure can be changed by suitably altering the crossing site.

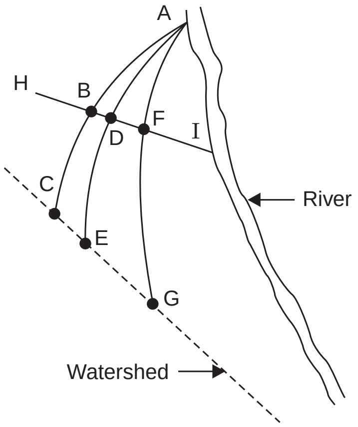

Fig. 11.8 Different canal alignments between offtake and watershed

When the crossing site is such that the canal FSL is well above the stream HFL the choice between aqueduct and siphon aqueduct is made depending on the stream discharge. For larger stream discharges (*i.e.*, when the stream bed is much wider) an aqueduct is more suitable than a siphon aqueduct which requires lowering of the stream bed by a drop. Besides being costly, lowering of the bed may result in silting on the lowered stream bed which increases the risk of failure. However, an aqueduct necessitates heavy canal embankments towards the crossing (Fig. 11.9). This is due to the wide flood cross-section of streams in plains and the requirements that the canal must be well above the HFL, and the aqueduct has to be constructed in a smaller part of the cross-section of the stream.

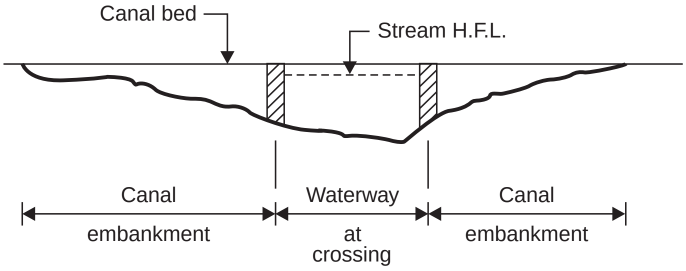

Fig. 11.9 Canal embankments near crossing site

Siphon aqueducts are more suitable when the stream size is small compared to the canal size. In case of siphon aqueducts, the relative differences of water and bed levels of the canal with those of the crossing stream is small and, hence, embankments of only small height are required.

If the stream HFL is well above the canal FSL, superpassage is generally preferred in comparison to siphon as the latter involves considerable head loss in the canal. In addition, the construction of a siphon under a stream with an erodible bed requires heavy protection works. The foundations of the superpassage and siphon have to be carried up to much below the erodible bed of the stream. A separate bridge across the stream trough has to be provided to carry the canal road across the stream. The construction of these structures is relatively difficult and costly due to the requirements of extensive training works and large stream trough to carry the high flood discharge. If the canal serves navigation needs also, then sufficient headway should be provided for the passage of boats.

If the bed and water levels of the canal and stream at the crossing site are approximately the same, a level crossing is provided. Sometimes, due to prohibitive costs of siphons and siphon aqueducts, the canal alignment between the offtake and watershed is suitably altered so that the level crossing can be provided at the crossing site. The initial cost of a level crossing is generally much lower than the cost of other cross-drainage structures. Also, the perennial discharge of the stream can be diverted to the canal to provide additional irrigation. However, the level crossing requires permanent staff for continuous watch, maintenance, and operation of gates. Also, when the stream is passing the high flood discharge, the canal may have to be closed down to prevent the sediment load of the stream from entering the canal and silting it. Further, if the canal FSL is higher than the general ground level, the HFL of the stream would increase on the upstream side of the crossing site causing submergence of the land. To prevent such submergence of the land, marginal banks are provided.

In addition to the above factors, the topography of the terrain, foundation conditions, regime of the stream, and dewatering requirements would also affect the choice of the type of cross-drainage structures. Detailed examination of the terrain topography and the foundation is necessary to locate a stable reach of the stream with good foundations and permitting preferably a right-angled crossing. For streams carrying high sediment discharge, the possibility of chocking up of the siphon and the effect of fluming of the stream should be kept in mind. Dewatering of foundations is necessary in the construction of foundations for cross-drainage structures. An accurate estimate of the cost and method of dewatering must be worked out when design involves laying of foundations below the ground water table.

11.4. DESIGN OF CROSS-DRAINAGE STRUCTURES

Before undertaking detailed designs of any important cross-drainage structure, collection of relevant field data is required. Besides, a note regarding the several alternative alignments surveyed and reasons for final selection of a particular crossing site is also necessary. A location map of the site along with the results of subsurface explorations of the site, and stream cross-sections at different locations around the site should also be prepared. The following specific hydraulic data regarding the canal and stream should be made available.

(*i*) *Canal*

(*a*) Discharge, depth of flow, and water level at full supply,

(*b*) Bed width,

(*c*) Canal bed slope and the water surface slope,

(*d*) Levels of canal bed and the top of canal banks,

(*e*) Channel cross-section,

(*f*) Characteristics of material of the bed and sides of the channel, and

(*g*) Width of roadway and type and class of IRC loading.

(*ii*) *Stream*

(*a*) Extent and nature of stream and its catchment,

(*b*) Detailed records of rainfall in the catchment,

(*c*) Maximum observed discharge,

(*d*) Maximum flood level and water surface slope as observed under the highest flood condition at the proposed site,

(*e*) Site plan of the proposed crossing including contours,

(*f*) Log of borehole or trial pit data.

(*g*) Information about the sediment load,

(*h*) Characteristics of the bed material including Manning's n and silt factors.

(*i*) Longitudinal section of the stream for suitable distance upstream and downstream of the chosen site for cross-drainage structure depending upon the site conditions,

(*j*) Cross-section of the stream for about 100 to 300 m upstream and downstream of the site at intervals of 10 to 50 m,

(*k*) Waterways provided in road/railway bridges or other hydraulic structures of the area,

(*l*) Spring water levels at the crossing site.

Any cross-drainage structure should preferably be located in a straight reach of the stream crossing the canal at right angle as far as possible. The alignment of the canal should also be such that it results in minimum lengths of embankments (for aqueduct and siphon aqueduct structures). If required, the site of the structure may even be shifted away from the existing stream channel, when it is possible to divert the channel and also keep it there by reasonable training works. One obvious advantage of such an alternative would be that the construction will be carried out in dry conditions.

The design discharge from any cross-drainage structure would depend upon the size and importance of the structure. The failure of a large cross-drainage structure may result in the submergence of considerable cultivable and residential areas besides interrupting irrigation and resulting in reduction of crop yield over a large area. Therefore, the design discharge for very large cross-drainage structures should be based on the maximum probable flood from the maximum probable storm. However, for small cross-drainage structures, it would be very uneconomical to use the highest peak flood for design. The failure of a small cross-drainage structure would not cause much submersion, and interrupt irrigation only to a marginal extent. In the long run, it may prove to be more economical to repair or even replace relatively small cross-drainage structures at long intervals than to spend very large sums in order to provide for the highest peak flood.

For major cross-drainage structures, the design flood discharge can be taken as the discharge of a 1-in-50 to 1-in-100-years flood. For small cross-drainage structures, however, the design flood discharge may correspond to a 10- to 25-years frequency flood with increased afflux. In cases of important structures, an additional margin of safety is provided in the foundation design and fixation of the freeboard to take care of the unexpected and unforeseen nature of flood intensities (Table 11.1).

Table 11.1 Suggested increase in design discharge (1)

Catchment area (Km^2)	*Increase in discharge* (%)
More than 25,000	0—10
5000-25,000	10—20
500-5000	20—25
Less than 500	25—30

The type of foundation for cross-drainage structures will depend primarily on the depth of scour calculated from Lacey's equation [Eq. (8.32) or Eq. (8.33)], and the bearing capacity of the soil. The depth of scour around piers is taken as twice the depth of scour calculated from Lacey's equation. In alluvial streams, a well foundation is usually provided where deep foundation is required. With the provision of an impervious floor (necessary for siphon and siphon aqueduct) along with cutoff walls, the depth of foundation may be reduced. The floor itself may be designed as either a gravity floor or a raft. The floor is designed to resist the total uplift pressure caused by subsoil water and the water seeping from the canal. The uplift pressure is counterbalanced by the dead weight of the gravity floor. The worst condition occurs when there is no water in the barrel and, hence, the weight of water in the barrel is not included in the design. At times, it may be economical to design the floor as a raft so that the uplift is counterbalanced by the entire weight of the superstructure. The spacing of the piers (*i.e.,* the span) depends on structural and economic considerations. Fewer piers (*i.e.,* longer span) are preferable at sites which require costly foundation.

In case of siphon aqueducts and siphons, the drop at the upstream end of the culvert may be vertical (generally economical) or sloping. But, at the downstream end of the culvert, the rise should always be at a slope flatter than 1 in 4 so that the bed load can be moved out of the siphon barrel. The culvert floor should extend upstream of the barrel inlet by a distance equal to the difference between the HFL and the culvert floor level. Barrel inlet should be bell-mouthed to reduce the head losses.

A suitable arrangement has to be provided to pass the service road across the stream. This requirement does not pose much of a problem in cross-drainage structures of type I and II in which earthen embankments are continued. In cross-drainage structures of type III, the simplest arrangement is to carry the road on either side (or only on one side for economic reasons) by providing slabs and arches on either side (or on one side) of the canal trough.

The sides of the canal trough are generally designed as beams in reinforced concrete structures. The bottom slab is suspended from these beams. Additional beams, if required, are projected into the canal to divide the canal trough into a number of parallel channels. For wider troughs having intermediate beams, the service road may be provided on one of the compartments. Canal troughs of the smaller width can be designed as a hollow box girder and the service road can be provided on the top slab.

The forces acting on a cross-drainage structure consist mainly of the hydrostatic and uplift pressures, earth pressures, and dead weights and live loads of construction equipment and traffic. The overall design of the structure should be such that the total weight of the structure is as small as possible. Possibilities of differential settlement and excessive scour during floods must always be kept in mind while designing the foundations.

Wing walls of the stream are suitably connected to high ground. The stream should be guided towards the structure by means of suitable river training works. Similarly, the canal banks, adjacent to the crossing, should be protected by measures such as pitching, launching apron, *etc.*, wherever necessary.

The design of any cross-drainage structure, like any other hydraulic structure, includes hydraulic, structural and foundation aspects. The hydraulic aspects include the surface and subsurface flow considerations. The surface flow determines the configuration of the structure so that the structure is economic and functionally efficient. The surface and the subsurface flow considerations enable determination of the following:

(*i*) Waterway and headway of the stream,

(*ii*) Head loss through the cross-drainage structure,

(*iii*) Fluming or contraction of the canal waterway,

(*iv*) Uplift pressures on the trough,

(*v*) The uplift pressures on the culvert floor,

(*vi*) The exit gradient, and

(*vii*) Protection works.

The hydraulic aspects of the design of cross-drainage structures have been dealt with in the following paragraphs.

11.5. WATERWAY AND HEADWAY OF THE STREAM

Economic and safety considerations limit the waterway of the stream. Lacey's regime perimeter equation [Eq. (8.29)] can be used for determining the permissible waterway for structure without rigid floor. In structures with rigid floors, however, the waterway can be further reduced, but keeping the flow velocities within permissible limits (Table 11.2).

The purpose of providing waterway equal to the Lacey's perimeter is to let a stable channel develop between the training banks (*i.e.,* the guide banks) upstream of the cross-drainage structure. At the site of the structure, the piers for the structure will reduce the available clear waterway. For small structures, the permissible reduction in clear waterway is up to 20%. It should be noted that at the site of the structure, the regime conditions do not exist and the reduction in clear waterway up to 20% is not a cause of concern if the scour is calculated using the increased discharge per unit length of clear waterway.

Table 11.2 Maximum permissible velocities (1)

Type of floor	*Maximum permissible velocity (m/s)*
Steel-and cast iron-lined face	10
Concrete-lined face	6
Stone masonry with cement pointing	4
Stone (or brick) masonry with cement plastering	4
Hard rock	4
Brick masonry with cement pointing	2.5

In aqueducts, the height of barrels is fixed such that the canal trough is about 0.6 m above the HFL of the stream. The requirement of the distance between the full supply level of canal and the bottom of the stream trough is, however, less in case of superpassage as FSL is relatively a certain quantity. But, in siphon aqueducts, the required waterway area is calculated on the basis of permissible scouring velocity (generally 2-3 m/s) through the barrels. Velocities higher than the permissible velocity will result in higher afflux upstream of the structure resulting in higher and longer marginal banks.

In aqueducts as well as siphon aqueducts, it is necessary to have sufficient headway between the bed level of the stream (downstream of the crossing) and underside of the culvert roof. This headway should be at least 1 m or half the height of the barrel, whichever is less. In the absence of a clear headway, there will exist the risk of the barrels being blocked because of silting. To fulfil the requirements of the headway, the stream bed may have to be lowered by providing a fall upstream of the crossing.

11.6. HEAD LOSS THROUGH CROSS-DRAINAGE STRUCTURES

The HFL of a stream downstream of a cross-drainage structure remains unchanged but the upstream water level will rise by an amount equal to the head loss (or afflux) due to the flow in the barrels of the cross-drainage structure. The length and top elevation of the guide banks upstream of the structure will depend on the raised HFL. Depending upon the flow conditions in the barrels, the broad-crested weir discharge formula or orifice discharge formula can be used for calculating afflux (1).

Alternatively, one can determine the afflux Δh from the following empirical formula proposed by Yarnell (2):

$$\frac{\Delta h}{h_3} = KF_3^2[K + 5F_3^2 - 0.6]\,(\alpha + 15\,\alpha^4) \tag{11.1}$$

Here, h_3 is the depth of flow sufficiently downstream of the piers, and F_3 is the corresponding Froude number. The term α is the ratio of the width of pier with the spacing (centre-to-centre) of the piers, and K is dependent on the shape of the pier as given in Table 11.3.

Table 11.3 Values of K in Eq. (11.1)

Shape of pier	*K*
Semicircular nose and tail	0.90
Nose and tail formed of two circular curves each of radius equal to twice the pier width and each tangential to pier face	0.90
Twin-cylinder piers with connecting diaphragm	0.95
Twin-cylinder piers without diaphragm	1.05
90°-triangular nose and tail	1.05
Square nose and tail	1.25

The total head loss, h_L, for a flow through siphon or siphon aqueduct can be obtained as the sum of the losses at the inlet and outlet and the friction loss. If approach velocity V_a is also significant, the approach velocity head may also be taken into account. Assuming the downstream velocity head to be negligible, the head loss (or afflux) h_L is expressed as,

$$h_L = \left(1 + f_1 + f_2 \frac{L}{R}\right)\frac{V^2}{2g} - \frac{V_a{}^2}{2g} \qquad \text{...(11.2)}$$

Here, L is the length of the barrel, R the hydraulic radius of the barrel section, V the velocity of flow through the barrel section, and V_a is the approach velocity which is generally neglected. f_1 is the entry loss coefficient whose value is 0.505 for an unshaped entrance and 0.08 for bell-mouthed entrance (1). f_2 is a coefficient similar to the friction factor and is equal to $a\ [1 + (b/R)]$. Here, R is expressed in metres. The values of a and b for different types of barrel surface are given in Table 11.4.

Table 11.4 Values of a and b for f_2 (3)

Barrel surface	*a*	*b*
Smooth iron pipe	0.00497	0.025
Incrusted iron pipe	0.00996	0.025
Smooth cement plaster	0.00316	0.030
Ashlar or brickwork or planks	0.00401	0.070
Rubble masonry or stone pitching	0.00507	0.250

In order to minimise the head loss and afflux, the barrel surface should be made smooth and the entrance of the barrel bell-mouthed.

11.7. DESIGN OF TRANSITIONS FOR CANAL WATERWAY

The cost of an aqueduct or siphon aqueduct will depend on its width and other factors. The width of the aqueduct is, therefore, reduced by contracting the canal waterway. However, the canal waterway is not contracted in case of earthen banks and, hence, contraction of the canal waterway is considered only in structures of type III [Fig. 11.4 (*c*)]. There is, however, a limit up to which a canal can be contracted or flumed. The fluming should be such that it should not result in supercritical velocity in the canal trough. The supercritical velocity in the canal trough may cause the formation of a hydraulic jump before the supercritical flow of the canal trough meets the subcritical flow of the normal canal section. The jump formation would result in additional loss of energy and large forces on the structure. Also, the lengths of transitions increase with the amount of reduction in the canal width. The fluming should be such that the cost of additional length of transition is less than the savings in cost on account of the reduction in the width of the aqueduct (or siphon aqueduct). The canal should not be flumed to less than 75 per cent of bed width. If the velocity and the head loss permit, greater fluming may be allowed ensuring subcritical flow conditions in the flumed canal (1). Proper transitions need to be provided between the flumed portion and normal section of the canal. A minimum splay of 2 : 1 to 3 : 1 for the upstream contracting transition, and 3 : 1 to 5 : 1 for the downstream expanding transition is always provided (1).

An increase in the depth of flow in the transitions and also in the canal trough will result in deeper foundation and higher pressures on the roof as well as higher uplift on floor of the culvert. The canal trough and transitions are, therefore, generally designed keeping the depth of flow the same as in the normal section.

While constructing a cross-drainage structure, it is often required to connect two channels of different cross-sectional shapes. Usually, a trapezoidal channel is required to be connected to a rectangular channel or vice versa. A channel structure providing the cross-sectional change between two channels of different cross-sectional shapes is called channel transition or, simply, transition. Transition structure should be designed such that it minimises the energy loss, eliminates cross-waves, standing waves and turbulence, and provides safety for the transition as well as the waterway. Besides, it should be convenient to design and construct the structure. These transitions are usually gradual for large and important structures so that the transition losses are small. Abrupt transitions may, however, be provided on smaller structures.

For a contracting transition (Fig. 11.10), the flow is accelerating and as such any gradual contraction which is smooth and continuous should be satisfactory. A quadrant of an ellipse with its centre on *bb* can be chosen for determining the profile of the bed line. For a splay of 1 in m_1, the bed line profile can be written as

$$\left[\frac{x}{0.5m_1\,(B_c - B_f)}\right]^2 + \left[\frac{y}{0.5\,(B_c - B_f)}\right]^2 = 1 \tag{11.3}$$

Side slopes and the bed elevation may be linearly varied. The value of m_1 should be kept higher than 3. In the case of an expanding transition, however, more care must be exercised. Following discussions pertain to the design of expanding transitions.

Irrigation channels carry subcritical flow and the hydraulic design of gradual transitions for subcritical flow requires: (*i*) prediction of flow conditions at section *f-f* (Fig. 11.10) for the given size and bed elevation of the flume and also the flow condition at the exit section *c-c*, and (*ii*) determination of the boundary shape and flow conditions within the transition. For given flow conditions in the exit channel, the depth and velocity of flow (and, hence, the energy loss) within the transition are governed by the following three boundary variations:

$$B = f_1\,(x) \tag{11.4}$$

$$\Delta z = f_2\,(x) \tag{11.5}$$

and

$$m = f_3\,(x) \tag{11.6}$$

where, B = the bed width of the transition at distance x from the flume end of the transition (Fig. 11.10),

Δz = change in bed level (with respect to the flume bed) within the transition,

and m = channel side slope (*i.e.*, $m(H) : 1\ (V)$ within the transition).

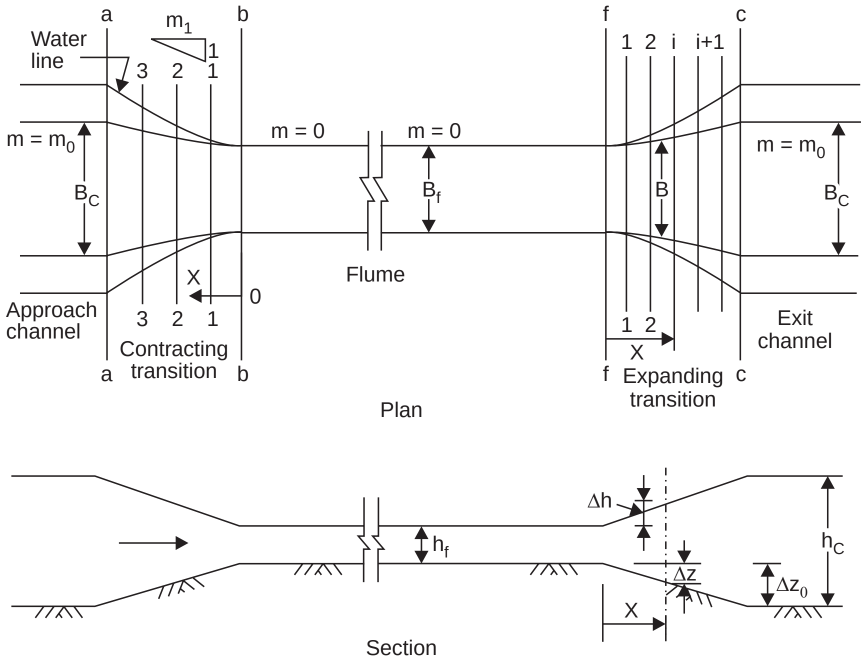

Fig. 11.10 Line sketch of canal trough and transitions

The flow rate Q is related to the depth of flow h and velocity of flow v in the transition as follows:

$$Q = (B + mh)\, vh \tag{11.7}$$

Applying Bernoulli's equation between the i^{th} and $(i + 1)^{th}$ sections (with flume bottom as the datum), one obtains

$$-\Delta z_i + (\Delta z_i + h_f + \Delta h_i) + \frac{v_i^2}{2g} = -\Delta z_{i+1} + (\Delta z_{i+1} + h_f + \Delta h_{i+1}) + \frac{v_{i+1}^2}{2g} + h_{L_{i,i+1}} \tag{11.8}$$

Here, h_f is the depth of flow in the flume and $h_{L_{i,i+1}}$ is the energy loss between the i^{th} and $(i + 1)^{th}$ sections. Further, Δz is considered as positive when the transition bed is lower than the flume bed, and the water surface elevation increment Δh (with respect to the water surface in the flume) is positive when the water surface in the transition is higher than the water surface in the flume (4). Equation (11.8) can, alternatively, be written as

$$E_{i+1} = E_i + \Delta z_{i,\,i+1} - h_{L_{i,i+1}} \tag{11.9}$$

where, $\Delta z_{i,\,i+1} = \Delta z_{i+1} - \Delta z_i$

and specific energy $$E = \Delta z + h_f + \Delta h + \frac{v^2}{2g}$$

Hinds (5) proposed the following equation for estimating the head loss through transitions:

$$h_{L_{i,i+1}} = K_h \frac{v_i^{\,2} - v_{i+1}^2}{2g} = K_h \,\Delta\, h_v \tag{11.10}$$

On substituting Eq. (11.10) into Eq. (11.9) and simplifying, one gets,

$$\Delta z_{i+1} + h_f + \Delta h_{i+1} + \frac{v_{i+1}^2}{2g} = \Delta z_i + h_f + \Delta h_i + \frac{v_i^{\,2}}{2g} + \Delta z_{i+1} - \Delta z_i - K_h\, \Delta h_v$$

$$\therefore \qquad \Delta h_{i+1} - \Delta h_i = \Delta h_v\,(1 - K_h)$$

or

$$\Delta h_v = \frac{\Delta h_{i+1} - \Delta h_i}{1 - K_h} \tag{11.11}$$

Hence, from Eqs. (11.10) and (11.11), one obtains,

$$h_{L_{i,i+1}} = \frac{K_h(\Delta h_{i+1} - \Delta h_i)}{(1 - K_h)} \tag{11.12}$$

A good transition design would require proper selection of the transition geometry (or determination of suitable values for B, Δz, and m of Eqs. (11.4-11.6) which would yield minimum energy loss consistent with the convenience of design and construction.

Following methods of design of transitions have been described here:

(*i*) Hinds' method (5).

(*ii*) UPIRI method (6) which is commonly known as Mitra's method.

(*iii*) Vittal and Chiranjeevi's method (4).

11.7.1. Hinds' Method

Hinds (5) assumed a water surface profile,

$$\Delta h = f_4\,(x) \tag{11.13}$$

in the transition as a compound curve consisting of two reverse parabolas with an inflexion point in the middle of the transition and which join the water surface at either end of the transition tangentially. The water surface profile equation [Eq. (11.13)] is, therefore, written as

$$\Delta h = C_1 x^2 \tag{11.14}$$

Here, C_1 is a coefficient to be determined from the coordinates of the junction of two parabolas, and x is to be measured from the transition end for the respective parabolas. Hinds (5) also assumed a linear rise (for contraction) or drop (for expansion) in the channel bed. Thus,

$$\Delta z = C_2\, x \tag{11.15}$$

where, C_2 is a constant.

The transition is now divided into N sub-reaches by cross-sections 1-1, 2-2, etc., and an arbitrary set of values for m, lying between 0 and m_0 (*i.e.,* $0 \le m \le m_0$), is assigned to these sections. Here, m_0 is the side slope of the exit (or approach) channel. Using Eqs. (11.14) and (11.15), one can compute Δh and Δz and, hence, the depth of flow ($= \Delta z + h_f + \Delta h$) for any cross-section of the transition. Using Eqs. (11.12) and (11.8), one can determine the flow velocity at the $(i + 1)^{th}$ section for known v_i, and substitution of v_{i+1} in Eq. (11.7) yields the bed width at the $(i + 1)^{th}$ section. These computations proceed from one end of the transition to the other end. If the resulting transition is not smooth and continuous, the computations are repeated with a new set of arbitrary values for m till a smooth and continuous bed width profile is obtained.

11.7.2. UPIRI Method

For channels of constant depth, it was assumed (6) that the rate of change of velocity per unit length of the transition should be constant throughout the transition length. This means,

$$(v_f - v_i)/x = (v_f - v_c)/L \tag{11.16}$$

Here, suffixes f and c are, respectively, for the flumed section and normal section of the channel, and x is the distance of the i^{th} section of the transition from the flumed section. Thus, v_i is the velocity of flow at the chosen i^{th} section. L is the length of the transition which was arbitrarily assumed as 2 $(B_c - B_f)$. Since the depth h is constant,

$$B_f v_f = B_i v_i = B_c v_c = Q/h$$

Hence,

$$v_f = \frac{Q}{h} \times \frac{1}{B_f}$$

$$v_i = \frac{Q}{h} \times \frac{1}{B_i}$$

and

$$v_c = \frac{Q}{h} \times \frac{1}{B_c}$$

Therefore, using Eq. (11.16),

$$\frac{(Q / hB_f) - (Q / hB_i)}{x} = \frac{(Q / hB_f) - (Q / hB_c)}{L}$$

or

$$B_i = \frac{B_c B_f L}{LB_c - x(B_c - B_f)} \tag{11.17}$$

Equation (11.17) is the hyperbolic bed-line equation for constant depth and can be used for transition between rectangular flume and rectangular channel.

11.7.3. Vittal and Chiranjeevi's Method

Vittal and Chiranjeevi (4) examined the above two methods and offered the following comments:

(*i*) Hinds' method involves a trial procedure which can be easily avoided if the values assigned to m follow a smooth and continuous function [Eq. (11.6)] instead of an arbitrary set of values for m. The function to be chosen should be such that the side slope varies gradually in that part of the transition where the velocities are higher and it varies rather fast in other part of the transition where the velocities are lower.

(*ii*) A smooth and continuous bed width profile alone is not sufficient to avoid separation and consequently the high head loss.

(*iii*) The total head loss through transition can be obtained by summing up all the head losses in the sub-reaches of the transition. Using Eq. (11.10), the total head loss H_L is given by

$$H_L = \sum h_{L_{i,i+1}} = K_h \sum \frac{v_i^{\,2} - v_{i+1}^{\,2}}{2g} = K_h \frac{v_f^{\,2} - v_c^{\,2}}{2g} \tag{11.18}$$

This means that the head loss in the transition depends only on the entrance and exit conditions, and is unaffected by the transition geometry. This, obviously, is not logical and is a limitation of Hinds' method.

(*iv*) Hinds' method first assumes free water surface and then computes the boundaries which would result in the assumed water surface. However, it is usually desirable to select the boundaries first and then compute the water surface profile.

(*v*) UPIRI method (6) would require lowering of the flume bed below the channel bed to achieve constant depth. This may not be always practical. For example, in case of a cross-drainage structure, the flume bed level may have to be lowered even below the drainage HFL. As a result, the cross-drainage structure, which otherwise can be an aqueduct, may have to be designed as a siphon aqueduct which is relatively more expensive.

On the basis of theoretical and experimental investigations, Vittal and Chiranjeevi (4) developed a method for the design of an expanding transition. The guiding principles for this method were minimisation of the energy loss and consideration of flow separation in the expanding flow. The design equations of Vittal and Chiranjeevi (4) for the bed width and side slope are as follows:

Transition bed width profile:

$$\frac{B - B_f}{B_c - B_f} = \frac{x}{L}\left[1 - \left(1 - \frac{x}{L}\right)^n\right] \tag{11.19}$$

where, $n = 0.80 - 0.26\, m_0^{1/2}$ (11.20)

and length of transition, $L = 2.35\,(B_c - B_f) + 1.65\, m_0\, h_c$ (11.21)

Side slopes, varying according to the equation,

$$\frac{m}{m_0} = 1 - \left(1 - \frac{x}{L}\right)^{1/2} \tag{11.22}$$

change gradually in the initial length of the transition where the flow velocity is high, and rapidly in the latter length of the transition where the velocity is low. For head loss computations, use of Hinds' equation, *viz.,* Eq. (11.10) with $K_h = 0.3$ has been suggested.

This method of design of expanding transition is applicable to all three types of conditions, *viz.,* constant depth, constant specific energy, and variable depth-variable specific energy. The profiles of the bed line and the side slope will be the same for all the three conditions. The bed of the expanding transition would always rise in the direction of flow for the constant depth scheme. On the other hand, constant specific energy and variable depth-variable specific energy schemes would always result in the falling bed transition.

For the constant specific energy condition,

$$E_c = h_c + \frac{Q^2}{2g\,B_c^{\,2} h_c^{\,2}} = E_i = h_i + \frac{Q^2}{2g\,B_i^{\,2} h_i^{\,2}}$$

Therefore, one can determine h_i and hence v_i. The transition loss between successive sections can be determined from Eq. (11.10) with $K_h = 0.3$. The bed has to be lowered at successive sections just sufficient to provide for the transition loss [as can be seen from Eq. (11.9)] so that the specific energy remains constant throughout the transition. This means that,

$$\Delta z_{i,\,i+1} = h_{L_{i,i+1}} \tag{11.23}$$

Equations (11.19) to (11.23) along with Eq. (11.10) enable the design of an expanding transition as has been illustrated in Example 11.1.

Similarly, for the constant depth condition, $h_f = h_c = h_i$

Hence, one can calculate L, n, B_i and m_i for known value of x using Eqs. (11.19 – 11.22). Thereafter, one can compute the velocity v_i and the head loss $h_{L_{i,i+1}}$ using Eqs. (11.7 and 11.10). Using Eq. (11.9), one can estimate the drop (or rise) $\Delta z_{i,\,i+1}$ in the bed elevation at the known value of x from the flume end.

The method based on variable depth-variable specific energy scheme presupposes the value of Δz_0 (*i.e.,* difference in elevations of canal bed and flume bed) and the transition bed slope is assumed to be constant in the entire length of the transition. Length of transition L, the width of transition B_i and side slope m_i at known value of x (from the flume end) are calculated using the relevant equations discussed above. The depth of flow h_i is determined by trial using energy equation. To start with, the head loss term (unknown since the depth and, hence, velocity are unknown) is neglected and the value of depth of flow h_i is determined from the energy equation. Using this value of the depth of flow one can determine the velocity and the head loss between the adjacent sections and using energy equation, one can compute new value of h_i which is acceptable if it does not differ from the just previous trial value. Otherwise, one should repeat the trial.

11.8. UPLIFT PRESSURE ON TROUGH

In case of siphon aqueducts, the roof of the culvert (or the underside of the trough) is subjected to uplift pressure because the downstream water level is higher than the lower surface of the culvert covering. This uplift pressure head at any point of the culvert covering is equal to the vertical distance between the hydraulic gradient line and the underside of the trough (or the roof of the culvert). The uplift pressure is maximum when the stream is carrying the highest flood. The worst condition for the design of roof covering of the culvert would occur when the highest flood condition coincide with no flow in the canal. Knowing the afflux and the downstream level, the hydraulic gradient line can be drawn as shown in Fig. 11.11. The maximum uplift pressure, obviously, occurs at the upstream end. The thickness of the culvert roof designed to support the load of canal water and its own dead weight would generally be adequate for uplift pressures too as part of the uplift pressure is counterbalanced by the weight of canal water.

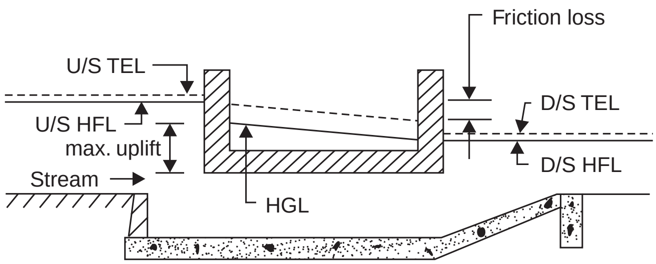

Fig. 11.11 Siphon aqueduct

11.9. UPLIFT PRESSURE ON CULVERT FLOOR

In siphon aqueducts, the culvert floor is subjected to uplift pressure due to: (*i*) the subsoil water (below the stream bed), and (*ii*) the water that seeps from the canal to the stream bed through the embankment (Fig. 11.12).

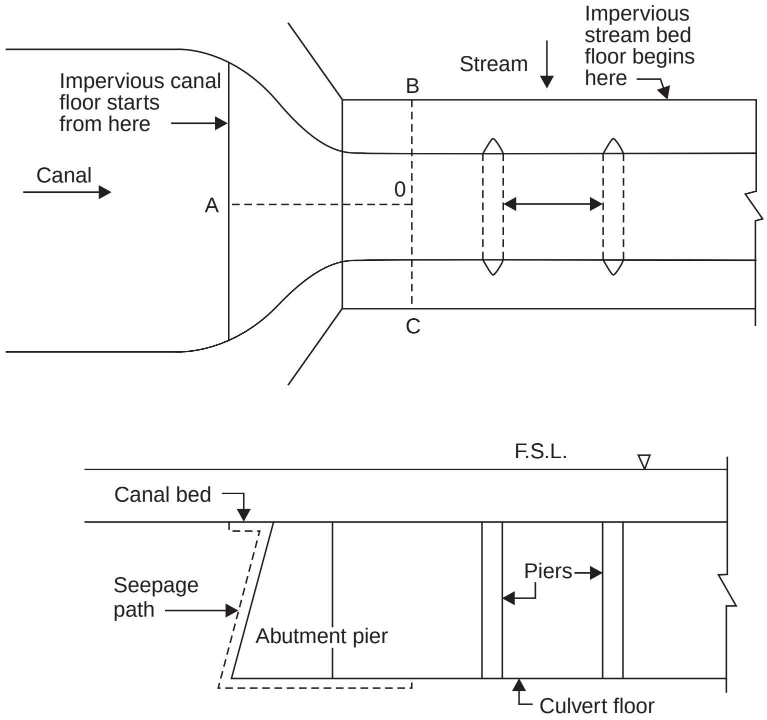

Fig. 11.12 Assumed path of seepage from canal to stream

For some part of a year, the water table below the stream bed may rise up to the bed level itself. This will exert uplift pressure on the floor of the culvert. The maximum uplift will occur when the stream is dry and will be equal to the difference of levels of the stream bed and bottom of the culvert floor.

As shown in Fig. 11.12, the canal water seeps to the stream bed and reappears at *B* and *C* on both sides of the culvert floor. This seepage is, obviously, due to the difference of head between the canal water level and stream water level. The maximum value of the uplift pressure on the culvert floor due to this seepage will occur when the canal is running at FSL and the stream is dry. This seepage flow is three-dimensional in nature and can only be approximately solved by numerical methods or model studies. However, for relatively small structures, the following simple method based on Bligh's creep theory can be used for the design (3). The method can also be used in the preliminary design of major important structures which should, however, be checked by model studies.

Measure the length of the seepage path *AOB* or *AOC* which, in fact, will be equal to the length of the seepage path, shown with dotted lines in elevation, and the distance *OB* (or *OC*). But, for further simplifying the calculation, the seepage length between *A* and *O* is taken as equal to the length of impervious concrete floor on the canal bed up to the flumed end and one-half of the barrel span. If the seepage head, *i.e.,* the difference between the canal FSL and level of the bottom of the culvert floor is H, and the total seepage length is l then the residual head at O, H_0 is equal to $H((l-l_l)/l)$ in which, l_1 is the length of seepage path between A and O, *i.e.,* from the start of the impervious canal floor to the centre of the bottom of the floor of the first barrel. This residual head H_0 and the uplift due to subsoil water are to be counterbalanced by the weight of the culvert floor and, hence, the thickness of the culvert floor can be calculated. In these calculations, it is assumed that the barrels are dry and free of any sediment deposit so that the structure is safe even under the worst condition. However, the resulting floor thickness will be excessive and uneconomical. Besides, higher floor thickness further increases the static uplift pressures. Therefore, the uplift pressure is transferred to the piers by utilising the bending strength of the floor. With such a provision, a floor of reasonable thickness (say, around 0.50 m) would be adequate. Sometimes, inverted arches are also used.

When uplift pressures on the culvert floor are excessive, some additional measures are taken to reduce the uplift. A simple measure would be to extend the impervious floor of the canal on both sides of the trough. Alternatively, the seepage water is made to emerge into the barrel by providing a number of openings (known as relief holes) in the culvert floor. Relief holes are in the form of pipes embedded in the barrel floor. However, the seeping water may bring with it the soil particles also resulting in subsidence of the floor in course of time. This can be prevented by providing inverted filter just below the culvert floor. This inverted filter would consist of one or more pervious layers of soil (inside the embedded pipes serving as relief holes) such that the permeability increases in the upward direction. However, the voids of these pervious layers are not large enough to permit the movement of subsoil particles with the seepage water. Terzaghi's criteria are used for the design of different layers of the inverted filter. These criteria are as follows:

$$\frac{D_{15} \text{ of filter layer}}{D_{15} \text{ of protected soil}} \geq 4$$

and

$$\frac{D_{15} \text{ of filter layer}}{D_{85} \text{ of protected soil}} \leq 4$$

When the canal is dry, there exists a possibility of sediment-laden stream water entering the filter layers through the holes in the culvert floor and, thus, filling the filter voids with sediment. This can be prevented by providing upward-opening flap valves on the top of the relief holes.

11.10. MISCELLANEOUS DETAILS

The exit gradient of the seepage water at the downstream end of the culvert floor must be within permissible limits. Some nominal protection in the form of loose stone may also be provided on the upstream as well as the downstream of the impervious floor.

The foundation of any cross-drainage structure should meet the requirement of the bearing capacity of the soil under optimum loads, seismic effects, anticipated scour, and settlement.

To facilitate drainage of the backfills due to rise in water table or otherwise, small openings are provided in abutments, wing walls, and return walls. These openings are called weep holes.

In big siphons, stop log grooves on the sides of the upstream entrance and downstream outfalls should be provided so as to isolate one or more barrels for periodic inspections, repairs, and maintenance.

Trash racks are desirable at the entrance of the siphon where large quantity of floating material is expected in the channel water. These racks are generally made in panels for convenient handling. These trash racks are usually inclined at a slope of 1(H) : 4(V).

Example 11.1 Design a suitable cross-drainage structure for the following data:

Discharge in the canal,	= 357.0 m^3/s
Bed width of the canal, B_c	= 23.0 m
Side slope of the canal, m_0	= 2.0
Bed level of the canal,	= 267.0 m
Depth of flow in the canal, h_c	= 6.7 m
Bed width of the flume, B_f	= 15.0 m
High flood discharge of the stream	= 500 m^3/s
High flood level of the stream	= 268.0 m
Bed level of the stream	= 265.0 m

Solution: Using Eq. (11.21), the length of expansion transition,

$$L = 2.35\,(B_c - B_f) + 1.65\,m_0 h_c$$
$$= 2.35\,(23 - 15) + 1.65 \times 2 \times 6.7 = 40.91$$
$$\cong 41 \text{ m}$$

Let the transition be divided into 9 sub-reaches of 4.0 m each and the remaining 5 m length be considered as the tenth sub-reach at the end.

Bed width, b is calculated using Eqs. (11.19) and (11.20).

$$n = 0.8 - 0.26\,m_0^{1/2}$$
$$= 0.8 - 0.26\,(2)^{1/2} = 0.432$$

$$\therefore \qquad \frac{B - B_f}{B_c - B_f} = \frac{x}{L}\left[1 - \left(1 - \frac{x}{L}\right)^n\right]$$

$$\text{or} \qquad \frac{B - 15}{23 - 15} = \frac{x}{41}\left[1 - \left(1 - \frac{x}{41}\right)^{0.432}\right]$$

$$\text{or} \qquad B = 15 + \frac{8x}{41}\left[1 - \left(1 - \frac{x}{41}\right)^{0.432}\right]$$

For example, at section 1-1,

$$B_{1-1} = 15 + \frac{8 \times 4}{41}\left[1 - \left(1 - \frac{4}{41}\right)^{0.432}\right]$$
$$= 15.034 \text{ m}$$

Values of B for different values of x have been similarly calculated and are as shown in col. 3 of Table 11.5.

Side slopes at various sections are calculated using Eq. (11.22). Thus

$$\frac{m}{m_0} = 1 - \left(1 - \frac{x}{L}\right)^{1/2}$$

or $$m = 2\left[1 - \left(1 - \frac{x}{41}\right)^{1/2}\right]$$

For example, at section 1-1,

$$m_{1-1} = 2\left[1 - \left(1 - \frac{4}{41}\right)^{1/2}\right] = 0.100$$

Values of m for different values of x have been similarly calculated and are tabulated in col. 4 of Table 11.5.

For constant specific energy condition:

$$E_f = E_c = h_c + \frac{v_c^{\,2}}{2g}$$

and $$v_c = 357/[6.7\,(23 + 2 \times 6.7)] = 1.464 \text{ m/s}$$

$\therefore$ $$E_f = E_c = 6.7 + \frac{(1.464)^2}{2 \times 9.81} = 6.809 \text{ m}$$

$$h_f + \frac{Q^2}{2gB_f^{\,2}h_f^{\,2}} = 6.809$$

or $$h_f + \frac{(357)^2}{2g(15)^2\,h_f^{\,2}} = 6.809$$

Solving for h_f, one obtains $h_f = 6.006$ m

Similarly, $$E_{1-1} = h_{1-1} + \frac{Q^2}{2g(b_{1-1} + m_{1-1}h_{1-1})^2\,h_{1-1}^2} = E_c = 6.809 \text{ m}$$

$\therefore$ $$h_{1-1} + \frac{(357)^2}{2 \times 9.81\,(15.034 + 0.1\,h_{1-1})^2\,h_{1-1}^2} = 6.809$$

On solving this equation, $h_{1-1} = 6.088$ m

Further, $$v_f = \frac{357}{15 \times 6.006} = 3.963 \text{ m/s}$$

and $$v_{1-1} = \frac{357}{(15.034 + 0.1 \times 6.088)\,6.088} = 3.749 \text{ m/s}$$

Using Eq. (11.10), $$h_{L_{f,1-1}} = 0.3 \times \frac{1}{2 \times 9.81}(3.963^2 - 3.749^2) = 0.025 \text{ m}$$

Using Eq. (11.23), $\Delta z_{f,1-1} = 0.025$ m

The positive value of Δz indicates fall in the bed elevation with respect to the flume bed. Values of Δz for other sections can be similarly obtained. The values of Δz and h have been tabulated in cols. 5 and 6 of Table 11.5.

Table 11.5 Designed parameters of expanding transition (Ex. 11.1)

Section	x	*Vittal and Chiranjeevi method*				*Hinds' method*					
		B (m)	m	z	h (m)	Δh (m)	h (m)	z	v (m/s)	m	B (m)
1	*2*	*3*	*4*	*5*	*6*	*7*	*8*	*9*	*10*	*11*	*12*
f-f	0.000	15.000	0.000	0.000	6.006	0.000	6.006	0.000	3.963	0.000	14.999
1	4.000	15.034	0.100	0.025	6.088	0.013	6.019	0.020	3.793	0.100	15.037
2	8.000	15.140	0.206	0.050	6.173	0.053	6.059	0.040	3.614	0.200	15.090
3	12.000	15.325	0.318	0.074	6.255	0.119	6.125	0.061	3.427	0.300	15.172
4	16.000	15.601	0.438	0.096	6.327	0.211	6.217	0.081	3.228	0.400	15.302
5	20.000	15.980	0.569	0.117	6.408	0.330	6.336	0.101	3.017	0.500	15.510
6	24.000	16.482	0.712	0.135	6.463	0.239	6.461	0.121	2.789	0.600	15.935
7	28.000	17.138	0.874	0.152	6.517	0.140	6.560	0.141	2.541	0.800	16.168
8	32.000	18.002	1.063	0.168	6.572	0.067	6.633	0.162	2.266	1.000	17.115
9	36.000	19.196	1.302	0.183	6.613	0.021	6.679	0.182	1.953	1.200	19.350
c-c	41.000	23.000	2.000	0.203	6.700	0.000	6.700	0.207	1.472	2.000	22.795

Steps for the design of the expanding transition using the Hinds' method would be as follows:

Consider the same length of the transition as obtained earlier *i.e.,* 41 m. One could, alternatively, select a suitable length on the basis of minimum splay consideration. Consider also the same condition of constant specific energy so that the velocity and the depth at the flume end are the same as obtained earlier.

Using Eq. (11.14) at the mid-section (*i.e.,* = 41/2 = 20.5 m from the transition end) of the transition where,

$$\Delta h = (6.7 - 6.006)/2 = 0.347 \text{ m}$$

$$\therefore \quad C_1 = \Delta h/x^2 = 0.347/(20.5)^2 = 8.26 \times 10^{-4}$$

Thus, Eq. (11.14) becomes $\Delta h = 8.26 \times 10^{-4}\, x^2$

For section 3-3, x = 12 m (from the flume end)

$$\therefore \quad \Delta h = 0.119 \text{ m}$$

$$\therefore \quad h = 6.006 + 0.119 = 6.125 \text{ m}$$

Similarly, for section 8-8, x = 9 m (from the canal end)

$$\therefore \quad \Delta h = 0.067 \text{ m}$$

$$\therefore \quad h = 6.700 + 0.067 = 6.633 \text{ m}$$

The values of Δh and h for other sections of the chosen sub-reaches of the transition are computed similarly and the values have been tabulated in cols. 7 and 8 of Table 11.5.

Head loss in transition, $h_{L,\,f-c}$ is given by Eq. (11.10),

$$h_{L,f-c} = 0.3\,(v_f^2 - v_c^2)/2g$$

$$= 0.3\,[(3.963)^2 - (1.464)^2]/(2 \times 9.81) = 0.207 \text{ m}$$

For constant specific energy, the change in bed elevation between the two ends of the transition would be equal to the head loss between those ends. Therefore,

$$\Delta z_{f-c} = 0.207 \text{ m}$$

This drop (transition being an expanding one) of 0.207 m is assumed to be linear and the values are listed in col. 9 of Table 11.5. Writing Eq. (11.10) between flume and any section,

$$h_{L,f-i} = 0.3\,(v_f^2 - v_i^2)/2g$$

$$\therefore \qquad v_i^2 = \sqrt{(15.705 - 65.4 \times h_{L,f-i}}$$

Since $h_{L,f-i}$ is equal to Δz_{f-i}, one can compute v_i. The computed values are listed in col. 10 of Table 11.5. The transition width, B at any section can now be computed, using Eq. (11.7), if the side slopes at various sections of the transition are known. These side slopes are to be assumed arbitrarily so that the resulting profile of the transition is smooth and also feasible. For the chosen values of side slope as listed in col. 11 of Table 11.5, the computed values of the transition widths at various sections are as shown in col. 12 of Table 11.5. One needs to adjust the width suitably or try another trial with other set of the values of the side slopes.

Design of Contracting Transition:

The bed-line profile of the contracting transition can be obtained from Eq. (11.3). Adopting a value of m_1 equal to 4, Eq. (11.3) becomes

$$\left(\frac{x}{16}\right)^2 + \left(\frac{y}{4}\right)^2 = 1$$

Length of transition $= 0.5\, m_1(B_c - B_f)$

$= 0.5 \times 4(23 - 25) = 16$ m

Dividing the contracting transition reach into eight sub-reaches, one can determine the values of y and, hence, the bed width $(= B_c - 2y)$ for known values of x measured from the flume end of the transition.

The side slopes and bed elevation may be varied linearly. The computed values of bed width and side slopes are shown in Table 11.6.

Table 11.6 Values of bed width and side slopes of contracting transition (Example 11.1)

x, (m)	*0*	*2*	*4*	*6*	*8*	*10*	*12*	*14*	*16*
y, (m)	4.0	3.97	3.87	3.71	3.46	3.12	2.65	1.94	0.0
$B = B_c - 2y$, (m)	15.00	15.06	15.25	15.58	16.08	16.76	17.70	19.12	23.00
m	0	0.25	0.5	0.75	1.0	1.25	1.5	1.75	2.00

The trough can be divided into three compartments each of width 5.0 m, separated by two intermediate walls 0.30 m thick. The two side walls may be kept 0.60 m thick and 7.3 m high so that a freeboard of 0.60 m is available over 6.7 m depth of flow. For further illustration, let the thickness of the bottom slab be 0.60 m. These dimensions yield the overall width of the trough (*i.e.,* the length of siphon barrel) equal to 15 + (0.3 × 2) + (0.6 × 2) = 16.8 m.

Waterway of the Stream:

Since the bed level of the flumed canal (*i.e.,* 267.203) is below the HFL of the stream (*i.e.,* 268.00), a siphon aqueduct will be suitable.

Lacey's regime perimeter, $P = 4.75\sqrt{Q}$

$= 4.75\sqrt{500} = 106.2$ m

If the clear span of barrel is fixed as 8 m, the pier thickness is obtained as

$$0.55\sqrt{\text{barrel span in metres}}$$

$$= 0.55\sqrt{8} = 1.56 \text{ m} = 1.6 \text{ m (say)}$$

The overall waterway for 12 spans of 8 m each and 1.6 m pier thickness would be

$$12 \times 8 + 11 \times 1.6 = 113.6 \text{ m}$$

The clear waterway is 96 m.

For velocity of flow in the barrels equal to 2.5 m/s, the height of barrel should be

$$\frac{500}{96 \times 2.5} = 2.083 \text{ m}$$

Providing a height of 2.0 m for barrels, the flow velocity in the barrels is

$$\frac{500}{96 \times 2.0} = 2.60 \text{ m/s}$$

For these barrels, hydraulic radius, $R = \dfrac{8 \times 2}{2\,(8+2)} = 0.8$ m

The loss of head through siphon barrels (16.8 m long) can be obtained from Eq. (11.2). Neglecting approach velocity,

$$h_L = \left(1 + f_1 + f_2 \frac{L}{R}\right)\frac{V^2}{2g}$$

where $f_1 = 0.505$, and $f_2 = a\left(1 + \dfrac{b}{R}\right)$

For smooth cement plaster surface, $a = 0.00316$ and $b = 0.03$

$$\therefore \quad f_2 = 0.00316\left(1 + \frac{0.03}{0.8}\right) = 0.00328$$

$$\therefore \quad h_L = \left(1 + 0.505 + 0.00328\frac{16.8}{0.8}\right)\frac{(2.6)^2}{2 \times 9.81} = 0.54 \text{ m}$$

$\therefore$ Upstream HFL = 268 + 0.54 = 268.54 m

Uplift pressure on the barrel roof (or flume trough):

Consider the downstream end of the flume and transition designed using Vittal and Chiranjeevi's method.

For the chosen thickness of the trough slab as 0.6 m and the computed value of Δz as 0.203 m, R.L. of the bottom of the trough (*i.e.,* culvert roof) slab

$$= 267.0 + 0.203 - 0.6$$

$$= 266.603 \text{ m}$$

Loss of head at the entry of the barrel $= 0.505\dfrac{V^2}{2g} = 0.505\dfrac{(2.6)^2}{2g} = 0.174$ m

and velocity head in the barrel $= \dfrac{V^2}{2g} = \dfrac{(2.6)^2}{2g} = 0.345$ m

Part of the hydraulic head available at the upstream end of the barrel, Fig. 11.11, is utilized in meeting the entry (to the barrel) loss and developing the velocity head in the barrel. Therefore, maximum uplift (at the upstream end) pressure on the trough slab (Fig. 11.11)

$$= \text{HFL} - \text{Entry loss} - \text{velocity head in the barrel} - \text{RL of the bottom of the trough}$$

$$= 268.54 - 0.174 - 0.345 - 266.603$$

$$= 1.418 \text{ m}$$

Uplift pressure on the floor of the barrel :

$$\text{RL of barrel floor} = 266.03 - 2.0 = 264.603 \text{ m}$$

Let the floor thickness of the barrel be 1.5 m

$\therefore$ RL of the bottom of the barrel floor $= 264.603 - 1.5 = 263.103$ m

$\therefore$ Static pressure on the barrel floor $= 265 - 263.103 = 1.897$ m

The culvert floor should extend toward the upstream by a distance equal to the difference between HFL and the culvert floor level, *i.e.,* $268 - 264.603 = 3.397 \text{ m} \cong 3.5$ m (say)

Length of upstream contracting transition (of canal) = 16 m

Half of the barrel span = 4 m

End of the culvert floor from the centre of the barrel = 3.5 + (16.8/2) = 11.9 m

Total seepage head (at the upstream end of the culvert floor)

$$= \text{FSL of canal (in the flumed portion)} - \text{Bed level of the stream}$$

$$= (267.203 + 6.006) - 265.0 = 8.209 \text{ m}$$

Therefore, residual seepage head at the centre of the barrel (calculated approximately)

$$= 8.209 \times \frac{[31.9 - (16 + 4)]}{(16 + 4 + 11.9)}$$

$$= \frac{8.209 \times 11.9}{31.9} = 3.06 \text{ m}$$

Therefore, total uplift at the bottom of the culvert floor = 1.897 + 3.06 = 4.957 m

As in case of other hydraulic structures, suitable wing connections and protection works on both the upstream and downstream sides of the canal and the stream at the site of the structure are to be provided.

EXERCISES

11.1 What is the purpose of providing a cross-drainage structure ? In which reach of a canal, does the need of such structures arise and why ?

11.2 Explain how the type of a cross-drainage structure may change with the shift in the site of the structure along the stream.

11.3 Design a siphon aqueduct across a stream for the following data:

Canal

Full supply discharge $= 56 \text{ m}^3/\text{s}$

Bed width = 32 m

Depth of flow	= 2.0 m
Bed level	= 267.0 m
Stream	
High flood discharge	= 425 m^3/s
High flood level	= 268.20 m
Bed level	= 265.50 m
General ground level	= 267.20 m

Make suitable assumptions, if required. Draw plan and sectional elevation of the designed siphon aqueduct.

11.4 The following are the data for a canal siphon:

Canal	
Fully supply discharge	= 32 m^3/s
Full supply level	= 262.00 m
Bed level	= 260.00 m
Bed width	= 22.00 m
Side slope	= 1.5 (H) : 1 (V)
Stream	
Design flood discharge	= 500 m^3/s
High flood level	= 263.50 m
Drainage bed level	= 261.00 m
Silt factor for stream bed	= 1.50
Other given data are as follows:	
Permissible velocity through siphon barrels	= 2.5 m/s
Manning's roughness coefficient for barrel friction	= 0.018
Barrel entrance loss coefficient	= 0.25
Thickness of barrel walls	= 0.30 m
Hinds' contraction coefficient	= 0.25
Expansion coefficient	= 0.35

Assume any other data suitably. Work out the following items of design:

(*a*) Barrel size and its vertical location,

(*b*) TEL, WSL, and BL at the six sections along the canal, and

(*c*) Total uplift force on barrel roof.

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12

RIVERS AND RIVER TRAINING METHODS

12.1. GENERAL

Rivers have always played an important role in human development and in shaping civilisations. Primary function of a river is the conveyance of water and sediment. Besides serving as a source of water supply for domestic, irrigation, and industrial consumption, rivers have been useful in providing facilities for navigation, recreation, hydropower generation, and waste disposal. Rivers, except when flowing through well-defined narrow sections confined by high and stiff banks, have also generally caused problems of flooding, change of course, banks erosion etc.

The structure and form of rivers including plan-forms, channel geometry (*i.e.,* cross-sectional shape of river), bed form, and profile characteristics together form what is termed river morphology. The morphology of river changes considerably on account of natural causes. Besides, changes made by man in an attempt to harness a river strongly influences behaviour of the river.

12.2. CLASSIFICATION OF RIVERS

Rivers can be classified as follows:

(*i*) Based on variation of discharge in river, as
- (*a*) Perennial rivers,
- (*b*) Non-perennial rivers,
- (*c*) Flashy rivers, and
- (*d*) Virgin rivers.

(*ii*) Based on stability of river, as
- (*a*) Stable rivers,
- (*b*) Aggrading rivers, and
- (*c*) Degrading rivers.

(*iii*) Based on the location of reach of river, as
- (*a*) Mountainous rivers,
- (*b*) Rivers in flood plains,
- (*c*) Delta rivers, and
- (*d*) Tidal rivers.

(*iv*) Based on the plan-form of river, as

(*a*) Straight rivers,

(*b*) Meandering rivers, and

(*c*) Braided rivers.

12.2.1. Perennial Rivers

Perennial rivers obtain their water from melting snow for the larger part of any year besides getting rain water during the rainy season. Being snow-fed, perennial rivers carry significant flow all through the year.

12.2.2. Non-perennial Rivers

Non-perennial rivers are not snow-fed rivers and, hence, get completely dried up or carry insignificant flow during the summer season. They get their supplies only during the monsoon as a result of rains in their catchment areas.

12.2.3. Flashy Rivers

In case of flashy rivers, the river stage rises and falls in a very short period of a day or two due to the steep flood hydrograph. A small flow may, however, continue for sometime.

12.2.4. Virgin Rivers

In arid regions, waters of some rivers may get completely lost due to evaporation and percolation. Such rivers become completely dry much before they join another river or sea, and are called virgin rivers.

12.2.5. Stable Rivers

When the alignment of a river channel, river slope, and river regime are relatively stable and show little variation from year to year except that the river may migrate within its permanent banks (*i.e., khadirs*) (Fig. 12.1), the river is said to be stable. However, changes in bed and plan-forms of a stable river do take place, but these are small.

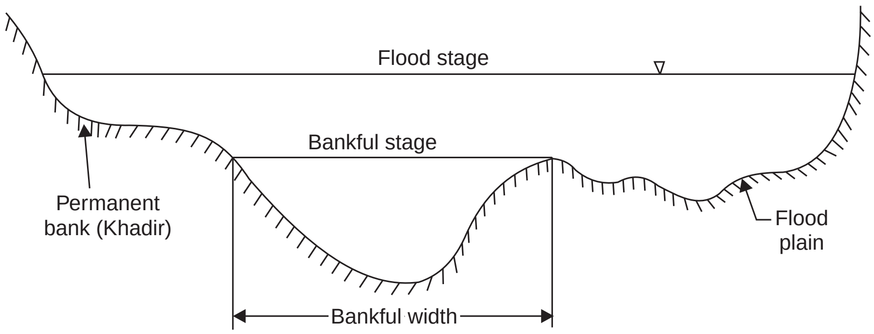


Fig. 12.1 Typical cross-section of river in flood plains

12.2.6. Aggrading Rivers

When the sediment load entering a river reach is greater than the sediment load leaving a river reach, the river in that reach becomes an aggrading river due to deposition of excess sediment. This situation may arise due to obstructions (*e.g.,* barrage or dam) across a river, extension of delta at the river mouth, or sudden intrusion of sediment from a tributary.

Aggrading rivers usually have straight and wide reaches with shoals in the middle which shift with floods. The flow in the river channel gets divided into a number of braided channels.

12.2.7. Degrading Rivers

When the sediment load entering a river reach is less than that leaving the river reach, the river in that reach becomes a degrading river due to erosion of the bed and bank material.

12.2.8. Mountainous Rivers

Rivers in mountainous reaches are further divided into incised rivers and boulder rivers. Incised rivers have a steep bed slope and high velocity of flow. The bed and the banks of these rivers are made up of rocks and very large boulders which are, usually, highly resistant to erosion. The sediment transported by an incised river is often different from that of the river bed and comes from the catchment due to soil erosion.

The bed and sides of a boulder river consist of a mixture of boulder, gravel, shingle, and sand. The bed slope and the velocity of flow are smaller than those of incised rivers. The river cross-section is usually well-defined. There is, however, considerable subsoil flow due to high permeability of the bed material.

12.2.9. Rivers in Flood Plains

After the boulder stage, a river enters the alluvial plains. The bed and banks are now made up of sand and silt. The bed slope and the velocity of flow in the river are much smaller than those of boulder rivers. The cross-section of the river is decided by the sediment load and the erodibility of the bed and banks of the river. A typical cross-section of a river with a flood plain is shown in Fig. 12.1. The sediment transported by such rivers is predominantly of the same type as the material forming the channel bed. During high floods, these rivers inundate very large areas and cause considerable damage to life, property, and crops. Such rivers are also called *alluvial rivers*.

12.2.10. Tidal Rivers

All rivers ultimately meet the sea. In the reach of a river just upstream of the sea, there would occur periodic changes in water levels due to tides. This reach of the river is called tidal river and receives sea water during flood tides and raises its level. During ebb tides, the river water level is lowered. The length of the river reach affected by tidal effects depends on the river slope, the tidal range, discharge, river configuration *etc*.

12.2.11. Delta Rivers

A river, before becoming a tidal river, may split into number of branches due to very flat bed slopes resulting in shoal formation and braiding of the channel. This part of the river reach is called delta river. The delta river indicates a stage, rather than a type of river.

12.2.12. Straight Rivers

In the straight reaches of a river, its section has the shape of a trough and maximum velocity of flow occurs in the middle of the section. It is very difficult to find the straight reach of an alluvial river over large lengths. Alluvial rivers seldom run straight through a distance greater than ten times the river width (1). Even in the apparent straight reaches, the line of maximum depth - commonly known as *talweg* – moves back and forth from one *khadir* (permanent bank) to another *khadir*.

12.2.13. Meandering Rivers

On account of the slight asymmetry of flow in alluvial rivers, there is a tendency for such rivers to vary their plan-forms into bends which eventually result in a meandering pattern (Fig. 12.2). The term meandering has been derived from the Great Menderes river in Turkey which follows a winding or intricate course (Fig. 12.2). Rivers having such meandering patterns are known as meandering rivers which, in plan, comprise a series of bends of alternate curvature. The successive curves are connected through straight reaches of the river called 'crossing'. Meandering increases the length of river and decreases its slope.

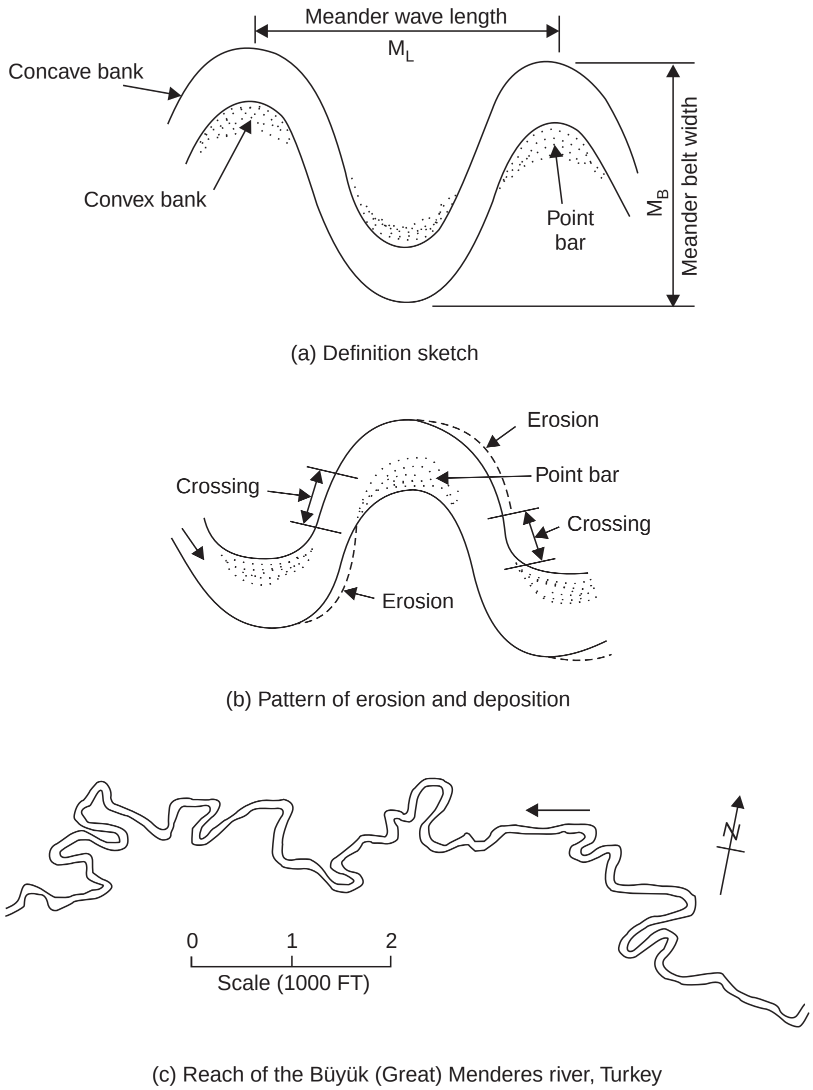


Fig. 12.2. Meandering channel

12.2.14. Braided River

When a river flows in two or more channels around alluvial islands, it is called a braided river (Fig. 12.3). The braided patterns in a river develop after local deposition of coarser material which cannot be transported under prevailing conditions of flow and which subsequently grows into an island consisting of coarse as well as fine material.

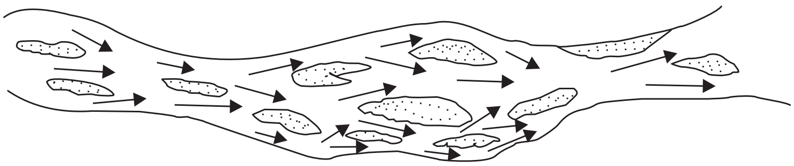


Fig. 12.3 Typical braided reach of a river

12.3. BEHAVIOUR OF RIVERS

The behaviour of a river is mainly affected by the characteristics of the sediment-laden water flowing in the river. The available energy of the flow is utilised in transporting the sediment load as well as in overcoming the resistance due to the viscous action and the roughness of bed and sides. On account of the interdependence of the factors affecting the flows, there is an inherent tendency of these rivers to attain equilibrium. As such, whenever the equilibrium of a river is disturbed by man-made structures or natural causes, the river tends to attain a new equilibrium condition by scouring the bed or by depositing the sediment on the bed or by changing its own plan-form. These changes can be either local or extended over a long reach. The behaviour of a river can, therefore, result in the variation of the shape of the river cross-section and/or its plan-form. Aggradation, degradation, scour and deposition of sediment around bends, and meandering are a few examples of such changes.

12.3.1. Bends

With slight asymmetry in flow, an alluvial river tends to develop bends which are characterised by scour and erosion of sediment on the concave (*i.e.,* outer) bank and deposition of sediment on the convex (*i.e.*, inner) bank. Because of curved flow lines around the bend, the flow is subjected to centrifugal forces and, hence, there is a transverse slope of the water surface due to the superelevation of the water surface at the concave bank. As a result, the bottom water (moving with relatively smaller velocity) moves from the concave bank to the convex bank and also carries with it the bed material and deposits it near the convex bank. To replace this bottom water, water dives in from the top at the concave bank and flows along the bottom carrying sand and silt to the convex bank where it is deposited. This secondary motion is primarily responsible for the erosion of the sediment on the concave bank and the deposition of the sediment on the convex bank. The depth of flow in a river at the bend thus becomes deeper at the concave bank (Fig. 12.4).

12.3.2. Meanders

The continued action of the secondary flow developed around river bends causes further erosion and deposition of the sediment, respectively, on the concave and convex banks of the river. Thus, the river bends become more sharp and the river attains a meander pattern and becomes a meandering river. Meander patterns are usually associated with wide flood plains comprising easily erodible material.

There have been several attempts to explain the mechanism of meander development. According to Inglis (2), "Meandering is nature's way of damping out excess energy during a wide range of varying flow conditions, the pattern depending on the grade of material, the relation between discharge and charge (sediment load), and the rates of change of discharge and charge". Thus, a channel having excess energy attempts to increase its length by meandering thereby decreasing its slope.

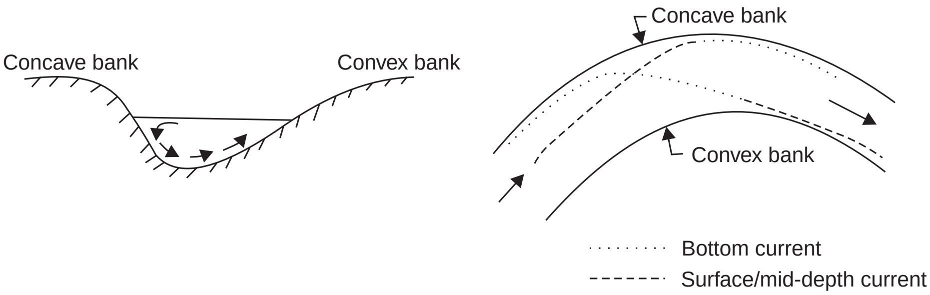


Fig. 12.4 Movement of water at a bend

Joglekar (3) and other Indian engineers do not agree with the theory of excess energy. According to them (3), 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, accentuating the curvature of flow and finally producing meanders in its wake.

Meanders can be classified (4) as regular and irregular or, alternatively, as simple and compound. Regular meanders are a series of bends of approximately the same curvature and frequency. Irregular meanders are deformed in shape and may vary in amplitude and frequency. Simple meanders have bends with a single radius of curvature. In compound meanders each bend is made up of segments of different radii and varying angles.

The geometry of meanders can be described by the meander length M_L and the width of the meander belt M_B (Fig. 12.2), or by the sinuosity or the tortuosity. Many investigators have attempted to relate the geometry of meanders with the dominant discharge. The *dominant discharge* is defined (5) as that hypothetical steady discharge which would produce the same result (in terms of average channel dimensions) as the actual varying discharge. Inglis (2) found that for north Indian rivers, the dominant discharge was approximately the same as the bankful discharge and recommended that the dominant discharge be taken as equal to half to two-thirds of the maximum discharge.

Inglis (2) gave the following relationships for M_L and M_B (both in metres) in terms of the dominant discharge (or the bankful discharge) Q (in m^3/s) for rivers in flood plains:

$$M_L = 53.6\ Q^{1/2} = 6.06\ W_s \tag{12.1}$$

$$M_B = 153.4\ Q^{1/2} = 17.38\ W_s \tag{12.2}$$

Here, W_s (in metres) is the bankful width of river (Fig. 12.1).

Agarwal *et al*. (6) have re-examined the laboratory and field data for discharges ranging from 9×10^{-6} to 10^4 m^3/s and found that the following relationships proposed by them are better than Inglis' relationships:

$$M_L = 29.70\, Q^{0.32} \quad \text{for } Q < 9 \text{ m}^3\text{/s} \tag{12.3}$$

$$M_L = 11.55\, Q^{0.75} \quad \text{for } Q > 9 \text{ m}^3\text{/s} \tag{12.4}$$

and

$$M_B = 0.476\, M_L \tag{12.5}$$

The sinuosity of a river is defined as the ratio of *talweg* length to the valley length. Joglekar (3) defines tortuosity as,

$$\text{Tortuosity} = \frac{\text{talweg length} - \text{valley length}}{\text{valley length}} \times 100$$

The sinuosity varies from 1.02 to 1.45 over 1500 km length of the river Indus. For the river Ganga, the sinuosity varies from 1.08 to 1.51 (3). Table 12.1 shows the extent of tortuosity of the river Ganga.

Table 12.1 Tortuosity of the river Ganga in different reaches (3)

Reach	*Valley length (km)*	*Talweg length (km)*	*Tortuosity (%)*
Balawalli to Garhmukteshwar	104.6	117.5	12
Garhmukteshwar to Rajghat	59.5	67.6	14
Rajghat to Kanpur	281.6	313.8	11
Kanpur to Allahabad	117.0	217.3	23
Allahabad to Varanasi	137.8	209.2	51
Varanasi to Sara	684.0	869.0	27
Sara to the Bay of Bengal	297.7	321.9	8

It should be noted that the meandering pattern of alluvial rivers is commonly encountered in alluvial stream. River training methods are generally adopted for meandering rivers. It may also be noted that the meander pattern is not stationary and moves slowly in the downstream direction.

12.3.3. Cutoffs

Cutoffs can be defined (3) as a process by which an alluvial river flowing along curves or bends abandons a particular bend and establishes its main flow along a comparatively straighter and shorter channel. During the development of meanders, there is always a lateral movement of the meanders due to their gradual lengthening. Increased frictional losses and bank resistance tend to stop this lateral movement. When the bend and the bank resistance become too large for continued stretching of the loop, the flow finds it easier to cut across the neck than to flow along the loop (Fig. 12.5). This results in a cutoff. Cutoff is, thus, a natural way of counter-balancing the effect of the ever-increasing length of a river course due to the development of meanders. Usually, a river has shallow side channels within the neck of the meander loop. These side channels may either be part of the main channel of an earlier river course or are formed by floods spilling over the banks of the river channel. Cutoffs can develop along these shallow side channels. Alternatively, cutoff may be artificially induced for some other purpose.

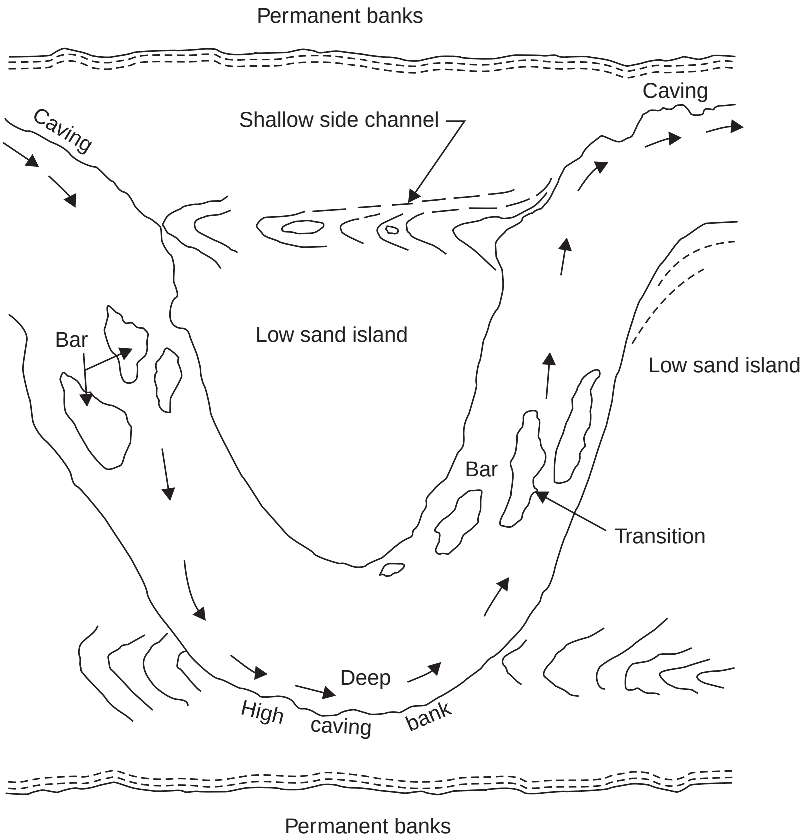


Fig. 12.5 A river bend

The rapidity with which a cutoff channel develops depends on local conditions. For example, the cutoff on Chenab near Shershaha took one year but the great Golbethan bend of the Ganga downstream of Hardinge Bridge began to cutoff in 1911, and the cutoff developed only after five years (7).

Whenever a river succeeds in establishing a cutoff, there follows a period of non-equilibrium for long distances upstream and downstream of the newly-formed channel. Banks start caving in and new channels are formed while some other channels get silted up. Only after a couple of floods, the equilibrium is, once again, established.

Sometimes it is advantageous to make a controlled artificial cutoff to avoid the chaotic or non-equilibrium conditions when a natural cutoff develops. An artificial cutoff reduces flood levels and flood periods. Artificial cutoffs have been used to shorten the travel distance and increased ease of manoeuvring of boats along the bend during navigation. In such situations, use of training measures like groynes and revetment on banks usually becomes necessary to prevent bank erosion and arrest the natural tendency of the river to meander.

For inducting an artificial cutoff, a suitable pilot cut (or pilot channel) of small cross-section is initially made so as to carry 8 to 10 per cent of the flood discharge (5). The pilot channel is then allowed to develop by itself and sometimes such gradual development is assisted by dredging. Pickles (8) has made the following recommendations for design and execution of artificial cutoffs:

(*i*) The pilot channel should be tangential to the main direction of river flow approaching and leaving the cutoff.

(*ii*) The pilot channel is usually made on a mild curve, the curvature being less than the dominant curvature of the river itself.

(*iii*) Entrance to the pilot channel is made bell-mouthed. Such transition at the exit is considered unnecessary because the cut develops first at the lower end and works progressively upstream.

(*iv*) The cut, when unlikely to develop because of either coarseness of the material or low shear stress, should be excavated to average river cross-section.

(*v*) The width of the pilot cut is unimportant as the cut ultimately widens due to scouring. Hence, in practice, the width is determined by consideration of the type and size of the dredging equipment used.

(*vi*) When a series of cutoffs is to be made, the work should progress from the downstream to upstream.

12.3.4. Lateral Migration of Alluvial Rivers

Some alluvial rivers have shown a tendency for lateral migration over a period of years. Some rivers, which might have had very little lateral movement over a long period of time, may suddenly start migrating laterally during a succeeding period. Such changes in river courses have been found in the Yellow river, and in other rivers of China (5). Lateral movement of the Kosi river in India offers a classic example of the lateral migration of alluvial rivers (3). The Kosi river brings in a large amount of sediment load which it is unable to transport. In the process of building up an inland delta in the plains, it shifted over 110 km westward during the period from 1736 to 1964 (3). However, it did not shift from its original position at the Belka hills (near Chatra where the river leaves the Himalayas and enters the Gangetic plains) and also at Kursela (close to the confluence of the Kosi and the Ganga rivers). The lateral migration of the Kosi river was arrested by the construction of levees. Table 12.2 gives the average rates of lateral migration of the Kosi river for the period 1736-1950.

Observations of the migration of the Kosi river and the Yellow river indicate that the lateral migration of alluvial rivers is mainly due to the excessive sediment load during the floods, large variation in the river discharge, large slope, and geologically young rock formations (5).

Table 12.2 Average rates of migration of the Kosi river (9)

Period	*Period of Movement (years)*	*Approximate distance moved (km)*	*Rate of movement (km/year)*
1736-1770	34	10.8	0.32
1770-1823	53	9.3	0.18
1823-1856	33	6.1	0.18
1856-1883	27	12.9	0.48
1883-1907	24	18.5	0.77
1907-1922	15	10.9	0.73
1922-1933	11	29.0	2.63
1933-1950	17	17.7	1.04

12.3.5. Delta Formation

As a river approaches the sea, it dumps its sediment load into it and the river mouth extends towards the sea on account of the sediment deposition. This lengthening of the river further reduces the already small slope of the river at its mouth. Thus, the sediment transporting capacity of the river decreases and the river deposits the sediment on the bed and banks of the river channel raising the river stage. In general, the rise in the river stage results in spilling over the banks and cutting through the banks if they are not sufficiently resistant. The spilled water may form branch channels or spill channels which, after their full development, start behaving like the parent river. As a result, the sediment transport capacity decreases considerably and the river bed starts rising resulting in the formation of delta (Fig. 12.6).

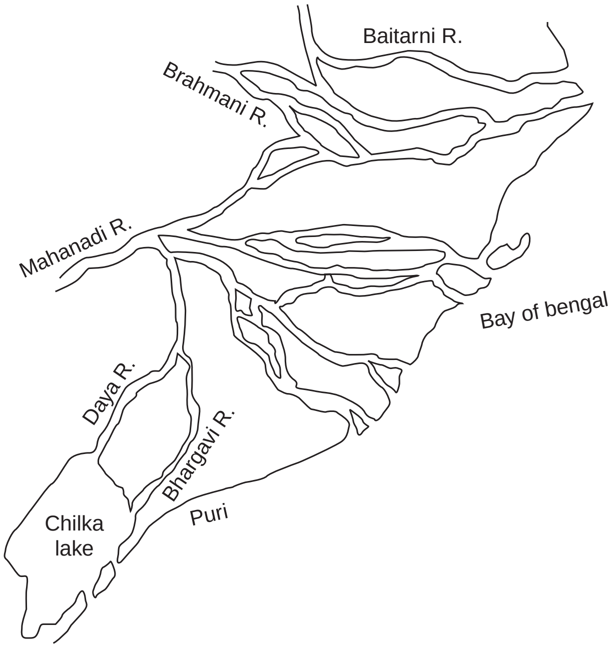


Fig. 12.6 Delta formation of the Mahanadi river

12.3.6. Aggradation

When the sediment load in a river is in excess of its sediment-transporting capacity, the excess sediment gets deposited on the river bed. Therefore, the sediment load entering a given reach is greater than the sediment load leaving the reach during the same time. This causes a rise in the bed level (and, hence, the flood level) and increase in the bed slope with time. The phenomenon itself is known as aggradation. Aggradation usually occurs because of an increase in the sediment load at a section without change in discharge and sediment size. Aggradation occurs most commonly upstream of a reservoir. Because of the construction of a dam, the sediment-transporting capacity of the river upstream of the reservoir is reduced. The coarser sediment deposits farther away from the dam while the finer sediment is deposited closer to the dam. Aggradation can also occur when a tributary brings into the main river sediment load in excess of the river's sediment-carrying capacity. Also, when rivers are divided into two or more branches, as in the case of deltaic rivers, it may not be possible for every branch to maintain the state of equilibrium in respect of sediment flow, and, hence, aggradation may result. If an offtaking canal takes relatively sediment-free water, it would result in aggradation in the main river downstream of the offtake point.

12.3.7. Degradation

When the amount of sediment load being transported by a river is less than the sediment-transporting capacity of the river, the excess sediment needed to satisfy the capacity of the river will be eroded from the erodible bed, thereby lowering the bed. If the banks are also erodible, widening of the river would also result. This phenomenon of erosion of bed and banks is known as degradation. Degradation occurs in streams downstream of a reservoir. The reservoir stores a large percentage of the sediment load carried by the river. The flow downstream of the reservoir is relatively free of sediment, and, therefore, cause degradation resulting in lowering of the bed levels.

An interesting example of large bed level changes is that of degradation on the Ratmau torrent downstream of a level crossing on the Upper Ganga canal in UP (10). The flood discharge in the Ratmau torrent has been of the order of 1000 m^3/s with a maximum of 2250 m^3/s in 1947. The level crossing was constructed in 1850 to pass the torrent across the canal carrying a discharge of 300 m^3/s. During the period from 1854 to 1977, the bed level of the torrent immediately downstream of the level crossing was lowered by 8.0 m. The bed slope of the torrent downstream of the crossing has decreased from 1.56×10^{-3} to 5.5×10^{-4}. The lowering of the bed required frequent modifications in the stilling basin (10).

If degradation has occurred in a river near its confluence with a tributary, the tributary slope increases. Thus the sediment-transporting capacity of the tributary increases near the confluence, resulting in degradation near the confluence and this effect extends further upstream and also to other sub-tributaries. Degradation can also occur if the water discharge in a river is increased without increase in sediment load or sediment size. If the sediment-free waste water of an irrigation project is added to a river, it may result in degradation in the river.

Downstream of a spillway, a hydraulic jump is usually formed to dissipate the excess energy. Due to lowering of the bed and water levels on account of degradation, jump may shift downstream of the apron and endanger its safety. Dams on pervious foundations are subjected to uplift pressure which depends on the effective head which is equal to the difference between the reservoir and tail-water levels. The lowering of the tail-water level due to degradation increases the effective head and, hence, the uplift pressure on the dam. Lowering of water level at the intakes on account of degradation may make the diversion of water for irrigation more difficult. Lowering of bed in navigable rivers due to degradation may, at times, make the navigation locks inoperative.

Degradation can sometimes be beneficial too. Because of lowering of tail-water levels, the effective head increases. This would cause an increase in hydro-electric power generation. Degradation increases the river capacity to carry the flood flow. This lowers the high flood levels of a river. Lowering of the water level in a river on account of degradation lowers the ground water table in the adjoining areas.

12.4. RIVER TRAINING

River training includes all such measures as are taken for controlling and regulating river flow and river configuration. River training works are constructed either across a river, or along it. River training structures include levees or embankments built along the river to contain floods, and spurs and guide banks are constructed for altering the local flow conditions and guiding the flow. Besides, a river can be dredged to train it for navigation purposes. A river can also be trained by diverting its flow into a secondary channel or by executing artificial cutoffs on the main river so as to cause reduction in flood levels. Bank protection measures are also included in river training methods.

12.4.1. Objectives of River Training

River training measures aim at achieving one or more of the following objectives:

(i) Flood Protection

River floods of very small frequency inundate the fertile and thickly-populated plains adjacent to the river, and, thus, cause considerable loss to human life, property, agriculture, and public and private utilities. Statistics collected by the Central Water and Power Commission for flood damage in India during 1953-63 have been given in Table 10.3 to show the extent of loss due to floods.

Table 12.3 Average annual flood losses in India during 1953-63 (11)

Population affected	11.68 million
Total area affected	64.4 million hectares
Cropped area affected	20.2 million hectares
Human lives lost	518
Total damage to crops and utilities	Rs. 684.0 million

During the years of large floods, damage is likely to be several times more. Flood control measures for thickly-populated flood plains, therefore, become essential, even if these measures do not assure complete protection under all conditions. River training for flood protection, also known as 'high water training' or 'training for discharge,' is achieved by one or more of the following four methods :

(*a*) Construction of levees or embankments to confine water in a narrower channel,

(*b*) Increasing the discharge capacity of natural channels by some means such as straightening, widening or deepening,

(*c*) Provision of escapes or diversion from the main channel into an auxiliary channel for water in excess of the carrying capacity of the main channel, and

(*d*) Construction of reservoirs.

(ii) Navigation

For a river to be navigable, sufficient depth and width required for navigation should be available even at low water level in the river. River training for navigation is also known as 'low water training' or 'training for depth'. Measures to achieve adequate depth in a river for navigation include dredging the shallow reaches of the river and using spurs to contract the river channel, thus, increasing its depth. Sometimes, low flow is supplemented from another source to achieve the desired depth and width. Canalisation makes a non-navigable river navigable, and, is accomplished by building a series of small dams or weirs and locks. Sharp curves along the river need to be eliminated so that ships can move easily.

(iii) Sediment Control

River training for sediment control is also called 'mean water training' or 'training for sediment'. This type of training aims at rectification of river bed configuration and efficient movement of sediment load for keeping the channel in a state of equilibrium (3). River training methods for this purpose involve construction of such structures which would induce the desired local curvature to the flow. Spurs and pitched islands are normally used for training the river for sediment.

(iv) Guiding the Flow

Hydraulic structures, such as canal headworks, and communication structures such as bridges, have to be protected against outflanking and the direct attack of flow. This requires training of the river over its considerable reach by building a system of guide banks, known as Bell's guide banks, on one or both sides of the stream at the bridge site. The purpose of these guide banks is to make sure that water flows between the abutments of the bridge. The spacing between these guide banks conforms to the width required for the river to pass the design flood discharge. Similarly, guide banks are provided to guide the flow at the weir site. Marginal bund and lateral spurs guide the flow through the guide banks.

Sometimes the flow in a river needs to be deflected away from a bank in order to protect some portions of the river bank or for contracting the river. This is done by constructing one or more spurs projecting into the river from its banks.

(v) Stabilisation of River Channel

Weak river banks, which are likely to cave in or get eroded, need to be protected by training methods, such as stone pitching, lining, and so on. In some cases, the stability of the bed may also be endangered in some reaches due to increase in the bed shear on account of local flow conditions.

12.5. RIVER TRAINING METHODS

The planning and design of river training structures is accomplished by using empirical methods and reliance has to be placed on the intuition and judgement of experienced engineers. Model investigations are also resorted to for finalising the plans and design of river training structures. Commonly used methods of river training have been briefly described in the following sections.

12.5.1. Levees

A levee (also known as an embankment, bund, marginal bunds or dike) is an embankment running parallel (or nearly so) to the river and is constructed to protect the area on one side of it from flooding. The method of constructing levees on one or both sides of a river to contain the flood within the leveed portion is the oldest and most commonly used method of flood control. Levees along the Nile river in Egypt were constructed prior to 600 BC. Levees have been constructed recently on many important rivers of the world such as the Ganga, the Kosi, the Mahanadi and the Gandak in India, the Yellow, the Pearl, the Yangtze and the Huai in China, the Mississippi in the USA, and the Danube and the Rhine in Europe.

The alignment of levees for a river is decided by the location of important cities, industries, and other areas along the river which need to be protected against floods. Closely-spaced levees will be very high and, hence, massive and uneconomic. Hence, levee spacing is also governed by economic considerations. Levees should be located farther apart considering: (*i*) the desirability of having high discharge capacity of river for a given stage, and (*ii*) the requirement that the entire meander belt be within the levees so that they are not strongly attacked by the river. The levees should, obviously, have the general curvature of the river so that the river does not attack the levees.

The design of a levee is similar to that of an earth dam. It should, however, be noted that while the upstream face of an earth dam is exposed to water most of the time, that of a levee is exposed to water for a very short period during the flood season only. The top width of a levee is generally kept between 3 to 8 m or more depending upon the levee height. The levee height is decided such that the levee is able to contain a flood of a reasonable return period of, say about 500 years (5). The flood stage at any section of a river corresponding to such a flood can be obtained by routing the flood through the river. A freeboard of 1-2 m is added to the flood stage to obtain the elevation of the top of the levee. The probable settlement of levee after its construction should also be accounted for while determining the levee height. The side slopes of levees vary from $1V : 2H$ to $1V : 6H$. In case of high levees, berms are also provided on the land-side slope.

One of the major effects on regime of river due to levee construction is the reduction in the river width and, hence, increase in velocity of flow. As such, the sediment, which would have deposited on the river bed/flood plains in the absence of levees, is now carried downstream and deposited either in an unleveed portion or in the sea. Other effects of confining the flood within levees are as follows (12):

(*i*) Increase in the rate of travel of flood wave in the downstream direction,

(*ii*) Rise in the water surface elevation in the river during flood,

(*iii*) Reduction of storage and, hence, an increase in the maximum discharge downstream, and

(*iv*) Decrease in the water surface slope of the stream above the leveed portion as a result of which aggradation occurs upstream of the leveed reach.

Failure of levees can be due to one or more of the following causes (12):

(*i*) Overtopping,

(*ii*) Erosion of riverside slope by river current,

(*iii*) Caving in of the banks,

(*iv*) Infiltration through the foundation,

(*v*) Infiltration through the embankment,

(*vi*) Leaks as a result of holes dug by rats, crabs, and white ants, or from rotten roots and cracks due to shrinkage of soil,

(*vii*) Loosening of the embankment by wind action on large trees planted on it, and

(*viii*) Human action.

As such, levees are very susceptible to failure during floods. Hence, continuous supervision, particularly during floods, and availability of enough labour and material on the spot are necessary to detect and plug breaches in the levees, if they occur.

The method of flood control by levees is fairly simple and economical as it uses locally available material and labour for its construction. Besides, levees can also be extended gradually to cover more and more area.

12.5.2. Spurs

Spurs (also known as groynes, spur dikes, or transverse dikes) are structures constructed in a river transverse to the river flow, extending from the bank into the river (3). Spurs guide the river flow, promote scour and deposition of the sediment where desired, and trap the sediment load to build up new river banks. Spurs are generally made from locally available earth. The nose (or head) and the sloping faces of the spurs must be protected against wave action by hand-placed rubble facing. Stone apron is provided to prevent the failure of spurs due to excessive scour at the nose and sides. Spurs are probably the most widely used river training structures and serve the following function in river regulation:

(*i*) Training a river along the desired course by attracting, deflecting or repelling the flow in the river channel,

(*ii*) Creating a slack flow with the object of silting up the area in the vicinity of spur,

(*iii*) Protecting the river bank by keeping the flow away from it, and

(*iv*) Contracting a wide river channel for the improvement of depth for navigation.

Spurs can be used either singly or in series or in combination with other river training measures. The design of spur depends on the following:

(*i*) River discharge,

(*ii*) Angle of attack,

(*iii*) Sediment load,

(*iv*) Meander length,

(*v*) Curvature of the river, and

(*vi*) Upstream and downstream river training measures.

Spur length is usually restricted to less than 20% of the river width to avoid adverse effects on the opposite bank and, at the same time, the spur length is kept longer than 1.5 to 2 times the depth of flow (13). Shorter spur length in deeper rivers induces swirling motion on both the upstream and downstream sides of the spur. This swirling motion may extend up to the adjacent river bank and cause the bank erosion necessitating bank protection measures. The spacing of spurs in a wide river is larger than that in a narrower river for similar conditions. A larger spacing can be satisfactory for convex banks and a smaller spacing is desirable at concave banks. At crossings (*i.e.,* the straight reach between two consecutive bends of a river), an intermediate spacing can be adopted. Spacing between adjacent spurs is generally kept between 2 and 2.5 times the spur length (14). Ahmad (15) has suggested that spurs used for bank protection be spaced at five times their length. However, spurs used in navigation channels are generally spaced at 0.75 to 2 times their length (16). Maintenance of the nose of longer spurs during floods would generally be difficult as has been experienced in the past on the rivers Kosi and Gandak. Moreover, a longer spur would result in relatively higher afflux on the upstream side of the spur and may induce excessive seepage through the spur which may lead to piping and breach in the spur. Such breaches have indeed occurred in the rivers Kosi and Gandak (17). The top width of a spur would be between 3 and 6 m and a freeboard of 1 to 1.5 m above HFL should always be provided in case of non-submerged spurs (14). Slopes on the upstream shank and nose should be $1V : 2H$ and the slope on the downstream face may be $1V : 1.5H$ to $1V : 2H$ (14) (Fig. 12.7). Stone pitching on the slopes of a spur is placed manually as per the standard practice. A graded filter 20 to 30 cm in thickness, satisfying the standard filter criteria should be provided below the pitching (14). A launching apron (Art. 12.5.3.1) should also be provided to protect the stone pitching.

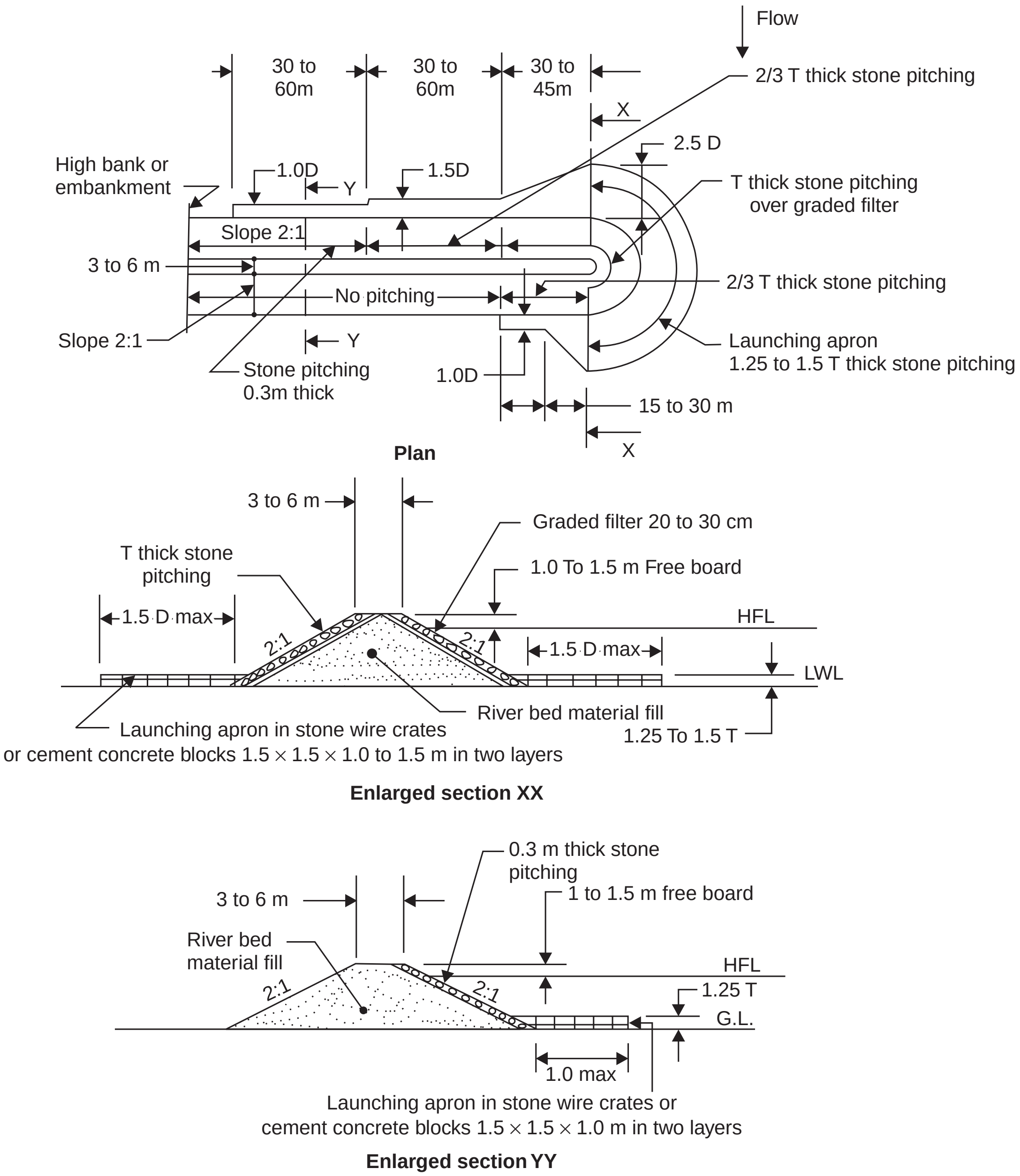
It is always advisable to finalise the spur designs only after conducting model studies. Spurs can be classified as follows:

(*i*) Classification based on the methods and material of construction : permeable and impermeable.

(*ii*) Classification based on the height of the spur with respect to high flood level : Submerged and non-submerged.

(*iii*) Classification based on the functions : attracting, deflecting, repelling and sedimenting, and

(*iv*) Special types : Denehy's T-headed groynes, hockey type, *etc.*



Note : See Fig. 12.10 and Eq. (12.7) for meanings of D and T.

Fig. 12.7 A typical impermeable spur

A series of permeable spurs reduces the flow velocity between the spurs which results in deposition of the sediment carried by the river water. Such spurs are, therefore, more suitable for rivers carrying heavy sediment load. In rivers carrying clear water, these spurs dampen the erosive strength of the current and thus prevent local bank erosion. Experience has indicated that permeable spurs are more effective than solid spurs for regulating the river course or protecting the banks and levees, especially in a sediment-laden river flow (3). Further, flow through the permeable spurs does not change abruptly (as it does in passing around a solid spur) and, hence, does not cause serious eddies and scour holes. Permeable spurs can be either submerged or non-submerged. These spurs are relatively cheap, but, are not strong enough to resist shocks and pressures from debris, floating ice, and logs and are, therefore, unsuitable for the upper reaches of a river (3).

Usually, permeable spurs are either tree spurs or pile spurs. A tree spur has a thick wire rope of about 25 mm diameter. This wire is firmly anchored to the bank at one of its ends and tied to a heavy buoy or concrete block at the other end (Fig. 12.8). Leafy trees with strong stem and branches are tied to the main rope by subsidiary ropes through holes drilled in the tree stem. The trees should be packed as closely as possible. When trees become heavy due to entrapped sediment, they sink.

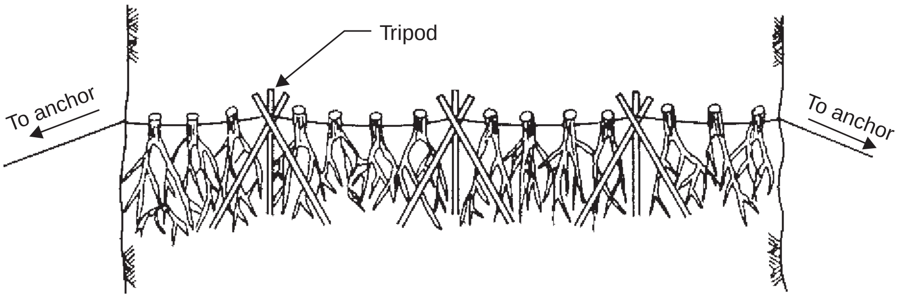


Fig. 12.8 A typical tree groyne

Pile spurs are constructed (3) by driving piles of timber or RCC or sheet piles up to about 6 to 9 m inside the river bed. These piles are 2.4 to 3 m apart and form at least two to three rows. Each row of piles is closely inter-twined either by brushwood branches or by horizontal railings. The upstream row is braced to the downstream row by transverses and diagonals. The space between the rows of piles is filled by alternate layers of 1.8 m thick brushwood weighted by 0.6 m thick boulders or sand bags. The filling should not be completely of stone since the spur is intended to be permeable to start with. Deep scour holes developed at the nose of these spurs do not cause danger because stones from the face of the spur fall into the scour hole and create a blanket which prevents undermining.

Impermeable or solid spurs are constructed as either rockfill or earth-core embankment armoured with a scour resistant surface. These can be made to attract or repel the flow away from the bank along the desired course. The side slopes vary from $1V{:}1H$ to $1V{:}5H$ depending on the material of construction (5). The nose of the spur is usually flat with a slope of $1V{:}5H$. Spurs used in river training for navigation are generally kept straight. But, other shapes of spurs, such as hockey spurs and T-shaped, have also been used. Shapes of scour holes for different types of spurs have been shown in Fig. 12.9. Obviously, T-head spur (first constructed by Denehy at the Okhla headworks on the Yamuna river and, hence, also known as Denehy's spur) requires stone apron protection against scour for relatively small area and, therefore, is most economical. T-head spurs have also been effectively used on the Ganga river at the Narora headworks.

Spurs meant for contracting the river channel are generally oriented with their axes normal to the current. Sometimes, spurs are oriented to point upstream, the advantage of which is that during the flood the flow is directed towards the centre of the river. Thus, the strong currents are kept away from the flood plains and flood dikes. Such spurs, pointing upstream, are also called repelling spurs. On the other hand, spurs pointing in the downstream direction attract the flow towards the bank and are, therefore, known as attracting spurs. A repelling spur produces a more desirable curvature to the flow downstream, leading to pronounced deposition (5). Besides, such a spur has large stagnation region on the upstream side and is, therefore, able to protect a greater length of the bank than that protected by the attracting spur. The repelling spur is usually inclined at 5° to 20° (to the line normal to the bank) in the upstream direction.

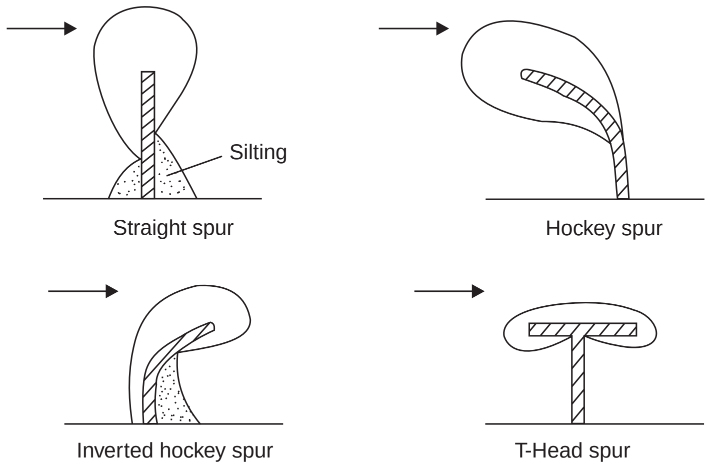


Fig. 12.9 Scour patterns for different spurs

12.5.3. Guide Banks

While selecting a site for bridge on an alluvial river, certain site requirements are always kept in mind. These requirements include straight reach of the river and small width of the river at the bridge site. The crossing reach between two successive bends of a meandering site is suitable from these considerations. However, the meandering pattern itself migrates and, hence, steps must be taken to ensure that the flow path does not change through the waterway at the bridge site, and also that the approach road embankment is not endangered due to the smaller waterway provided. For this purpose, earthen embankments are provided on one or both sides of the river at the bridge site. These embankments are known as guide banks (or guide bunds).

Guide banks are artificial embankments meant for guiding the river flow past a bridge (or other hydraulic structures such as weirs or barrages) without causing damage to the bridge and its approaches (3). Guide banks are built along the flow direction both upstream and downstream of the structure on one or both sides of the river as desired. Guide banks for a bridge restrict the waterway at the bridge site and prevent the outflanking of the bridge by the changing course of the river. The design criteria of guide banks are based on the works of Spring (18) and Gales (19).

The first step in the design of a bridge on an alluvial river is the estimation of the minimum and also a safe waterway. A reasonable estimation of clear waterway to be provided between guide banks can be obtained by equating it to Lacey's regime perimeter given by Eq. (8.29). The overall waterway between the guide banks is obtained by adding the thickness of piers to the clear waterway. Sharma and Asthana (20), based on their studies of design and performance of 20 bridges constructed during 1872-1966, recommended a waterway width (in metres) varying from 3.3 $\sqrt{Q}$ to 7.1$\sqrt{Q}$ in the alluvial stage and from 2.4$\sqrt{Q}$ to 4.8$\sqrt{Q}$ in the boulder stage. Here, Q is the river discharge in m^3/s. Obviously, a smaller waterway would cause a large afflux resulting in danger of outflanking.

Figure 12.10 shows the plan and sections of a typical guide bank. In plan, the guide bank can be either parallel, converging upstream or diverging upstream. In general, guide banks diverging upstream need to be longer than the straight or converging guide banks (5). Flows with acute curvature result in shoal formation (21) near the shanks [Fig. 12.11 (*a*)]. Hence, one may consider providing elliptical shanks, [Fig. 12.11 (*b*)] instead of straight shanks. However, there is a possibility of shoaling, away from the elliptical guide bank, due to the divergence and this should be studied carefully in a model before finalising the design.

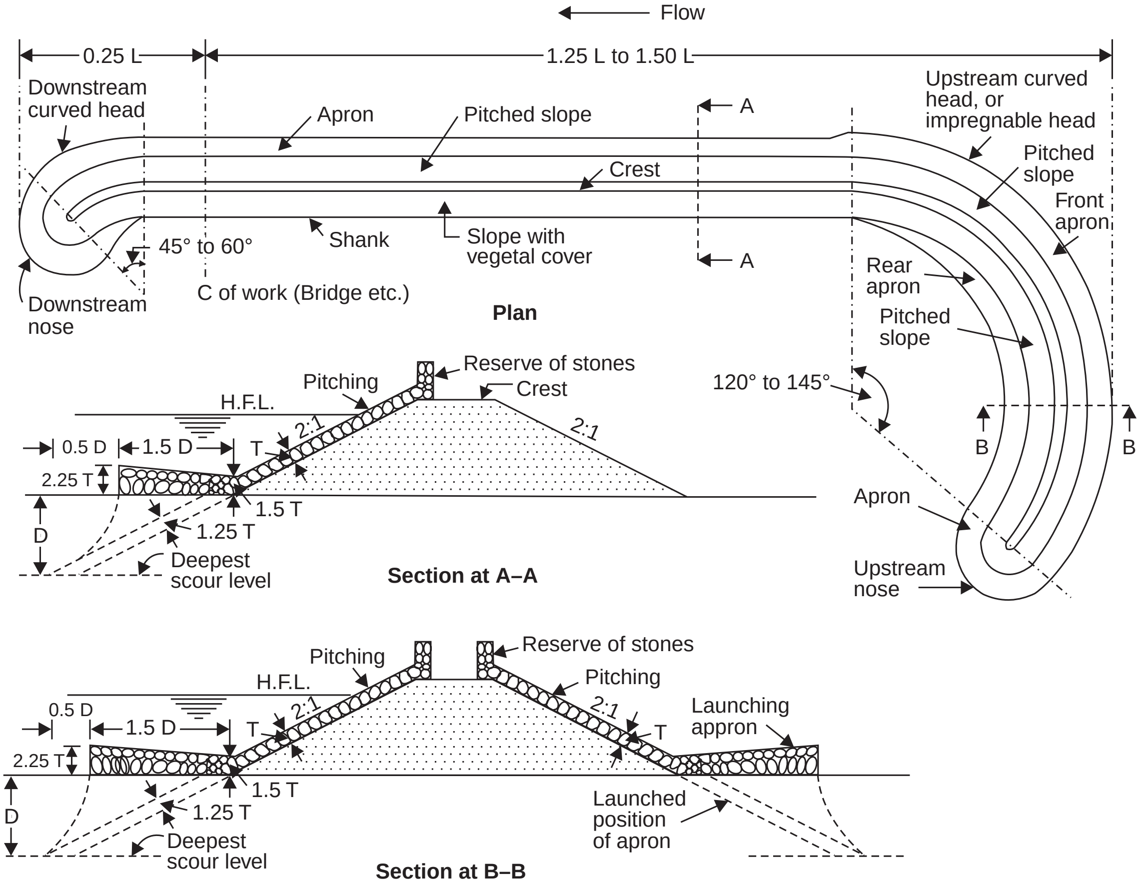


Fig. 12.10 Guide bank

The length of guide banks upstream of the bridge should be 1.1 times the length of the bridge (18). Gales (19), however, has recommended that this length should be between 1.25 and 1.5 times the bridge length for flood discharges ranging from 7000 to 70,000 m^3/s. The length of guide banks downstream of the bridge should be about 0.25 times the bridge length (5).

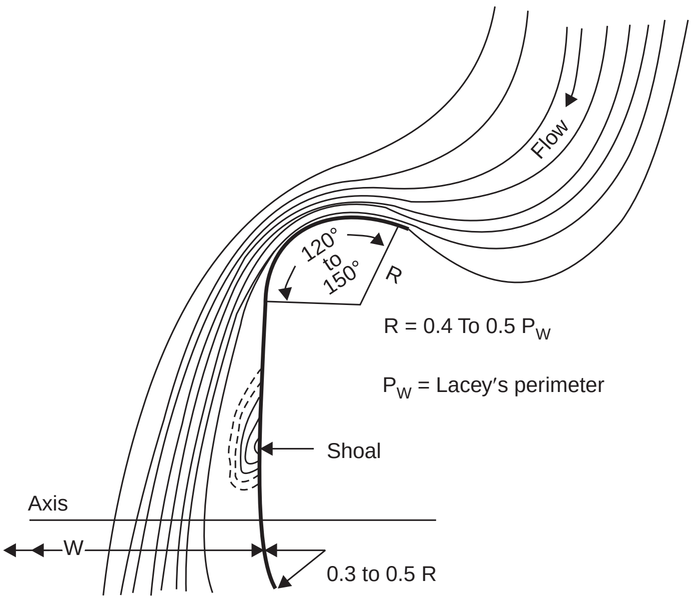


Fig. 12.11 (*a*) Flow near a straight guide bund with circular head

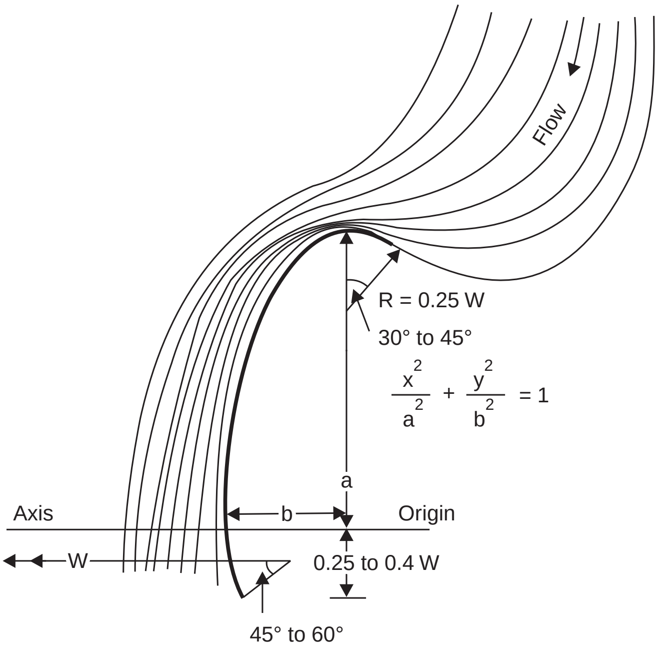


Fig. 12.11 (*b*) Flow near an elliptical guide bund

Radius of the upstream curved head R_1 should be equal to $2.2\sqrt{Q}$ metres. A smaller radius is permissible for smaller rivers. The sweep angle generally varies between 120° and 145° for the upstream curved head. The radius of the downstream curved head R_2 is generally kept equal to $1.1\sqrt{Q}$ metres. A sweep angle of 45° to 60° for the downstream curved head is considered satisfactory.

The elevation of the guide banks is obtained by adding a freeboard of 1.5 to 2.5 m to the high flood level of 100-year-flood. Alternatively, a freeboard of 1.0 m is added to the high flood level of a 500-year flood to determine the elevation of the top of guide bank. The top width of guide banks is generally fixed between 6 to 9 m (5). Both faces of the guide banks have side slopes of $1V{:}2H$ or flatter. Locally available sand, silt, and gravel are used for the construction of the core of the guide banks. To protect the face towards the river and the back of the curved heads of the guide banks from severe erosion, large stone pitching is provided. The embankment - side faces of the guide banks, however, do not need such protection. The stones used for slope protection must be large enough to withstand the force of current and stay in place. The minimum size of stones of relative density 2.65 required for this purpose can be calculated by the empirical relation (2, 22),

$$d = 0.023 \text{ to } 0.046U^2 \tag{12.6}$$

where, d is in metres and U is in m/s. Normally, angular and graded stones having the ability to interlock and weighing between 450 and 1800 N are used for slope protection. The thickness of stone pitching T in metres is related to river discharge Q (in m^3/s) by an empirical equation as follows (2, 5, 22):

$$T = 0.04 \text{ to } 0.06Q^{1/3} \tag{12.7}$$

For large streams, the constant 0.06 should be reduced to 0.04 (5). Thickness for pitching should be increased by 25 per cent for the curved head region. The pitching must be provided on both sides on the guide bank embankments in the curved head regions.

12.5.3.1. Launching Apron

Heavy scour of the river bed at the curved heads and shanks of guide banks can cause undermining of the stone pitching thereby resulting in failure of the guide banks. Such failure of guide banks can be prevented by providing launching aprons (consisting of stones) beyond the toe of the guide banks as shown in Fig. 12.10. As the scour continues, the launching apron is undermined and it eventually covers the face of the scour hole adjacent to the guide bank. The slope of the apron in the launched position varies between $1(V) : 1.25(H)$ and $1(V) : 2.5(H)$. For the purpose of design, a slope of $1(V) : 2(H)$ for loose boulders and $1(V) : 1.5(H)$ for concrete blocks can be assumed (23). The scour depths (below HFL) in the vicinity of the guide banks can be taken as K times Lacey's normal scour depth given by Eq. (8.32) or Eq. (8.33). The value of K is taken as 2.25 to 2.75 at the upstream nose and 1.5 to 2.0 at the downstream nose of guide banks, 1.25 for transition from the straight portion to the nose of guide bank and 1.5 to 2.0 for straight reach of guide banks (2, 5, 22). The width of the launching apron is generally kept equal to 1.5 times the scour depth (below the bed) at that place. The stone requirement for the launching apron is computed on the assumption of uniform apron thickness of 1.25 T in its launched position. Thus, if D is the depth of scour below the bed, the quantity of stone required for launching apron of 1 m length (along the guide bank) would be $\sqrt{5}\,D \times 1.25T$ *i.e.,* $2.8\,DT$ m^3. This volume is provided in the form of a wedge (to account for the non-uniformity of stone layer thickness in launched condition) as shown in Fig. 12.10. The launching apron should be provided on both sides of the guide bank embankements in the curved head regions. No spur should project from a guide bank. For maintenance purposes, a reserve of stones is usually kept ready on the top of the guide bank for dumping, if the bank is threatened.

Generally, a filter is always provided below the stone pitching provided for the bank protection. However, as per the earlier practices, the filter was generally, not provided between the launching apron and the river bed. It was believed that filter might hinder the launching of the apron. In the absence of a filter between the launching apron and the river bed, finer particles from the bed near the toe of the bank would escape through the voids in the apron which may result in vertical sinking of the apron and the toe of the bank gets exposed for possible failure of the bank protection. Hydraulic model studies (24) have indicated that the provision of filter below launching apron does not hinder its launching. During the launching of the apron, however, some filter material may get carried away downstream by the flow. Therefore, one should provide a relatively thicker filter layer. Or, alternatively, one may provide synthetic fibre filter.

Example 12.1 Design guide bunds (or banks) and launching apron required to be provided for a bridge across a river whose total waterway is 658.88 m. The design flood discharge is 13100 m^3/s which may be increased by 20% for the design of launching apron. Mean size of the river bed material is 0.3 mm.

Solution: The geometric parameters of the guide bunds are calculated from the equations given by Spring (18).

Length of the guide bund upstream of the bridge, L_1 is given as

$$L_1 = 1.1\ \text{L} = 1.1 \times 658.88 \cong 725.0\ \text{m}$$

Length of the guide bund downstream of the bridge, L_2 is given as

$$L_2 = 0.25\ \text{L} = 0.25 \times 658.88 \cong 165.0\ \text{m}$$

The radius of the upstream curved head, R_1 is given as

$$R_1 = 2.2\sqrt{Q} = 2.2\sqrt{13100} = 251.8\ \text{m} \cong 250\ \text{m}$$

The radius of the downstream curved head, R_2 is given as

$$R_2 = 1.1\sqrt{Q} = 1.1\sqrt{13100} \cong 125\ \text{m}$$

The upstream curved head is extended to subtend an angle (θ_1) of 120° to 145° at its center, while the downstream curved head subtends an angle (θ_2) of 45° to 60°. For the present design, θ_1 and θ_2 are assigned values of 145° and 60°, respectively. Plan-form of the design guide bund is shown in Fig. 12.12.

Cross-section of the Guide Bunds :

The top width of guide bunds varies from 6.0 m to 9.0 m and the side slopes for the two faces of guide bunds are generally kept 1V:2H or flatter. A freeboard of 1.5 m to 1.8 m above the maximum flood level is provided. For the guide bunds, the top width is proposed as 8.0 m, side slopes 1V:2H and the freeboard as 2.5 m. The freeboard is kept on higher side to accommodate afflux. Cross-sections at two locations of the guide bunds are shown in Fig. 12.12.

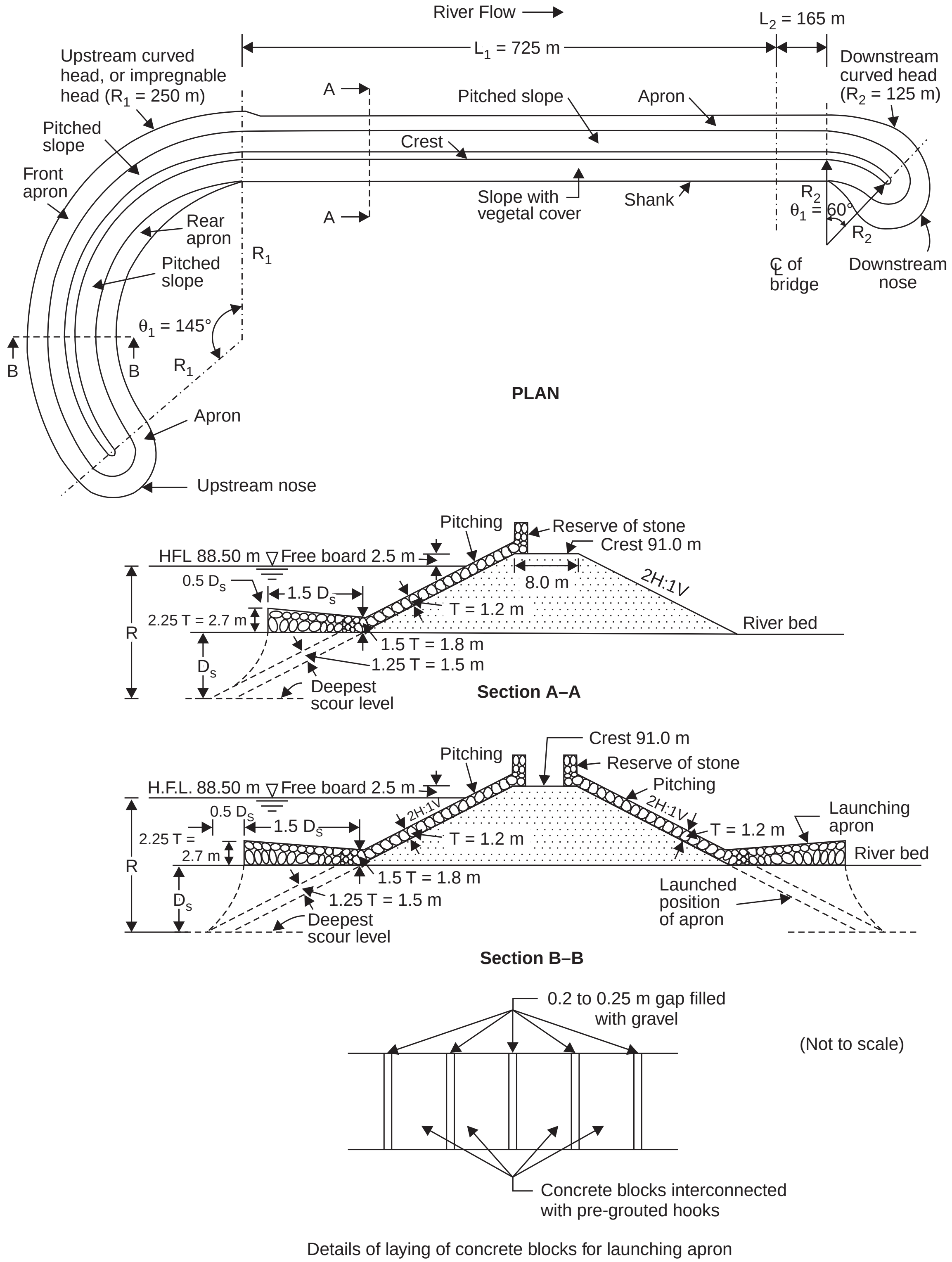


Fig. 12.12 Details of guide bund (plan, cross-sections of shank and curved head)

Bank Revetment :

The thickness T required for the bank revetment is related to the high flood discharge by the following empirical equation proposed by Inglis (2, 5):

$$T = 0.04 \text{ to } 0.06\ Q^{1/3} \text{ in SI units}$$

Adopting an average value of 0.05

$$T = 0.05Q^{1/3} = 0.05\ (13100)^{1/3} \cong 1.2 \text{ m}$$

A geosynthetic filter or a conventional sand-gravel inverted filter of 0.30 m thickness may be placed on the sloping surface of the guide bunds facing the river flow and the revetment provided over this filter; see Fig. 12.13.

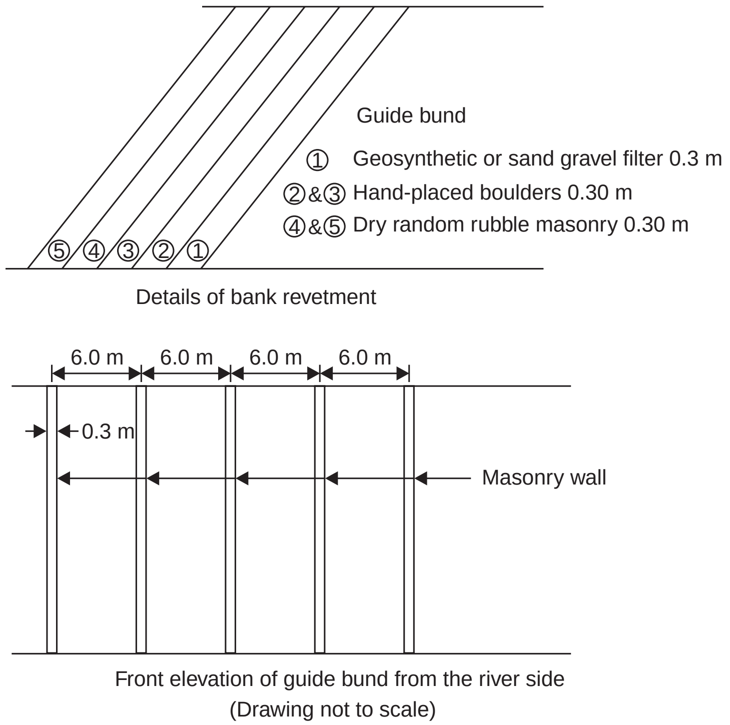


Fig. 12.13 Bank revetment

For the revetment itself, four layers of 0.30 m size boulders are suggested. The first two layers should be of about 0.30 m size hand-placed boulders are suggested. The first two layers should be of about 0.30 m size hand-placed boulders followed by third and fourth layers of thickness 0.30 m each of dry random rubble masonry on top of the hand-placed boulders. The rubble may be placed in a closely-packed formation inside a grid formed by masonry walls of 0.30 m width along the bank slope at a spacing of about 6.0 m measured in the direction of the river flow. The revetment should extend to 0.5 m above the HFL of the river.

Design of Launching Apron :

Design of launching apron requires estimation of scour depth. Discharge for the design of foundation is usually taken as 1.1 to 1.2 times maximum flood discharge. Adopting the value of 1.2Q *i.e.*, 15,720 m^3/s, the normal scour depth below HFL, R may be computed from Lacey's equation, Eq. (8.32).

$$R = 0.48 \left(\frac{Q}{f}\right)^{1/3}$$

where, $f = 1.76 \sqrt{d} = 1.76 \sqrt{0.3} = 0.96$

Thus, $$R = 0.48\left(\frac{15720}{0.96}\right)^{1/3} = 12.19 \text{ m}$$

Actual scour depth below HFL is increased to the following values for different parts of the guide bund following the recommendations of Inglis (2):

Upstream nose	:	$2.25R = 2.25 \times 12.19 = 27.43$ m
Downstream nose	:	$1.625R = 19.81$ m
Straight portion	:	$1.5R = 18.29$ m

The average thickness of the launching apron in its launched position is generally taken as 1.25 T. Since T = 1.2 m, the required thickness of the launching apron in its launched position is 1.5 m.

The minimum size of the boulder d_{min}, to be placed in the apron (so that it is not washed away by the flow), is related to the maximum velocity U_{max} in the vicinity of the guide bund (22).

$$d_{min} = 0.023 \text{ to } 0.046\ (U_{max})^2 \text{ in SI units}$$
$$= \text{say, } 0.0345\ (U_{max})^2$$

For the estimation of U_{max}, the value of Q is taken as 15720 m^3/s and Lacey's silt factor is taken as 0.96.

Using Lacey's regime equation, the regime velocity, U, can be computed as

$$U = \left(\frac{Qf^2}{140}\right)^{1/6} = \left(\frac{(15720)(0.96)(0.96)}{140}\right)^{1/6} = 2.17 \text{ m/s}$$

Assuming the maximum velocity to be 50% higher than the regime velocity,

$$U_{max} = 1.5 \times 2.17 = 3.26 \text{ m/s}$$
$$d_{min} = 0.0345\ (U_{max})^2 = 0.0345\ (3.26)^2 = 0.37 \text{ m, say } 0.4 \text{ m}$$

The stone requirement for the launching apron is computed on the assumption of uniform apron thickness of 1.25 T in its launched position. Thus, if D_s is the depth of scour below the river bed, the quantity of stone required for launching apron of 1 m length (along the guide bund) would be $\sqrt{5}\,D_s \times 1.25T$, *i.e.,* $2.8D_sT$ m^3, *i.e.,* $3.36D_s$m^3 for T = 1.2m. This volume of stone is provided in the form of a wedge (to account for nonuniformity of stone layer thickness in launched condition) as shown in Fig. 12.12. Boulders of size ranging from 0.3 m to 0.4 m may be used to construct launching apron. Alternatively, one may provide cement concrete blocks of size 1.0 m thick, 1.5 m long and 1.5 m wide instead of the boulders. The concrete blocks are to be laid leaving a gap of about 0.2 to 0.25 m between them as shown in Fig. 12.12. This gap would be filled with gravelly material. Further, these blocks are to be interconnected with the help of pregrouted hooks to prevent dislodging of individual blocks and ensure that these blocks launch as a monolithic apron into the scour hole.

12.5.4. Bank Protection

Banks caving due to wave action or erosive action of river flow can lead to river breach causing large amount of losses in terms of human life, property, agriculture, and other utilities. Bank protection measures are, therefore, important to prevent bank failures.

Bank protection measures provide a shield against erosion of bank material and maintain the alignment of banks. These can be of either direct or indirect type. Indirect bank protection measures, such as spurs are not constructed directly on the bank. But, direct bank protection measures, such as revetment, riprap, etc. are constructed on the bank itself.

For providing direct bank protection, all irregularities on the bank surface are removed, and the bank is graded to an acceptable slope. The value of this slope for banks of alluvial material containing little gravel ranges between $1(V) : 5(H)$ to $1(V) : 6(H)$ below low water line and between $1(V) : 3(H)$ to $1(V) : 4(H)$ above this line (16). A layer (several centimetres thick) of coarse material, such as gravel or broken stone is spread on this slope and the chosen revetment is laid on this layer.

Revetments are structures aligned parallel to the current and used to protect eroding banks. These revetments can be of different types such as the woven willow mattress, the framed willow mattress, the lumber mattress, the reinforced asphalt mattress, and the articulated concrete mattress. These types of bank protection measures are usually very costly, and have been used mainly in the USA and Europe.

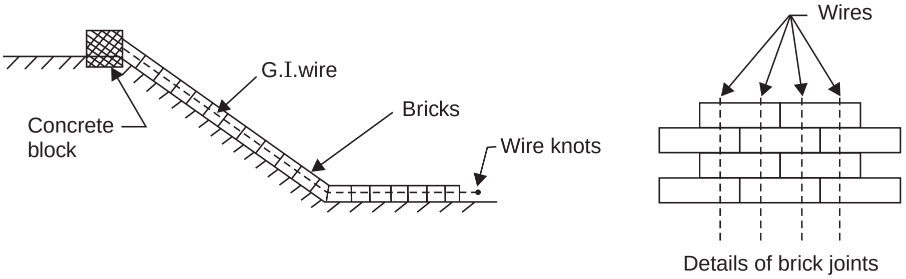


Fig. 12.14 Flexible brick pitching

A riprap paving with a toe trench is preferable to other types of revetments at sites where stone is cheap and available in plenty. Riprap of hard angular rock fragments laid on a thick layer of rubble or quarry chips is considered most durable (5). Concrete blocks can also be used when rocks are not available at reasonable costs. Triangular and tetrahedral types of concrete blocks are more suitable to resist the displacement by flowing water.

Another way of providing bank protection is by means of 'flexible brick pitching. Bricks for this purpose are manufactured such that each brick has two holes across the full width of a brick at one quarter length from either end. These bricks are then laid on the bank slope and for some distance on the river bed at the toe of the bank to serve as a falling apron. The bricks are laid in such a manner that a GI wire passes through the brick holes as shown in Fig. 12.14. The wires are knotted together at their ends and anchored in concrete blocks at their upper ends. These wires hold the bricks in place and permit small movement of bricks.

The bank revetment and launching apron, considered so useful measures for protection of banks of the alluvial streams, are not considered suitable for the protection of banks of gravel and boulder streams. In case of boulder streams, the bed does not scour much and, therefore, the apron would not be able to launch itself. Further, the concrete blocks forming the apron could be damaged and dislodged by the impact of the boulders being transported by the mountainous streams. The boulders, rolling on top of the apron, could hit the bank to cause damage to the bank. In such cases, A *RC* retaining wall near the toe of the bank may be provided from below the anticipated scour bed to about one metre above HFL. The *RC* retaining wall not only prevents the movement of its backfill but also resists bank erosion due to impact of boulders rolling along the river bed. The *RC* retaining wall would be designed for earth pressure, hydrostatic and earthquake forces, and the forces due to boulder impact.

12.5.5. Pitched Islands

A pitched island is an artificially created island in the river bed. It is protected by stone pitching on all sides. A pitched island is constructed with sand core and boulder lining. To protect it from scouring, a launching apron is also provided. The location, size, and shape of pitched islands are usually decided on the basis of model studies. Pitched islands serve the following purposes (3):

(*i*) Correcting an oblique approach upstream of weirs, barrages, and bridges by training the river to be axial,

(*ii*) Rectifying adverse curvature for effective sediment exclusion,

(*iii*) Redistributing harmful concentration of flow for relieving attack on marginal bunds, guide banks, river bends, *etc.*, and

(*iv*) Improving the channel for navigation.

A pitched island causes scour around it and, thus, redistributes the discharge on its two sides. Pitched islands upstream of barrages and weirs have been found to be quite effective.

12.5.6. Flush Bunds

Subsidiary channels of a braided river may flow very close to one of the banks of a river and result in bank erosion. Flush bund may be constructed across such channels as close to its offtake as possible. Top of the flush bund should be kept lower than the top levels of the surrounding shoal/islands to prevent outflanking of the bund on both sides when the water level in the main river rises (17).

12.5.7. Secondary Current Generating Structures

In addition to the above-mentioned conventional river training methods, there are other methods which depend on their capability to generate secondary currents to train a river. One such method is bandalling that has been successful in improving navigation conditions of rivers in Assam. Bandals are in the form of vertical mats or screens made of bamboos. These bandals are supported by bamboo poles that are driven into the river bed. The bandals are so erected that their upper edge is above the water surface and the lower edge extends upto about one-third to one-half the flow depth. The bandals are inclined to the stream to divert the surface currents towards the navigation channel. Bandals usually require replacement almost every year.

Surface panels consist of an assembly of deflectors suitably connected to an anchored barge. These panels cause double helical circulation stretching downstream along the river channel to be deepened for navigation purposes. Submerged bottom panels are placed on the river bed near the bank of the channel to be deepened. These panels perform rather poorly and are not as reliable as other panelling methods. Compared to the submerged bottom panels, the submerged vanes seem to be more effective.

The submerged vanes are small river training structures placed on the bed of a curved river channel. In the curved reach of a stream, the streamlines are curved and the centrifugal forces (proportional to the square of the local velocity) are exerted on river water. These forces are largest at or near the water surface. Therefore, the water near the water surface is driven towards the outer concave bank of the river and, to satisfy continuity requirements, the water near the river bed moves towards the inner convex bank. Thus, a spiral (or secondary) motion is imparted to the water flowing in a channel band. Since the sediment concentration is the highest near the river bed, the secondary flow near the river bed moves sediment towards the inner bank and deposits some of it near the inner convex bank. The sediment-deficient water coming from upper layers causes erosion of the outer convex bank. Short vertical submerged vanes, placed at suitable intervals in the outer half of the river bend and suitably inclined to the channel axis, are very effective in nullifying the secondary currents responsible for scouring of the outer convex bank of the curved channel (25). Submerged vanes, simple in design as well as construction, do not cause any significant change in either the area of channel cross-sections or the longitudinal slope of the water surface. Therefore, the stream characteristics upstream and downstream of the curved reach remain unaffected due to the submerged vanes.

The vane height above the stream bed is about 0.2 to 0.4 times the bank-full flow depth. The vanes are placed in arrays along the outer bank of the river. Each array may have two or more vanes. The vanes in an array will be spaced laterally at a distance of two to three times the vane height. The stream-wise spacing between the arrays will be about 15 to 30 times the vane height. The vanes should not be farther from the bank by more than four times the vane height.

12.5.8. Other River Training Methods

In addition to the major river training methods discussed above, there are other types of training methods too. A reservoir can be constructed for controlling floods by storing water during a flood and releasing it after the flood has receded. Another effective and economical way of flood control is by diverting part of the flood discharge from the main river. The diverted water can flow through either a natural or an artificial river channel and ultimately either join a lake or meet the sea. Channel improvements (such as by reducing channel roughness, by dredging a channel to widen and deepen it, and by increasing discharge-carrying capacity of the river channel) enable it to pass the flood discharge at a relatively smaller stage. Soil conservation practices increase the infiltration and, hence, decrease the peak runoff.

Sills or 'bed sills' are useful in counteracting the tendency of excessive scouring (and, hence, deepening) in parts of the river cross-section. Bed sills are placed across the deepest part of the cross-section so as to partially block the flow in the deeper part of the channel. The flow near the bed is, therefore, diverted towards the shallower part of the channel. This increases the depth of flow in the shallower part. Sills are thus very useful to make non-navigable river bends navigable.

Artificial cutoffs (discussed in Sec. 12.3.3) are also useful training measures to divert the river from a curved path which might be endangering important land area.

EXERCISES

12.1 Describe different types of rivers.

12.2 What are the objectives of river training ?

12.3 Describe the following river training methods:

(*i*) Spurs, (*ii*) Guide banks, and (*iii*) Bank protection.

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13

CANAL HEADWORKS

13.1. GENERAL

An irrigation channel takes its supplies from its source which can be either a river (in case of main canal) or a channel (in case of branch canals and distributaries). The structures constructed across a river source at the head of an offtaking main canal are termed "canal headworks" or "headworks". The headworks can be either diversion headworks or storage headworks.

Diversion headworks divert the required supply from the source channel to the offtaking channel. The water level in the source channel is raised to the required level so as to divert the required supplies into the offtaking channel. The diversion headworks should be capable of regulating the supplies into the offtaking channel. If required, it should be possible to divert all the supplies (at times of keen demand and low supplies) into the offtaking channel. The headworks must have an arrangement for controlling the sediment entry into the channel offtaking from a river. By raising the water level, the need of excavation in the head reaches of the offtaking channel is reduced and the command area can be served easily by flow irrigation.

Storage headworks, besides fulfilling all the requirements of diversion headworks, store excess water when available and release it during periods when demand exceeds supplies.

Most headworks in India are diversion headworks which can be either temporary or permanent. For temporary diversion headworks, bunds are constructed every year across the source river after floods. Sometimes, temporary diversion headworks are constructed in the beginning and when the demand for irrigation has developed sufficiently, they are replaced by permanent headworks. The Upper Ganga canal was run with temporary headworks for about sixty years before the permanent headworks were constructed. In order to further increase the irrigation capacity of the Upper Ganga canal system, these permanent headworks have now been replaced by new structures. For all important headworks, only permanent headworks should be constructed.

13.2. LOCATION OF HEADWORKS ON RIVERS

Larger rivers, generally, have four stages, *viz.,* the rocky, boulder, trough (or alluvial) and delta stages. Of these, the rocky and delta stages are generally unsuitable for siting headworks. Usually, the command area is away from the hilly stage, and it would, therefore, involve avoidable expenditure to construct a channel from headworks located in the hilly stage to its command area. In the delta stage, the irrigation requirements are generally less and also the nature of the river at this stage poses other problems.

The boulder and alluvial stages of a river are relatively more suitable sites for locating headworks. The choice between the boulder stage and the alluvial stage is mainly governed by the command area. If both stages are equally suitable for siting the headworks from command area considerations, the selection of the site should be made such that it results in the most economical alternative. The following features of the two stages should be considered while selecting the site for headworks.

(*i*) The initial cost of headworks in the boulder stage is generally smaller than that in the alluvial stage because of: (*a*) local availability of stones, (*b*) smaller width of river (requiring smaller length of weir), (*c*) smaller scour depths which reduce the requirements of cutoffs and other protection works, and (*d*) close proximity of higher banks which requires less extensive training works.

(*ii*) An irrigation canal offtaking from a river in the boulder region will have a number of falls which may be utilised for generation of electricity. There is almost no scope for the generation of electricity in this manner in the alluvial reach of a river.

(*iii*) If the existing irrigation demand is less but is likely to develop with the provision of irrigation facilities, it is desirable to divert the river water into an irrigation channel by constructing a temporary boulder bund across the river. This bund will be washed away every year during the floods and will be reconstructed every year. This will, no doubt, delay the Rabi crop irrigation, but it is worthwhile to use temporary bunds for a certain period; when the irrigation demand grows, permanent headworks may be constructed. In this manner, it would be possible to get returns proportional to expenditures incurred on the headworks. Construction of temporary bunds is generally not possible in the alluvial stage of the river.

(*iv*) An irrigation channel offtaking in the boulder stage of a river will normally require a large number of cross-drainage structures.

(*v*) Because of the nature of the boulder region, there is always a strong subsoil flow in the river bed. This causes considerable loss of water and is of concern during the periods of short supply. Similarly, there will be considerable loss of water from the head reach of the offtaking channel. In alluvial reach of the river this loss of water is much less.

(*vi*) The regions close to the hills usually have a wet climate and grow good crops. The irrigation demand in the head reach of the channel offtaking in the boulder stage is, therefore, generally small. However, this demand would increase with the provision of irrigation facilities. In alluvial regions, the demand for irrigation is high right from the beginning.

Once the stage of a river has been chosen for locating the headworks, the site of the headworks is selected based on consideration of its suitability for the barrage (or weir), the undersluices, and the canal head regulator.

For irrigation purposes, the site for headworks should result in a suitable canal alignment capable of serving its command area without much excavation. For siting the headworks, the river reach should, as far as possible, be straight and narrow and have well-defined and non-erodible high banks. In the case of a meandering river, the headworks should be located at the nodal point.

From sediment considerations, the offtaking channel should be located at the downstream end of the outside of a river bend so that it has the advantage of drawing less sediment. However, a curved reach would need costly protection works against the adverse effect of cross currents. Moreover, if canals take off from both the banks, the canal taking off from the inner bank draws relatively more sediment.

In order to ensure adequate supply to the offtaking canal at all times, the undersluice should be sited in the deep channel. A river reach with deep channels on both banks and shallow channel at the centre is more suitable when canals take off from both sides.

Besides, the site must be accessible and suitable for making the river diversion and other related arrangements at a reasonable cost.

13.3. DIFFERENT UNITS OF HEADWORKS

Diversion headworks (Fig. 13.1) mainly consist of a weir (or barrage) and a canal head regulator. A weir has a deep pocket of undersluice portion upstream of itself and in front of the canal head regulator on one or both sides. The undersluice bays are separated from other weir bays by means of a divide wall. In addition, river training structures on the upstream and downstream of weir, and sediment excluding devices near the canal head regulator are provided. Detailed model investigations are desirable to decide the location and layout of headworks and its component units. A typical layout of diversion head works is shown in Fig. 13.1.

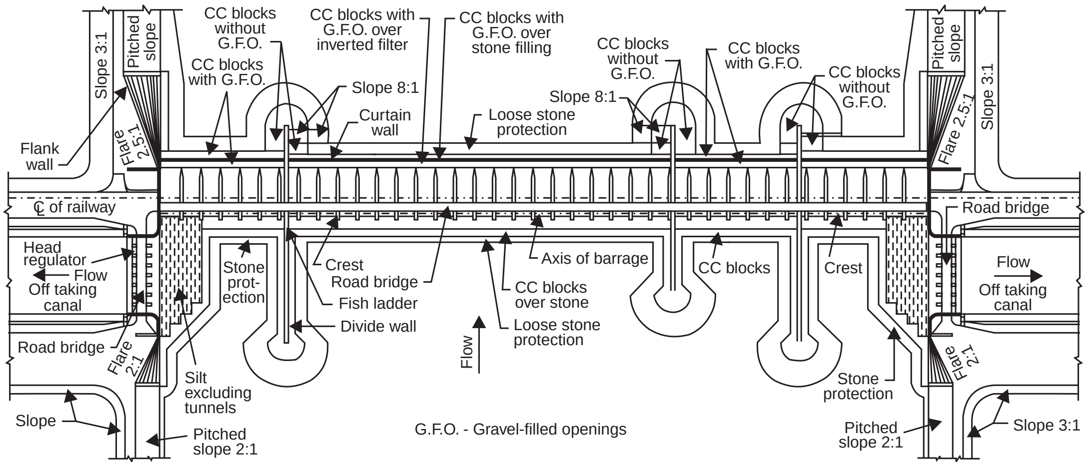


Fig. 13.1 Typical layout of headworks (1)

13.4. WEIR (OR BARRAGE)

A weir is an ungated barrier across a river to raise the water level in the river. It raises the water level in the river and diverts the water into the offtaking canal situated on one or both of the river banks just upstream of the weir. Weirs are usually aligned at right angles to the direction of flow in the river. Such weirs will have minimum length and normal uniform flow through all the weir bays thereby minimising the chances of shoal formation and oblique flow (1).

To increase the water level, the weir crest is raised above the river bed. Part of the raising of the water level is obtained by shutters provided at the top of the weir crest. These shutters are dropped down during floods so that the afflux is minimum. The afflux is defined (2) as the difference in water level between the upstream and downstream of a structure under free flow conditions as a result of construction of the structure across a river. Controlling pond levels by means of shutters becomes difficult when the difference between the pond level and the crest level is higher than 2.0 m. In such cases, a gate-controlled weir, better known as barrage, is preferred. Barrage is a gate-controlled weir with its crest at a lower level. A barrage and weir are similar structures and differ only in a qualitative sense. The crest of a barrage is usually at a lower level and the ponding up of the river for diversion into the offtaking canal is achieved by means of gates (instead of shutters). Barrages are considered better than weirs due to the following reasons:

(*i*) Barrages offer better control on the river outflow as well as discharge in the offtaking canal.

(*ii*) With proper regulation and with the help of undersluices and sediment excluders, the upstream region in the vicinity of the headworks can be kept free of sediment deposition so that sediment-free water enters the offtaking canal.

(*iii*) Because of the lower crest level of a barrage, the afflux during floods is small.

(*iv*) It is possible to provide a roadway across the river at a relatively small additional cost.

Because of these advantages, barrages are usually constructed at the site of headworks on all important rivers. At some barrages, the raised crest may not be provided at all and the complete ponding is obtained only by means of gates. Figure 13.2 shows the longitudinal section of Sarda barrage which does not have a raised crest.

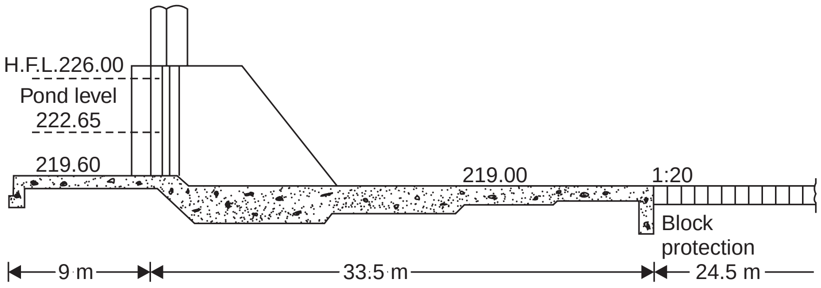


Fig. 13.2 Longitudinal section of Sarda barrage

The procedure of design of a barrage is similar to that of a weir. Weirs are of the following three types:

(*i*) Masonry weirs with vertical downstream face,

(*ii*) Rockfill weirs with sloping apron, and

(*iii*) Concrete weirs with glacis.

13.4.1. Masonry Weirs with Vertical Downstream Face

Figure 13.3 shows the typical sketch of a masonry weir which consists of a horizontal masonry floor and a masonry crest with vertical (or nearly vertical) downstream face. Shutters provided at the top of the crest raise the water level further. During floods, these shutters are dropped down to pass the floods effectively and reduce the afflux upstream. The stability of the crest should be examined for the following conditions:

(*i*) The water level on the upstream side is up to the top of the shutters with no flow on the downstream side and all the water is diverted into the offtaking canal. The overturning moment caused by the water pressure on the upstream side must be resisted by the weight of the crest without any tension at its upstream end. The stability of the crest against sliding due to water pressure should also be examined.

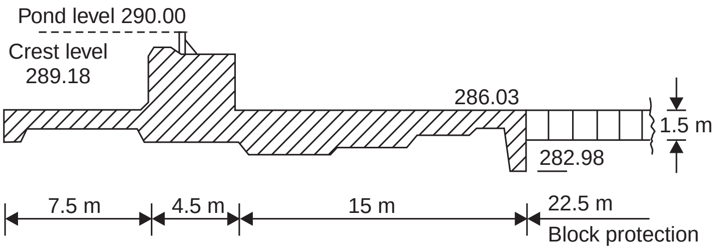


Fig. 13.3 The Bhimgoda weir (old) on the Ganga river at Hardwar

(*ii*) When the shutters are dropped down, water flows over the crest and the overturning moment is reduced due to the lowered water level on the upstream and presence of water on both sides of the crest. However, there will be some loss of weight (and, hence, the resisting moment) of the crest due to floatation because of the crest not being completely impervious. It is impossible to determine the amount of this loss of weight accurately. The reduced resisting moment is calculated on the basis of full weight of the masonry above the downstream level and submerged weight below the downstream level. The safety of the crest is examined for different stages of discharge up to the maximum flood discharge. At all such stages, the resisting moment must be more than the overturning moment and there should be no tension at the upstream end of the crest.

13.4.2. Rockfill Weirs with Sloping Aprons

Figure 13.4 shows the longitudinal section of a typical rockfill weir whose main body consists of dry boulders packed in the form of glacis with few intervening walls. This type of weir is the simplest one, but requires a large quantity of stones for construction as well as maintenance. As such, this type of weir is suitable in areas where a large quantity of stones is available in the vicinity of the site and where labour is cheap.

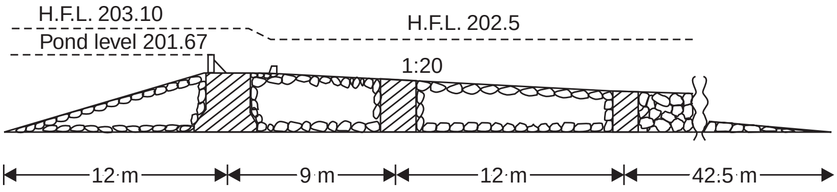


Fig. 13.4 The Okhla weir (old) on the Yamuna river near Delhi

13.4.3. Concrete Weirs with Glacis

Figure 13.5 shows the longitudinal section of a typical concrete weir in which the excess energy of overflowing water is dissipated by means of a hydraulic jump which forms near the downstream end of the glacis. Barrages are also constructed like concrete weirs. Design of such weirs is mainly based on the method proposed by Khosla *et al*. discussed in Chapter 9. On pervious foundations, only concrete weirs are constructed these days. Their detailed design requires the knowledge of: (*i*) the maximum flood discharge and corresponding level of the river at and near the selected site for weir, (*ii*) the stage-discharge curve of the river at the weir site, and (*iii*) the cross-section of the river at the weir site.

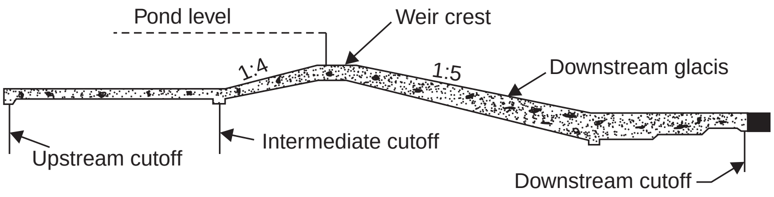


Fig. 13.5 Concrete weir

Based on the site conditions, general and economic considerations, and other data, the designer decides (*i*) the afflux, (*ii*) the pond level, (*iii*) the minimum waterway (or the maximum discharge per metre length of weir), and (*v*) the weir crest level.

13.5. UNDERSLUICES

The construction of weir across a river results in ponding up of water and causes considerable sediment deposition just upstream of the canal head regulator. This sediment must be flushed downstream of the weir. This is done by means of undersluices (also called sluice ways or scouring sluices). A weir generally requires deep pockets of undersluices in front of the head regulator of the offtaking canal, and long divide wall to separate the remaining weir bays from the undersluices. The undersluices are the gate-controlled openings in continuation of the weir with their crests at a level lower than the level of the weir crest. The undersluices are located on the same side as the offtaking canal. If there are two canals each of which offtakes from one of the banks of the river, undersluices are provided at both ends of the weir.

The undersluices help in keeping the approach channel of the canal head regulators relatively free from deposition of sediment, and minimise the effect of the main river current on the flow over the head regulator. In addition, the undersluices are also useful for passing low floods, after meeting the requirements of the offtaking canal, without having to raise the gates or drop the weir shutters. With the provision of the undersluices, the weir shutters have to be dropped (or the gates have to be raised) only to pass the high floods which occur only during the monsoon. The shutters are again raised (or the gates are lowered) at the end of the monsoon.

The level of the crest of an undersluice is related to: (*i*) cold weather river bed level at the site of the weir, and (*ii*) the crest level of the head regulator. The crest of the undersluices generally coincides with the lowest cold weather level of the river bed at the site of the weir. Also, the crest of the undersluices should be kept at least 1.20 m below that of the head regulator so that the sediment deposited upstream of the undersluices (and the head regulator) does not enter the offtaking canal and can be carried to the downstream of the undersluices. If a sediment excluder (a structure to reduce entry of sediment into the canal) is to be provided, it becomes necessary to lower the crest of the undersluices to about 2.0 m below the crest of the head regulator. Alternatively, the crest of the regulator is raised.

The discharge capacity of the undersluices is kept equal to the maximum of the values given by the following three considerations (2):

(*i*) To ensure sufficient scouring capacity, the capacity of the undersluices should be at least twice the canal discharge.

(*ii*) To reduce the length of the weir, the undersluices should be capable of passing about 10 to 20 per cent of the maximum flood discharge at high floods.

(*iii*) The undersluice should possess enough capacity to pass off low floods with the water surface in the reservoir at pond level so that the need of lowering the weir shutters (or raising the weir gates) does not arise.

The longitudinal section of the undersluices will be similar to that of a weir and, hence, the design procedure is the same as that of a concrete weir. However, because of the larger discharge per unit length, the undersluices would need much heavier protection downstream. When sediment excluders are provided, the width of undersluices is determined by the velocity required to induce siltation. On all major headworks, the design of undersluices and divide wall is finalised on the basis of model investigations.

13.6. AFFLUX, WATERWAY, AND DIFFERENT LEVELS FOR WEIR CONSTRUCTION

Due to the construction of a weir across a river, the high flood level of the river upstream of the weir rises. This rise is termed the afflux and is usually represented as the difference in the total energy levels of the upstream and downstream of the weir. In the beginning, the afflux is confined to a short reach of the river but, extends gradually very far upstream in case of the alluvial rivers. The top level of guide banks and marginal bunds and also the length of the marginal bunds are decided by the amount of the afflux. Besides, the afflux affects the dynamic action downstream of the weir and also the location and parameters of the hydraulic jump. A higher afflux may reduce the width of the waterway but, increases the discharge per unit length of weir. This results in increased depth of scour which, in turn, increases the cost of protection works. A higher afflux also increases the risk of failure of river training structures due to possible outflanking.

In the case of weirs located on alluvial rivers, an afflux of 1 m is considered satisfactory in the upper and middle reaches of the river. In the lower reaches with flat gradients, the afflux should be limited to about 0.3 m (2).

The pond level is the water level which must be maintained in the undersluices pocket (*i.e.*, upstream of the canal head regulator) so that full supply level can be maintained in the canal when full supply discharge is fed into it. The full supply level of a canal at its head is obtained from the longitudinal section of the canal. The pond level is kept about 1.0 to 1.2 m higher than the full supply level of the canal so that sufficient working head is available even when the head reach of the canal has silted up, or when the canal has to be fed excess water. If under certain situations, there is a limitation of pond level, the full supply level is fixed by subtracting the working head from the pond level.

The waterway and the afflux are interdependent. Normally, in plains, the width of waterway is kept equal to about 10 to 20% more than Lacey's regime perimeter for the design flood discharge. In rivers with coarser bed material, the width of waterway can be kept about 10-20% smaller than Lacey's perimeter. A smaller waterway increases the afflux and the cost of protection works. On the other hand, a larger waterway is uneconomical and may cause oblique approach, thereby silting part of the waterway. The ratio of the overall length of the weir provided to the minimum stable width of river obtained from Lacey's equation for the design flood discharge is termed *looseness factor* (2). Table 13.1 lists the values of looseness factor along with some other salient features of some barrages.

The weir crest level, afflux, waterway, and pond level are interrelated and a suitable set of values of all these four parameters are decided within their respective limits. The pond level can be maintained by keeping the weir crest at the pond level. Alternatively, the weir crest can be kept at a lower level and the pond level is, then, maintained with the help of shutters or gates. The level of the weir crest is decided as follows:

Table 13.1 Salient details of some barrages (3)

	Bhimgoda barrage	*Godavari barrage*	*Kosi barrage*	*Kota barrage*	*Nangal barrage*	*Narora barrage*	*New Okhla barrage*	*Ramganga barrage*
Location	Hardwar (UA)	Dowlaiswaram (AP)	Bhimnagar (Nepal)	Kota (Rajasthan)	Nangal (Punjab)	Bulandshahar (UP)	New Delhi	Hareoli (UP)
Purpose	Irrigation	Irrigation	Flood control, irrigation, and power	Irrigation	Irrigation and power	Irrigation supply	Irrigation and water supply	Irrigation
River	Ganga	Godavari	Kosi	Chambal	Sutlej	Ganga	Yamuna	Ramganga
Design flood (m^3/s)	19,300	91,475	26,897	21,238	11,327	14,165	8,495	7,365
Width of river (m)	675	5860	1158	487.68	291	–	445.73	–
Length of barrage (m)	455	3599	1149	551.69	291	922.43	552	408
Width of undersluice bays (m)	18	–	18.29	–	9.1	15.24	18.3	18
Number of undersluice bays	7	3	10	2	–	7	5	3
Width of barrage bays (m)	18	18.29	18.29	12.2	9.1	12.19	18.3	18
Number of barrage bays	15	175	46	19	26	54	22	17
Thickness of piers (m)	2.5	2.13	3.048	3.0	2.1	2.44	2.13	2.5
Looseness factor	0.69	2.51	1.48	0.80	0.42	1.63	1.26	1.00
River bed slope	–	1 in 2450	1 in 513	–	–	–	–	1 in 2347
Silt factor	–	1.1	1.3	–	–	–	–	1.28

(*i*) From the stage-discharge curve at the weir site, the high flood level (HFL) for the design flood (usually 50- to 100-year frequency flood) discharge is determined.

(*ii*) The level of the downstream total energy line (TEL) is determined adding the velocity head to the HFL. The velocity of flow is computed using the Lacey's regime equation for velocity.

(*iii*) The permissible afflux is added to the level of the downstream TEL to obtain the level of the upstream TEL.

(*iv*) The discharge intensity q is determined by dividing the design flood discharge by the width of clear waterway.

(*v*) Using the relation,

$$q = Ck^{3/2} \tag{13.1}$$

the height of TEL, above the weir crest, k is determined. Here, k (in meters) is the total head with respect to the weir crest. In Eq. (13.1), the value of C depends on many factors, such as the head over the weir crest, shape and width of the crest, the crest height over the upstream floor, and the roughness of the crest surface. It is, therefore, advisable to estimate C by the use of model studies if the values based on prototype observations based on similar structures are not available (2). For broad-crested weirs having crest width more than 2.5 times the head over the crest, C may be taken as 1.71. For the crest whose width is less than 2.5 times the head over the crest, the value of C is taken as 1.84.

(*vi*) The level of the weir crest is obtained by subtracing k from the level of the upstream TEL.

(*vii*) After fixing the weir crest level, length and suitable number of weir bays are decided. The total discharge capacity of the weir and undersluice bays is worked out using the following discharge equation which takes into consideration the reduction in width of flow on account of end contractions.

$$Q = C\,(L - KnH)\,H^{3/2} \tag{13.2}$$

Here, L is the overall waterway, H the head over the crest, n the number of end contractions, and K is a coefficient which ranges from 0.01 to 0.1 depending upon the shape of the abutment and the pier nose. The exact value of the coefficient C depends on several factors, *viz.,* head over the crest, the shape and size of the crest, the height of the crest over the upstream floor, and the roughness of the crest surface. Use of model studies is suggested for the estimation of C. In the absence of such studies, C can be assumed as 1.71 in SI units (4).

(*viii*) The height of shutters or gates will be equal to the difference between the pond level and the level of the weir crest.

For weirs without shutters (or gates), the crest level should, obviously, be at the required pond level. For weirs with falling shutters, the crest level should not be lower than 2 m below the pond level as the maximum height of the falling shutters is normally limited to 2m. If the crest level so fixed causes too much of afflux, the waterway of weir may be suitably increased. For barrages too, the crest level is similarly determined by the head required to pass the design flood at the desired afflux. It is desirable that the crest (of the barrage) and the upstream floor levels of the undersluices be kept at the lowest bed level of the deep channel of the river as far as practicable. The upstream floor level of the remaining bays should be kept normally 0.5 to 1.0 m above the upstream floor level of the undersluice bays or the general river bed level.

As a result of the construction of weir across a river, the downstream bed levels will be lowered due to degradation (or retrogression) and, hence, the downstream HFL will also be lowered. The lowering of water levels due to retrogression on the downstream increases the exit gradient. Retrogression is relatively more in alluvial rivers carrying fine sediment and having steep slope. When the proposed weir is sited downstream of a dam, the retrogression increases. Retrogression should always be considered for the design of the downstream floor and the downstream protection works.

During high floods, the river water carries lot of sediment which reduces the extent of retrogression and, hence, lowering of HFL is only marginal - of the order of about 0.3 to 0.5 m. But, during low floods the river water downstream of the weir is relatively clear and may increase retrogression thereby lowering the HFL by an appreciable amount ranging from 1.25 to 2.25 m, depending upon the amount of sediment in the river bed and the bed slope of the river (2). At the design flood in an alluvial river, the reduction in river stages due to retrogression may be considered to vary from 0.3 to 0.5 m depending upon whether the river is shallow or confined during floods (2). For other discharges, the effect of retrogression may be obtained by the variation of retrogressed flood levels with the flood discharges. The downstream TEL is lower than the upstream TEL by an amount H_L which is equal to the sum of the assumed afflux and the estimated retrogression.

During the first few years of weir construction, the sediment-carrying capacity of the river decreases due to ponding up of water upstream of the weir. This results in continuous deposition of the sediment upstream of the weir and the bed level rises. Ultimately, the bed slope regains its original slope and the afflux extends still further upstream of the backwater profile. The marginal bunds, constructed to take care of the rise in water level in the backwater region, will, then, have to be extended further upstream. A stage is thus reached when the upstream pond takes no more sediment. Since the offtaking canal withdraws relatively sediment-free water, the sediment is now carried downstream of the weir with reduced water discharge (and, hence, reduced sediment-carrying capacity). Therefore, the sediment will start depositing on the downstream side and raise the bed levels lowered due to initial degradation. Sometimes, the bed levels can rise even beyond the original bed levels.

13.7. DESIGN OF WEIR

Dimensions of different parts (such as vertical cutoff, impervious floor, *etc*.) of a weir on a permeable foundation are determined from surface and subsurface flow considerations as discussed in the following sections.

The data required for this purpose include:

(*i*) longitudinal section and cross-section of river at the weir site,

(*ii*) stage-discharge relationship including HFL and the corresponding discharge,

(*iii*) characteristics of sediment and river bed material, and

(*iv*) canal data such as full supply level and the corresponding discharge, canal cross-section, longitudinal section, *etc.*

13.7.1. Vertical Cutoffs

Vertical cutoff (or sheet pile lines) at the upstream and downstream ends of a weir are always provided to guard against scouring at the upstream and downstream ends and the piping effects at the downstream end. Intermediate cutoffs, provided at the ends of the upstream and/ or the downstream slopes of the impervious floor, are useful in holding the main structure, *i.e.,* the weir, in case of failure of the upstream and/or the downstream cutoffs. The depth of cutoffs should be such that its bottom is lower than the level of possible flood scour at that section. The downstream cutoff, in addition, should also be sufficient to reduce the exit gradient within safe limits which is decided by the subsurface conditions. The depth of scour below HFL, R is given by Lacey's equation, Eq. (8.33),

$$R = 1.35\left(\frac{q^2}{f_1}\right)^{1/3} \tag{13.3}$$

For fixing the depths of cutoffs, the scour depth R should be calculated with discharge intensity q taking into account the concentration factor. The *concentration factor* is a factor by which the discharge per unit length of a weir, assuming uniform distribution across the river width, is required to be multiplied to obtain the design discharge per unit length for designing stilling basin and the cutoffs of the weir (2). The concentration factor accounts for the nonuniform distribution of the flow along the waterway during the operation of weir bays.

Equation (13.3) computes scour depth R for the regime condition of flow existing during high floods. Other features, such as bends, increase the depth of scour. The extent of scour (below water level) in a river with erodible bed material varies at different places along a weir as shown in Table 13.2.

Table 13.2 Likely extent of scour along a weir (2)

Location	*Range*	*Mean*
Upstream of impervious floor	1.25 R to 1.75 R	1.50 R
Downstream of impervious floor	1.75 R to 2.25 R	2.00 R
Noses of guide banks and divide wall	2.00 R to 2.50 R	2.25 R
Transition from nose to straight part	1.25 R to 1.75 R	1.50 R
Straight reaches of guide banks	1.00 R to 1.50 R	1.25 R

If the sub-stratum contains any continuous layer of clay in the vicinity of the downstream cutoff, the depth of the upstream and downstream cutoffs should be suitably adjusted to avoid increase of pressure under the floor.

13.7.2. Weir Crest, Glacis, and Impervious Floor

The weir crest is provided flat at the computed level with a width of about 2 m (2). If the weir is to behave as a broad-crested weir, the width should be more than 2.5 times the head over the weir. The upstream slope of the weir is fixed between 2(H):1(V) to 3(H):1(V).

The downstream slope of the weir crest and downstream horizontal floor (*i.e.*, stilling basin) should be such that they result in the maximum dissipation of energy through stable hydraulic jump besides being economic. The slope of the downstream glacis should be around 3(H):1(V). The level of the downstream horizontal floor is fixed in such a manner that the hydraulic jump starts at the end of the glacis or upstream for all discharges. The location of the hydraulic jump is determined (for high flood and pond level discharges) by using the method of Sec. 9.2.8. The floor level is kept at or below the lower of the required floor levels for these two conditions. A concentration factor of 1.2 is usually adequate for the design of the stilling basin.

The total floor length of impervious floor includes the downstream basin length, glacis, weir crest, and upstream floor. The impervious floor in conjunction with the downstream cutoff should result in safe exit gradient. Besides, the hydraulic jump must remain confined within the downstream floor. It should also satisfy the requirements of uplift pressures.

The length of the downstream horizontal floor should be such that the entire jump is confined only to the floor. This will ensure that the filter and the stone protection provided on the downstream of the floor are not affected adversely by the jump. Hence, the length of the downstream horizontal floor is kept equal to the length of the jump which is equal to five to six times the height of the jump [(*i.e.*, 5 to 6 $(h_2 - h_1)$]. Here, h_1 and h_2 are pre-jump and post-jump depths of flow. Obviously, the maximum height of the jump should be considered for these calculations.

The length of the upstream horizontal floor, provided at the river bed level, is decided in such a manner that the resulting exit gradient, G_E is less than the safe exit gradient for the soil under consideration. In Eq. (9.66), the depth of the downstream cutoff is measured below the bed (or the scoured bed, if scour is anticipated). Theoretically, the safe exit gradient should be equal to the critical gradient which is unity for average soils as given by Eq. (9.50). However, in practice, the safe exit gradient is kept lower than the critical gradient mainly due to non-uniformity of the soil conditions. Besides, the presence of faults and fissures in the subsoil, possibility of scouring up to the bottom of the vertical cutoff, sudden changes in head due to sudden dropping of gates, and similar factors suggest that the safe exit gradient be kept lower than the critical gradient. The recommended values of the safe exit gradient are 1/4 to 1/5 for shingles, 1/5 to 1/6 for coarse sand, and 1/6 to 1/7 for fine sand (2).

Thus, knowing the value of the safe exit gradient G_E, the head H, and the depth of the downstream cutoff d, one can determine the total floor length required, *i.e.*, b, using Eq. (9.66) which is

$$G_E = \frac{H}{d}\frac{1}{\pi\sqrt{\lambda}} \tag{13.4}$$

where,
$$\lambda = \frac{1}{2}\left[1 + \sqrt{1 + (b/d)^2}\right]$$

The thickness of the impervious floor is decided from considerations of the uplift pressures which can be determined using the method proposed by Khosla *et al*. and as explained in Chapter 9. The thickness of floor in the jump trough should be examined for different stages of flow. For other parts of the downstream floor, however, the maximum uplift would occur when the water is at the pond level on the upstream without any tail-water (or flow) on the downstream side.

13.7.3. Upstream and Downstream Loose Protection

To protect a weir structure from the adverse effects of scour at the upstream and downstream ends of the structure, some additional measures in the form of cement concrete blocks over loose stones are needed.

Just beyond the upstream end of the impervious floor, pervious protection in the form of cement concrete blocks is provided. The blocks should be of adequate size so as not to get dislodged. For barrages in alluvial rivers, the blocks of size 1500 mm × 1500 mm × 900 mm are used (2). The size of the blocks should, obviously, be larger in case of barrages in the boulder reach of a river. The length of the upstream block protection is kept approximately equal to the depth of scour below the floor level. The blocks are laid over loose stones.

The pervious block protection is provided beyond the downstream end of the impervious floor as well. The cement concrete blocks of size not less than 1500 mm × 1500 mm × 900 mm are laid over a suitably designed inverted filter. The space of about 75 mm between the adjacent blocks is packed with gravel. The length of the downstream block protection is kept approximately equal to 1.5 times the scour depth below the floor level.

The graded inverted filter should approximately conform to the following criteria (2):

$$\frac{D_{15} \text{ of filter}}{D_{15} \text{ of river bed material}} \geq 4 \geq \frac{D_{15} \text{ of filter}}{D_{85} \text{ of river bed material}}$$

The filter may be provided in two or more layers. The grain size curves of the filter material and the bed material should be approximately parallel.

A toe wall of masonry or concrete should also be provided at the end of the inverted filter to prevent the filter from getting disturbed. The wall should extend up to 500 mm below the bottom of the filter.

Beyond the block protection on the upstream and downstream of a weir on permeable foundation, launching aprons of loose boulders or stones are provided so that these stones can spread uniformly over the scoured slopes. These stones should not weigh less than 400 N, and should always be larger than 300 mm in size. In case of non-availability of suitable material for launching aprons, cement concrete blocks or stones packed in wire crates are used. The quantity of stone provided in launching aprons should be adequate to cover the slope of scour holes (ranging from 2 (H) : 1 (V) to 3 (H) : 1 (V) for alluvial rivers and 1.5 (H) : 1 (V) for boulder rivers) with a thickness of 1.25 T where T (*i.e.*, thickness of pitching) is to be obtained from Table 13.3.

Table 13.3 Thickness of pitching (T) for loose stone protection in millimetres (2)

Type of river bed material	*River slope (m/km)*				
	0.05	*0.15*	*0.20*	*0.30*	*0.40*
Very coarse	400	500	550	650	700
Coarse	550	650	700	800	850
Medium	700	800	850	950	1100
Fine	850	950	1000	1100	1150
Very fine	1000	1100	1150	1250	1300

The depth of scour holes, calculated from Lacey's equation, is to be modified according to Table 13.2 for different locations of launching aprons. No allowance in the form of concentration factor need be made for computing normal scour depth for the upstream and downstream protection works (2).

The total quantity of stone for launching apron, worked out as mentioned above, should be laid in a length of about 1.5 D to 2.5 D. Here, D is the depth of scour below the floor level. The higher values are to be used for flatter launched slope. The thickness of loose stone at the inner edge should correspond to the required quantity of stone for thickness of launched apron equal to T. The material required for extra thickness of 0.25 T on launched slope should be distributed over the length of apron in the form of a wedge with increasing thickness towards the outer edge.

Example 13.1 For the following data related to a canal headworks, design the profile of the weir section:

Flood discharge	= 3000 m^3/s
Maximum winter flood discharge	= 300 m^3/s
HFL before construction of weir	= 255.5 m
RL of river bed	= 249.5 m
Pond level	= 254.5 m
Lacey's silt factor (for the river bed material)	= 0.9
Permissible afflux	= 1.0 m
Bed retrogression	= 0.5 m
Offtaking canal discharge	= 200 m^3/s
Looseness factor	= 1.1
Concentration factor	= 1.2
Permissible exit gradient	= 1/7

The stage-discharge curve of the river is as shown in Fig. 13.6.

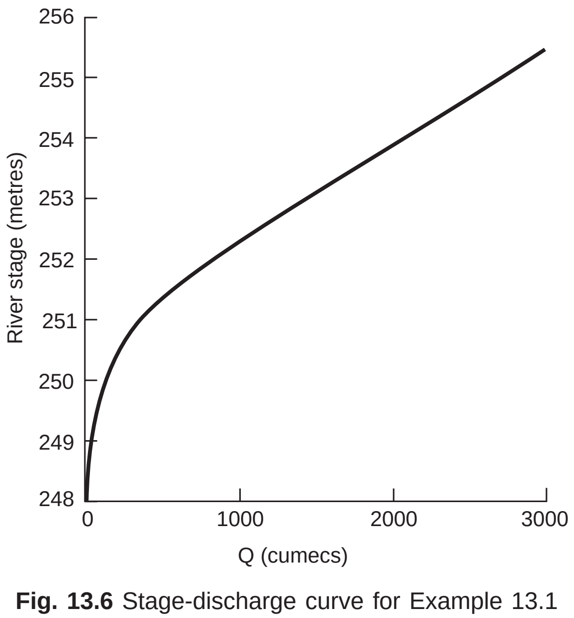


Fig. 13.6 Stage-discharge curve for Example 13.1

Solution:

Lecey's waterway, $P = 4.75\sqrt{Q} = 4.75\sqrt{3000} = 260.17$ m

Actual width of waterways $= P \times$ Losseness factor

$= 260.17 \times 1.1 = 286.19$ m

Lacey's regime velocity $= \left(\dfrac{Qf_1^{\,2}}{140}\right)^{1/6} = \left[\dfrac{3000 \times 0.9 \times 0.9}{140}\right]^{1/6}$

$= 1.61$ m/s

Velocity head $= \dfrac{1.61 \times 1.61}{2 \times 9.81} = 0.13$ m

Level of d/s TEL = 255.5 + 0.13

= 255.63 m (255.13 m after retrogression)

Level of u/s TEL = d/s TEL + Afflux = 255.63 + 1.0

= 256.63 m (256.13 m after retrogression)

Level of u/s HFL = 256.63 – 0.13 = 256.5 m (256.0 m after retrogression)

The undersluice bays should carry maximum of the following three discharges:

(*i*) Twice the discharge of the offtaking canal = 2 × 200 = 400 m^3/s

(*ii*) 20% of the design flood discharge = 0.2 × 3000 = 600 m^3/s

(*iii*) Maximum winter flood discharge = 300 m^3/s

∴ Undersluice discharge = 600 m^3/s

Providing the undersluice crest at the river bed level (*i.e.*, 249.5 m), the total head on the undersluice crest during high flood condition = 256.63 – 249.5 = 7.13 m.

Discharge intensity q through the undersluice portion = 1.71 $(7.13)^{3/2}$ = 32.56 $m^3/s/m$.

On providing two bays of 10.0 m each (and assuming pier contraction coefficient = 0.1),

Discharging capacity of undersluices $= 1.71\,(20 - 0.1 \times 2 \times 1 \times 7.13)\,(7.13)^{3/2}$

$= 604.77\ m^3/s = 604\ m^3/s$ (say)

It should be noted that the pier contraction coefficient would depend on the shape of the pier and may be as small as 0.01 for rounded nose piers.

The weir portion has to carry the remaining flood discharge

$= 3000 - 604 = 2396\ m^3/s$

On providing 18 bays of 15 m each for the weir portion,

Discharge intensity for weir $= \dfrac{2396}{18 \times 15} = 8.87\ m^3/s/m$

Height of TEL over the weir crest $= \left(\dfrac{8.87}{1.84}\right)^{2/3} = 2.85$ m

Here, the formula $q = 1.84\ k^{3/2}$ has been used since the weir crest is usually not wide enough to behave as a broad crest.

Required level of the weir crest = 256.63 – 2.85 = 253.78 m

Provide weir crest at 253.70 m so that the total discharge through the weir and undersluice

$= 1.84\,(270 - 0.1 \times 17 \times 2 \times 2.93)\,(2.93)^{3/2} + 604$

$= 2401 + 604$

$= 3005\ m^3/s$ against the required value of $3000\ m^3/s$

and clear waterway = 270 + 20

= 290 m against the required value of 286.19 m

Thus, the requirements of discharge capacity and clear waterway are met by providing 18 weir bays (with weir crest at 253.70 m) of 15 m each and two undersluice bays (with crest at 249.5 m) of 10 m each.

Required height of undersluice gates = 254.5 – 249.5 = 5.0 m

Required height of weir shutters = 254.5 – 253.7 = 0.8 m

Hydraulic jump calculations for weir sections:

Quantity	*High flood condition*	*Pond level condition*
Discharge intensity, q (in m^3/s/m)	$1.84\,(2.93)^{3/2} \times 1.2 = 11.07^a$	$1.84\,(0.8)^{3/2} \times 1.2 = 1.58^a$
Critical depth, h_c	2.32	0.63
D/S flood level (after retrogression) (m)	255.00	251.25*
D/S TEL (m)	255.13	251.33*
U/S TEL (m)	257.01***	254.58**
Head loss, H_L (m)	1.88	3.25
H_L/h_c	0.81	5.16
E_2/h_c (from Table 9.1)	2.01	2.74
E_2 (m)	4.66	1.73
$E_1 = E_2 + H_L$ (m)	6.54	4.98
Pre-jump depth, h_1(m)	1.07	0.162
Post-jump depth, h_2 (m)	4.32	1.69
Height of the jump = $h_2 - h_1$ (m)	3.25	1.528
Length of concrete floor required (= 5 × height of the jump) (m)	16.25	7.64
Level of the jump = D/S TEL – E_2 (m)	250.47	249.60

The level of the downstream floor should be at or lower than 249.60 m and the length of the downstream horizontal floor should be equal to or more than 16.25 m. However, the river bed is at 249.0 m. Therefore, provide downstream horizontal floor of length 17 m at 249.5 m.

Upstream and Downstream Cutoffs:

Concentration factor is accounted for in the computations of scour depth for determination of depths of cutoffs. Using Eq. (13.3), the depth of scour below HFL,

$$R = 1.35\left(\frac{11.07 \times 11.07}{0.9}\right)^{1/3} = 6.95 \text{ m}$$

---

[a] after taking into account concentration factor of 1.2.

* At the pond level, total discharge through weir and undersluice bays

$$= 1.84\,(270 - 0.1 \times 2 \times 17 \times 0.8)\,(0.8)^{3/2} + 1.71\,(20 - 0.1 \times 2 \times 1 \times 5)\,(5)^{3/2}$$
$$= 715.15 \text{ m}^3\text{/s}$$

∴ Corresponding river stage = 251.75 m

∴ D/S flood level (after retrogression) = 251.25 m

For Q = 715.15 m^3/s,

Lacey's regime velocity = 1.267 m/s

∴ Velocity head = 0.08 m

Level of D/S TEL = 251.25 + 0.08 = 251.33 m

** Level of U/S TEL = 254.50 + 0.08 = 254.58 m

*** Total head over the crest (for q = 11.07 m^3/s/m) = $(11.07/1.84)^{3/2}$ = 3.31 m

Level of U/S TEL = 253.7 + 3.31 = 257.01 m

RL of bottom of the scour hole on the upstream side

= Upstream HFL – 1.5R

= 256.5 – 1.5 × 6.95 = 246.08 m

Therefore, provide upstream cutoff up to the elevation of 246.0 m. Further, RL of bottom of the scour hole on the downstream side

= downstream HFL – 2R

= 255.5 – 2 × 6.95 = 241.6 m

Therefore, provide downstream cutoff up to the elevation of, say, 241.5 m.

Then, depth of the downstream cutoff, d = 249.5 – 241.5 = 8.0 m

(The permissible exit gradient of 1/7 indicates absence of boulder material in the bed and one may, therefore, provide sheet piles instead of concrete cutoff.)

Maximum static head = Pond level – D/S floor level = 254.5 – 249.5 = 5.0 m

Using Eq. (13.4), the exit gradient, $G_E = \frac{5.0}{8.0}\frac{1}{\pi\sqrt{\lambda}} = \frac{1}{7}$

$\therefore \quad \lambda = 1.94$

Further, $\lambda = 0.5\left(1 + \sqrt{1 + (b/d)^2}\right) = 1.94$

$\therefore \quad b = 21.61$ m

Total impervious floor length:

Downstream horizontal floor = 17 m

Horizontal length of the downstream glacis with a slope of 1(V) : 3(H)

= 3 (253.70 – 249.5) = 12.6 m

Let the crest width be 2 m.

Horizontal length of the upstream slope (1(V):2(H)) of the weir

= (253.7 – 249.5) × 2 = 8.4 m

After providing these essential floor lengths, the total imervious floor is already 40 m long. Providing 2.5 m long upstream horizontal floor, the total length of the impervious floor is 42.5 m, which is more than the required value of 21.61 m from the exit gradient consideration.

Loose protection on upstream and downstream:

Concentration factor is not to be accounted for in computation of scour depth for designing loose protection.

Discharge intensity of weir at high flood = 1.84 $(2.93)^{3/2}$ = 9.23 $m^3/s/m$

Depth of scour below HFL, $R = 1.35\left(\frac{9.23 \times 9.23}{0.9}\right)^{1/3} = 6.15$ m

$\therefore$ RL of bottom of scour hole on the upstream side

= upstream HFL – 1.5R

= 256.5 – 1.5 × 6.15 = 247.28 m

Depth of scour below river bed = 249.5 – 247.28 = 2.22 m

Similarly, the depth of scour below the river bed on the downstream

$$= 249.5 - (\text{downstream HFL} - 2R)$$

$$= 249.5 - (255.5 - 2 \times 6.15) = 249.5 - 243.2 = 6.3 \text{ m}$$

Thus,

Length of upstream block protection ≥ 2.22 m (Provide 2.3 m)

Length of upstream launching apron = 2 × 2.22 = 4.5 m (say)

Length of the downstream block protection ≥ 1.5 × 6.3 m (Provide 10 m)

Length of downstream launching apron = 1.5 × 6.3 = 10 m (say)

The line sketch of the designed weir profile is shown in Fig. 13.7. Following the procedure of Example 10.2, one can determine the floor thickness at various locations of the weir section. Similarly, one can determine the profile and design the floor thickness of the undersluice portion.

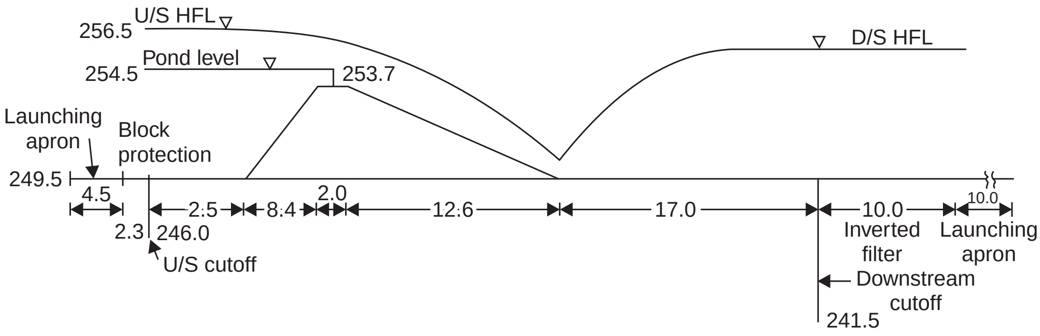


Fig. 13.7 Weir profile for Example 13.1

13.8. DIVIDE WALL

The divide wall is constructed parallel (or nearly parallel) to the canal head regulator. It separates the main weir bays from the bays of the undersluice as shown in Fig. 13.1. The wall extends on both sides of the weir. Extension of the divide wall towards the downstream of the weir avoids cross-flow in the immediate vicinity of the structure which, otherwise, may cause objectionable scour. The divide wall is usually extended up to the end of either the impervious floor or the loose apron on the downstream side. The divide wall serves the following purposes:

(*i*) It isolates the canal head regulator from the main river flow and creates a still pond of water in front of the canal head regulator. This results in deposition of sediment in the pocket and entry of relatively sediment-free water into the offtaking canal. It also improves scouring of the undersluices by ensuring straight approach.

(*ii*) It separates the weir floor from the floor of the undersluices which is at a lower level than the weir floor.

(*iii*) If the main current has a tendency to move towards the bank opposite to the canal head regulator, the weir forces the water towards the canal head regulator. This causes cross-currents which may damage the weir. Under such adverse flow conditions, additional divide walls at equal intervals along the weir are provided to keep the cross-currents away from the weir.

When only one canal takes off from a river, the length of the divide wall should be half to two-thirds the length of canal regulator (1). When more than one canal takes off from the same bank, the divide wall should extend a little beyond the upstream end of the canal farthest from the weir (1). Some experimental studies have shown that a slight divergence of the divide wall from the regulator improves its efficiency. This divergence should not exceed 1 in 10. To reduce the scour at the nose of the divide wall, the nose end of the wall is given a slope of 3(*V*):1(*H*).

The divide wall is generally constructed as a strong masonry wall with a top width of about 1.5 to 2.25 m and checked for safety for the following two conditions:

(*i*) For low stage of the river, the water levels on the two sides of the walls are the same but the silt pressure is assumed to correspond to the sediment deposit up to full pond level on the pocket side.

(*ii*) For the high stage of the river, the undersluices are discharging. At this condition, the water levels on the two sides are assumed to be different; the weir side level being higher by about 1.0 m.

It the river curvature is not favourable to sediment-free entry of water into the offtaking canal by inducing convex curvature opposite the head regulator, a second pocket of river sluice adjoining the undersluices improves flow conditions considerably. Such provision is useful in case of wide rivers to guide the river to flow centrally, minimising cross-flow, and prevent shoal formation in the vicinity of the head regulator. The location and layout of the river sluice should be decided by model studies for satisfactory performance (1).

13.9. FISH LADDER

Various kinds of fish are present in large rivers. Most of these fish migrate from the upstream to downstream in the beginning of the winter in search of warmth and return upstream before the monsoon for sediment-free water. While constructing a weir across a river, steps have to be taken to allow for the migration of fish. For this purpose, a narrow opening between the divide wall and the undersluices (where water is always present) is provided. Most fish can travel upstream, if the velocity of water does not exceed about 3.0 m/s. Hence, baffles or staggering devices are provided in the narrow opening adjacent to the divide wall. These openings are called fish ladder (or fishways or fish pass), (Fig. 13.8). It is advisable to know the requirements of the fish of the river, and the design of fish ladder should take into account these requirements.

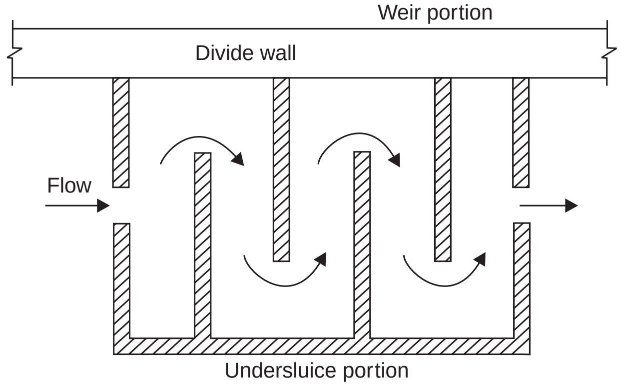


Fig. 13.8 Typical plan of a fish ladder

13.10. CANAL HEAD REGULATOR

A canal head regulator is required to serve the following functions:

(*i*) To regulate the discharge into the offtaking canal, and

(*ii*) To control the entry of sediment into the canal.

The head regulator is usually aligned at an angle of 90° to 110° (Fig. 13.9) to the barrage axis (5). This orientation minimises entry of sediment into the offtaking canal and prevents backflow and stagnation zones in the undersluice pocket upstream of the regulator. The discharge through the regulator is controlled by gates. Steel gates of 6 to 8 m spans are generally used. However, larger spans can also be used in which case the gates are operated by electric winches.

The pond level in the undersluice pocket, upstream of the canal head regulator, is obtained by adding the working head of about 1.0 to 1.2 m to the designed full supply level of the canal. The level of the crest of the head regulator is obtained by subtracting from the pond level, the head over the crest required to pass the full supply discharge in the canal at the specific pond level. The crest of the regulator is always kept higher than the cill of the undersluices to prevent entry of sediment into the canal. If a sediment excluder is also provided in the undersluice pocket, the level of the crest of the head regulator should be decided keeping in view the design requirements of the sediment excluder in addition to the requirements of waterway, and the working head available.

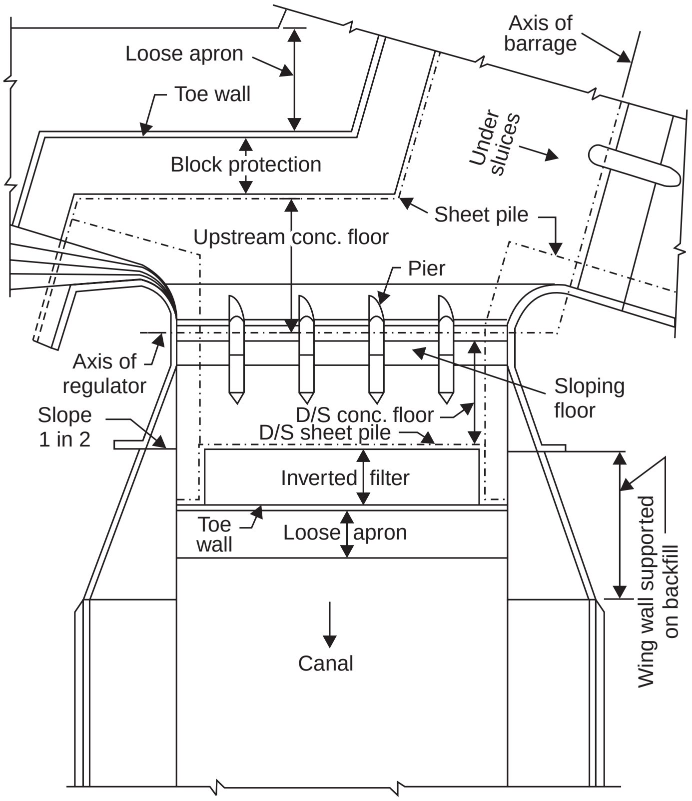


Fig. 13.9 Typical plan of a head regulator (6)

The width of waterway in the canal head regulator should be such that the canal can be fed its full supply with about 50 per cent of the working head provided (4). If the required width of waterway at the head regulator is more than the bed width of the canal, a converging transition is provided downstream of the regulator to attain the required canal width. The required head over the crest, H, for passing a discharge Q with an overall waterway L is worked out from Eq. (13.2).

The height of the gates is equal to the difference of the pond level and the crest level of the regulator. But during high floods, the water level in the river would be much higher than the pond level, and the flood water may spill over the gates. It would, obviously, be very uneconomical to provide gates up to the HFL. Besides the cost of heavier gates, the machinery required to operate them under large water pressures would also be expensive. To prevent such spilling of flood water into the canal, an RCC breast wall (Fig. 13.10) between the pond level and HFL, and spanning between adjacent piers is always provided. With this provision, the gate opening between the crest level and pond level is fully open when the gate is raised up fully, *i.e.,* up to the pond level. The opening is fully closed when the gate is lowered to the crest of the regulator.

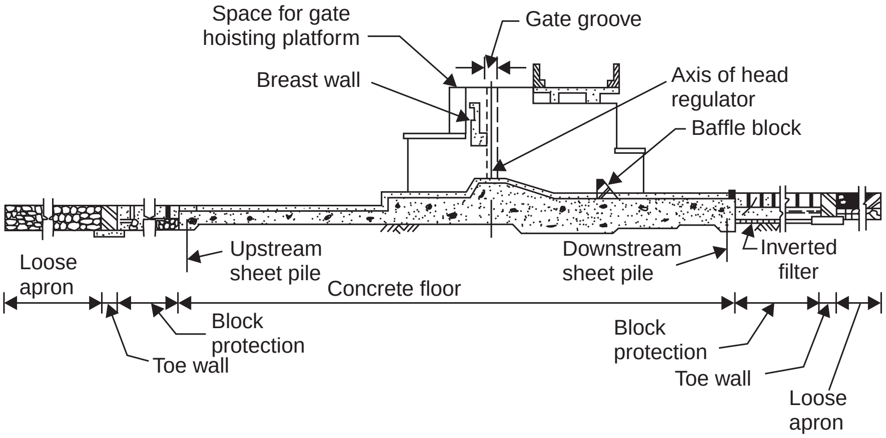


Fig. 13.10 Typical section of a head regulator (6)

Once the crest level, waterway, number of spans, and thickness of piers have been fixed, the head regulator is designed using the principles of weir design. The canal is generally kept closed when the highest flood passes through the river. This, obviously, would be the worst static condition and, hence, the floor thickness must be able to resist uplift pressures under this condition. The exit gradient for this condition should also be within safe limits. For economic reasons, the floor is designed such that it can support the uplift pressure by its weight and also the bending strength. For this purpose, the piers may have to be extended up to the floor to provide necessary support to the upward bending slab.

In the jump trough region, the worst condition of uplift may occur when some discharge is passing into the canal. The safety of this part of the floor should, therefore, be checked for different discharges including the maximum one also. The extension of the concrete floor upstream of the undersluices up to the end of the head regulator also reduces the uplift pressures on the downstream floor of the regulator.

A bridge and a working platform (for the operation of gates) are also constructed across the head regulator.

13.11. SEDIMENT CONTROL IN CANALS

Sediment entering into an offtaking canal, if excessive, causes silting, and thus, reduces canal capacity. Even the fine suspended sediment, in power canals, would cause damage to the turbine blades and, therefore, needs removal prior to the canal water entering the power plant. As such, it is important to control amount of sediment entering into the offtaking canal. In all diversion structures, therefore, provision of adequate preventive or curative measures for sediment control is essential. If a canal offtakes from the outer side of a curved reach of a river, it draws much less sediment than the one offtaking from the inner side. This is due to the secondary flow which develops along the curved reach of a river.

Entry of sediment into the offtaking canal can be controlled by one of the following three methods of barrage regulation:

(*i*) Still pond method,

(*ii*) Semi-open flow method, and

(*iii*) Wedge-flow method.

In the still pond method of the barrage regulation the undersluices are kept closed while the canal is taking its supplies, and the surplus water, if available, flows through some weir bays. This causes considerable reduction in the velocity of flow in the undersluice pocket which results in deposition of coarse sediment in the pocket and water containing much less (or no) sediment is drawn by the offtaking canal. However, with increasing amount of deposition of sediment in the pocket, the offtaking canal may start withdrawing sediment as well. At this stage, the canal is closed and the deposited sediment is flushed downstream of the undersluices by opening the undersluices. This methods has been found satisfactory but requires closing of canal at some regular interval.

Alternatively, the undersluice gates are kept partially open while the canal is withdrawing its supplies. This semi-open flow method of barrage regulation results in continuous flushing of sediment through the undersluices while the canal is withdrawing top layer water which contains much less sediment. Besides requiring surplus water, this method results in two streams – one entering the canal and the other entering the undersluices which may generate enough turbulence in the pocket upstream of the head regulator and thus bring sediment into suspension. The suspended sediment may enter the canal. Also, the method would not work satisfactorily if there is no surplus water. This method is, therefore, not suitable except during floods.

In the wedge-flow system of barrage regulation, the undersluices near the divide wall are opened more while those near the head regulator are opened less. This results in wedge-like flow cross-section which causes favourable curvature of flow in the undersluice pocket, and thus, reduces the amount of sediment entering the canal.

When the stream is carrying high sediment load, the sediment entry into the offtaking canal can be best checked by closure of the canal itself.

One of the most commonly used preventive measures is the sediment excluder (also known as silt excluder). The excluder is constructed in the river bed in front of the canal head regulator to prevent, as far as possible, excess sediment entering into the offtaking canal. Figure 13.11 shows a typical layout of a tunnel-type sediment excluder. The tunnels of the excluder, used for flushing the sediments, are parallel to the axis of the canal head regulator and are of different lengths, and the tunnels terminate at the end of the undersluice bays. Some kind of sediment control devices, such as skimming platform (6) and curved wings with sediment vanes (7) are provided in case of channels offtaking from main canals or branch canals for proportionate distribution of sediment. If the offtaking canal has already drawn more sediment, curative measures, such as construction of sediment ejectors, are adopted. A sediment ejector (or extractor), also known as silt ejector, is a curative measure and is constructed in the offtaking canal downstream of the canal head regulator to remove the excess sediment load which has entered the canal. Alternatively, a settling basin can be constructed in the offtaking canal for the purpose of sediment ejection.

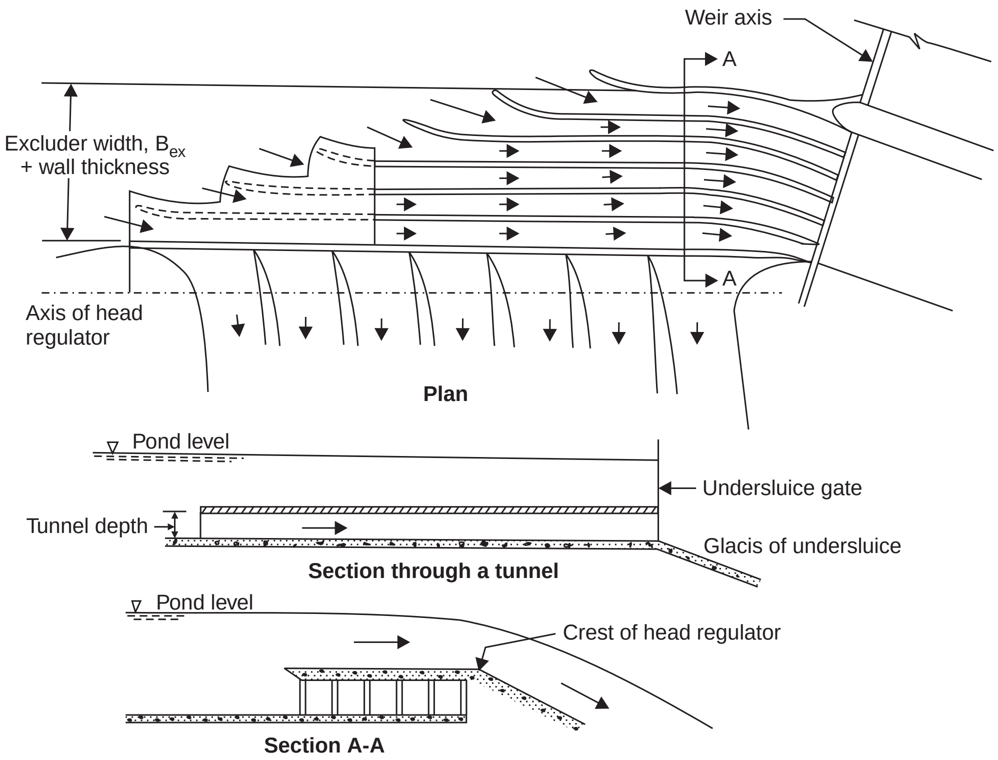


Fig. 13.11 Typical layout of a sediment excluder

13.11.1. Sediment Excluders

The sediment concentration is maximum in the bottom layers of a stream. Tunnel-type sediment excluders (Fig. 13.11) prevent these bottom layers from entering the offtaking canal and allow only the top layers of the stream, containing relatively less sediment, to enter the offtaking canal. Such an excluder was first conceived by Elsden in 1922 and was first constructed by Nicholson for the Lower Chenab canal at Khanki headworks in Punjab. It was followed by another at Trimmu headworks. The tunnel-type sediment excluder, provided in front of the Nangal hydel channel, has been divided into two chambers – upper and lower – such that the heights of the upper and lower chambers are in the ratio of 2 : 1, and each of them has a separate control. The upper chamber is operated only during very high floods. This reduces the escape discharge at low flows.

The present design procedure for designing a sediment excluder is based on thumb rules evolved from past experiences on such structures. The minimum discharge passing through the tunnels of the excluder is kept around 20 per cent of the canal discharge. The self-flushing velocity in the tunnel ranges from 1.8 to 4 m/s depending upon the sediment size.

Usually, 2 to 6 tunnels are provided in an excluder. The tunnels are to be accommodated in the space between the undersluice floor and the crest of the head regulator. The height of the tunnels is, thus, determined keeping in view convenience for inspection and repair, as well as self-flushing velocity. One can now estimate the width of waterway required for tunnels. This width is divided into a suitable number of tunnels such that a whole number of tunnels are accommodated in one undersluice bay. These tunnels are usually of rectangular cross-section and are bell-mouthed at the upstream end.

The water and sediment discharge carried by all the tunnels of an excluder should be the same. Also, the depth of all the tunnels are kept the same. Therefore, in accordance with the resistance and continuity equations, the width of the shorter tunnel will be smaller than that of the longer tunnel so that the head losses are the same in all tunnels.

The length of the tunnel nearest the head regulator must be equal to the length of the head regulator. Other tunnels are successively shorter in length such that the mouth of the one nearer the head regulator comes within the suction zone of the next tunnel so that no dead zone is left between the adjacent tunnels to cause sediment deposition. The excluder designed in this manner is usually tested through model study and is suitably modified, if required.

The design procedure outlined above takes into account the concepts of minimum energy loss and self-flushing velocity. It does not consider the actual sediment transport capacity of the tunnels for given set of conditions. It would, obviously, be more logical to compute the sediment load coming into the undersluice pocket and then design the excluder such that it is capable of carrying that sediment load without causing objectionable deposits of sediment. Garde and Pande (8) have suggested the following design procedure which takes into account the actual sediment transport capacity as well.

The velocity in the excluder tunnels, U_{ex} is chosen to lie between the critical velocity U_c (at which sediment starts moving) and the limit deposit velocity U_L (at which no sediment will be deposited) for the maximum sediment size, d_m. The critical velocity U_c is given as (9)

$$\frac{U_c}{\sqrt{\frac{\Delta\rho_s}{\rho} g d_m}} = 1.6 \left[\frac{R_{ex}}{d_m}\right]^{1/8} \tag{13.5}$$

and the limit deposit velocity U_L is given as (10)

$$U_L = F_1 \sqrt{8g\, R_{ex}(S_s - 1)} \tag{13.6}$$

where, $S_s = \dfrac{\rho_s}{\rho}$

ρ_s = the mass density of sediment,

ρ = the mass density of water,

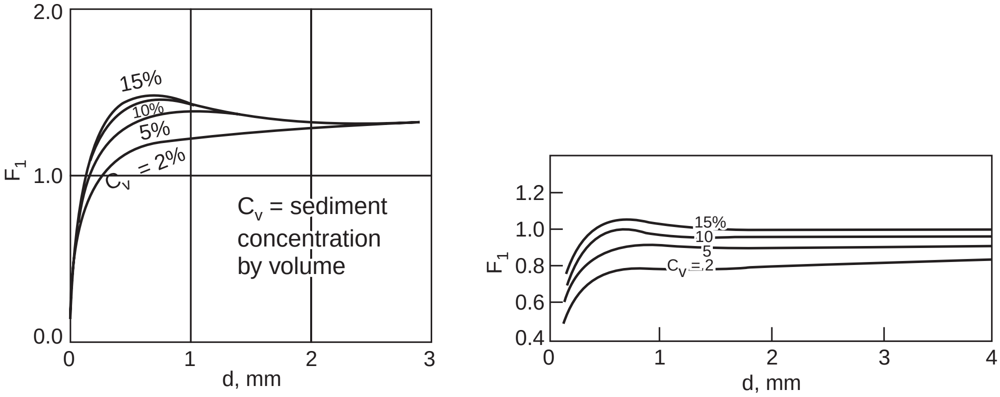
R_{ex} = the hydraulic radius of the excluder tunnel.

and F_1 depends on the concentration and size distribution of sediment as shown in Fig. 13.12. Garde and Pande (8) suggested that for sediment size greater than 0.5 mm, F_1 varies between 0.8 and 1.0, and can be approximated as unity.

If the excluder velocity U_{ex} is greater than U_L there will be no deposition of sediment in the excluder. However, the value of U_L is generally large and, hence, U_{ex} is usually less than

U_L and, therefore, the excluder tunnels are partially blocked. The free flow area in such a case can be calculated from

$$\frac{U_{fex}}{\sqrt{4gR_{fex}}} = \frac{U_L}{\sqrt{4gR_{ex}}} \tag{13.7}$$



(a) For uniform sediment (b) For non-uniform sediment

Fig. 13.12 Limit deposit velocity (10)

where, $$U_{fex} = \frac{Q_{ex}}{B_{ex}D_{fex}} \tag{13.8}$$

and $$R_{fex} = \frac{B_{ex}D_{fex}}{2\,(B_{ex} + D_{fex})} \tag{13.9}$$

Thus, combining Eqs. (13.6 – 13.9) one gets

$$\frac{Q_{ex}^2\left(1+\dfrac{D_{fex}}{B_{ex}}\right)}{gD_{fex}^3B_{ex}^2} = 4F_1^{\,2}(S_s - 1) \tag{13.10}$$

Here, the suffix '*f*' refers to the free (*i.e.,* unblocked) flow area available, and the suffix '*ex*' refers to the excluder. *B, D,* and *Q* are, respectively, the width, depth, and the discharge of the excluder. Using the above equations, D_{fex} can be calculated. Hence, the blockage of the tunnels is obtained as $D_{ex} - D_{fex}$. One can also calculate U_{fex} from Eq. (13.8).

The bed load and suspended load going into the undersluice pocket are worked out by using suitable bed load relation and the velocity and sediment concentration profiles. Thus, the total sediment concentration (by weight) in the tunnels, C_{ex}, can be obtained. The sediment transport capacity of the tunnels C_t (*i.e.*, sediment concentration by weight) must, obviously, be greater than C_{ex}. Further, C_t is obtained from (11).

$$R_e\sqrt{f} = \left[\frac{I}{d/(4R_{fex})}\right]^{s_1} C_t^{\,1/3} \tag{13.11}$$

Here, R_e is the Reynolds number of flow $[= U_{fex}(4R_{fex})/\nu]$, f the Darcy-Weisbach resistance coefficient, and I is a suitable function of d as given in Fig. 13.13. Further, the index s_1 is obtained from

$$s_1 = \frac{1}{0.89 d^{1/3}} \tag{13.12}$$

in which, d (in mm) can be either equal to d_m or slightly less than d_m.

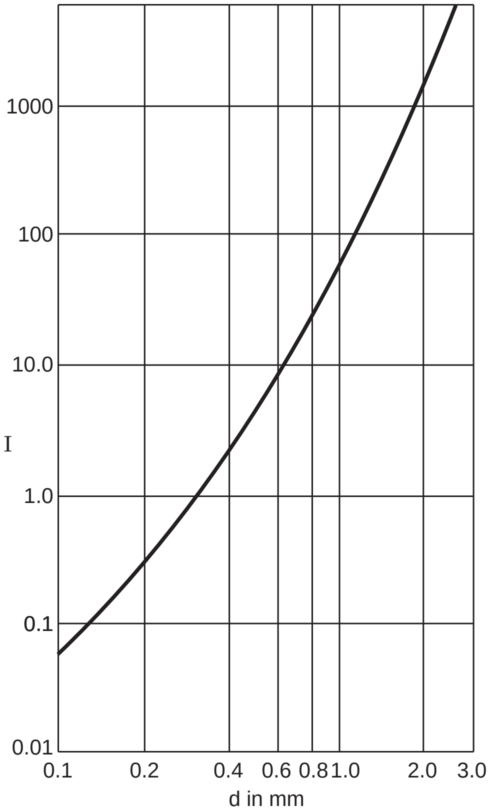


Fig. 13.13 Variation of *I* with *d* (11)

A suitable design for an excluder requires that the blockage be limited to a suitable value, say, about 35 per cent and also C_t is greater than C_{ex} in the excluder tunnels. Ideally, a design which satisfies all the conditions for minimum values of Q_{ex}, B_{ex}, and depth of tunnels will be the most suitable. However, there are some constraints on all these parameters. Therefore, one should choose the best combination satisfying all the requirements and constraints. Some margin of safety should always be provided for in order to account for the uncertainties in the sediment load computations. Following example illustrates the procedure for trial design of a sediment excluder based on the method given by Garde and Pande (8).

Example 13.2 Design a suitable sediment excluder using the following data:

River discharge, Q_R	= 4250 m^3/s
River slope	= 1/4000
Depth at pond level	= 4.33 m
Bed width (Barrage width), B_R	= 920 m
Canal discharge, Q_C	= 240.7 m^3/s

Average grain size of the bed material = 0.37 mm

Maximum size of the bed material = 0.60 mm

Solution:

(*i*) Find shear stress acting on the bed corresponding to the maximum discharge at which pond level is being maintained

$$\tau_o = 1000 \times 9.81 \times 4.33 \times (1/4000)$$
$$= 10.62 \text{ N/m}^2$$

Corresponding to this shear stress, the maximum size of the bed material which can be set into motion can be determined using Eq. (7.3) and is equal to 11.00 mm. As such, the maximum size of the bed material (*i.e.*, 0.6 mm) at the site would move under the given conditions.

(*ii*) Assume some suitable depth of tunnels, t (usually between 1 and 2 m).

Let $t = 1.6$ m

(*iii*) Assume excluder discharge Q_{ex} (between 10 and 20 per cent of canal discharge, Q_c).

Let $Q_{ex} = 48\text{m}^3/\text{s}$ (approx. 20 per cent of Q_c)

(*iv*) Find the critical velocity U_c and limiting velocity U_L for the maximum size of the bed material which is movable (step (*i*)). Using Eqs. (13.5) and (13.6), and assuming R_{ex} = 0.7 m and $F_1 = 1.0$.

$$U_c = 1.6\sqrt{1.65 \times 9.81 \times 0.6 \times 10^{-3}}\left[\frac{0.7}{0.6 \times 10^{-3}}\right]^{1/8} = 0.38 \text{ m/s}$$

$$U_L = 1.0\sqrt{8 \times 9.81 \times 0.7\,(2.65 - 1)} = 9.52 \text{ m/s}$$

(*v*) Choose excluder velocity U_{ex} such that it is greater than U_c and in the neighbourhood of U_L but not exceeding 3 m/s.

Let $U_{ex} = 2.5$ m/s.

$$\therefore \quad B_{ex} = \frac{48}{2.5 \times 1.6} = 12.0 \text{ m}$$

$$R_{ex} = \frac{12 \times 1.6}{2\,(12 + 1.6)} = 0.706 \text{ m}$$

$\therefore$ Revised $U_L = 1.0\sqrt{8 \times 9.81 \times 0.706\,(2.65 - 1)} = 9.56$ m/s

and revised $U_c = 1.6\sqrt{1.65 \times 9.81 \times 0.6 \times 10^{-3}}\left[\dfrac{0.706}{0.6 \times 10^{-3}}\right]^{1/8} = 0.493$ m/s

(*vi*) Determine the width B_{RE} which contributes the discharge to the undersluice pocket using the relation

$$B_{RE} = (Q_c + Q_{ex})\frac{B_R}{Q_R}$$
$$= (240.7 + 48)\frac{920}{4250} = 62.5 \text{ m}$$

(*vii*) Determine the amount of bed load coming into the tunnels fed by the stream width B_{RE} as shown in the following steps:

$$4250 = \frac{1}{n}(920 \times 4.3)\left[\frac{920 \times 4.33}{(920 + 2 \times 4.33)}\right]^{2/3}\left(\frac{1}{4000}\right)^{1/2}$$

$\therefore$ $$n = 0.039$$

and $$n_s = \frac{(0.37 \times 10^{-3})^{1/6}}{25.6} = 0.0105$$

From Eq. (7.31),

$$\left(\frac{0.0105}{0.039}\right)^{3/2}\left(\frac{920 \times 4.33}{920 + 2 \times 4.33}\right)\left(\frac{1}{4000}\right)\left(\frac{1}{1.65 \times 0.37 \times 10^{-3}}\right)$$

$$= 0.047 + 0.25\,\frac{q_B{}^{2/3}}{(2.65 \times 1000)^{2/3}(9.81)(0.37 \times 10^{-3})(1.65)^{1/3}}$$

or $$0.2454 = 0.047 + 0.3044 q_B{}^{2/3}$$

$\therefore$ $$q_B = 0.526 \text{ N/m/s}$$

$\therefore$ Total bed load coming into the excluder pocket = 0.526 × 62.50 = 32.875 N/s

(*viii*) Determine the suspended load coming into the tunnels of the excluder. An approximate simple equation of Engelund's curve (12) relating suspended load with the river discharge is

$$\frac{Q_s}{Q_R} = 0.48 \times 10^{-4}\left(\frac{u_*}{w}\right)^{4.36}$$

$$u_* = \sqrt{\tau_o / \rho} = \sqrt{\frac{10.62}{1000}} = 0.103 \text{ m / s}$$

w = fall velocity of the bed material

= 0.05 m/s (for d = 0.37 mm)

$\therefore$ $$Q_s = 4250 \times 0.48 \times 10^{-4}\left(\frac{0.103}{0.05}\right)^{4.36} = 4.765 \text{ m}^3\text{/s}$$

Suspended load going into the pocket, Q_{sp} is obtained from

$$Q_{sp} = \frac{Q_s}{B_R} \times B_{RE}$$

$$= \frac{4.765}{920} \times 62.5 = 0.324 \text{ m}^3\text{/s}$$

For determining the fraction of suspended load Q_{sp} entering the tunnel, *i.e.*, $Q_{st,}$ one can use curves of Fig. 13.14 (8). For the present problem,

$$\frac{t}{D} = \frac{1.6}{4.33} = 0.37$$

$$z = \frac{w}{u_* k} = \frac{0.05}{0.103 \times 0.4} = 1.2$$

$$k\,\frac{\bar{u}}{u_*} = 0.4\,\frac{4250 / (920 \times 4.33)}{0.103} = 4.14$$

$\therefore$ $$\frac{Q_{st}}{Q_{sp}} = 0.73$$

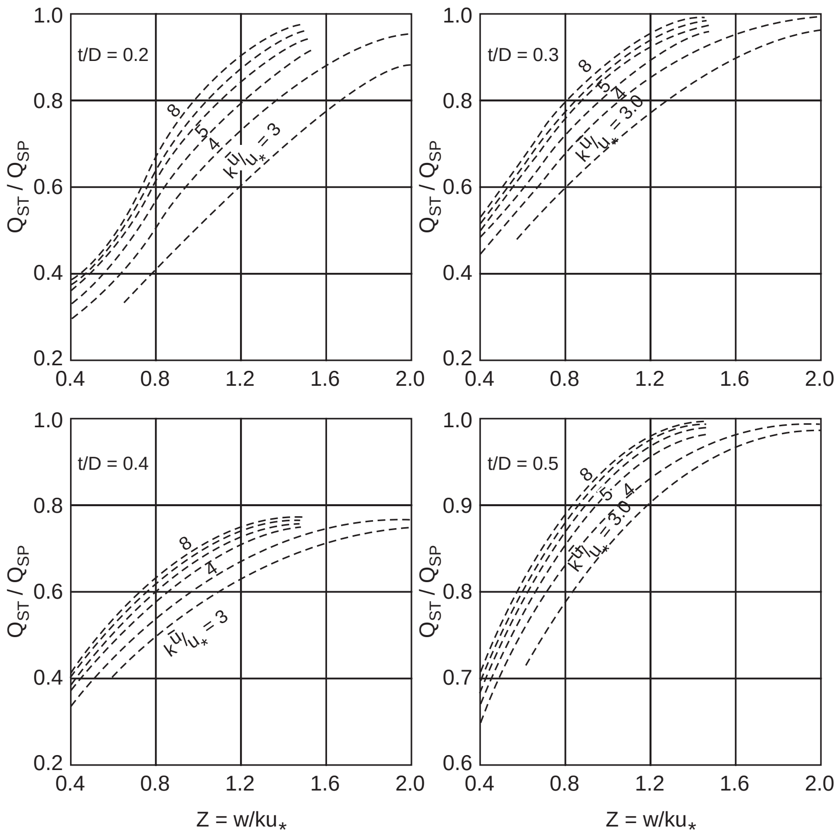


Fig. 13.14 Suspended load entering excluder tunnels

$$\therefore \qquad Q_{st} = 0.73 \times 0.324 \times 2.65 \times 9810 = 6148.7 \text{ N/s}$$

$$C_{ex} = \frac{(6148.75 + 32.875)}{48 \times 9810} = 0.013 \text{ (by weight)}$$

Using Eq. (13.10), one gets

$$\frac{12 + D_{fex}}{D_{fex}^3} = 48.56$$

$$D_{fex} = 0.64 \text{ m}$$

$$R_{fex} = (12 \times 0.64)/[2\ (12 + 0.64)] = 0.304 \text{ m}$$

$$R_e = [48/(12 \times 0.64)]\ [4 \times 0.304]\ [10^6] = 7.6 \times 10^6$$

Taking $d = 0.5$ mm (less than d_m which is 0.6 mm)

$$s_1 = \frac{1}{0.89\,(0.5)^{1/3}} = 1.416 \text{ and } I = 5.2$$

Hence, from Eq. (13.11)

$$7.6 \times 10^6 \sqrt{0.02} = \left[\frac{5.2}{0.5 \times 10^{-3} / (4 \times 0.304)}\right]^{1.416} C_t^{\,1/3}$$

$\therefore \qquad C_t = 4.66$

$\therefore \qquad C_t > C_{ex}$

Since this trial design has yielded relatively large blockage one needs to make another trial. The final design should be such that the sum of the clear waterway, *i.e.*, B_{ex} and thickness of tunnel walls is equal to the width of whole number of undersluice bays.

13.11.2. Sediment Ejector

Sediment ejectors too take the advantage of the concentration distribution in a vertical by ejecting the near-bed water layers having the largest sediment concentration from the canal at a suitable location downstream of the head regulator. The approach channel upstream of the ejector should, preferably, be straight since a curved approach disturbs the uniform distribution of flow and sediment concentration across the channel in front of the ejector. The approach channel can be designed so that the suspended particles may move to lower layers. This will improve the efficiency of the ejector. The ejector should neither be too near nor too far from the head regulator. If the ejector is located too near the regulator, the residual turbulence may keep most of the sediment particles in suspension and, thus, prevent their ejection to the desired extent. If the ejector is sited too far downstream of the regulator, the sediment may get deposited between the regulator and the ejector and, thus, reduce the channel capacity. Besides, a longer reach (between the regulator and the ejector) will have to be wide enough to carry larger amount of discharge on account of the escape discharge required at the ejector.

A schematic diagram of the tunnel-type sediment ejector is shown in Fig. 13.15. The main components of an ejector include a diaphragm, tunnels, control structure, and an outfall channel. The diaphragm is so shaped that it causes least disturbance to the sediment distribution in the bottom layers of flow upstream of the ejector. Diaphragm level is fixed keeping in view the desired sediment size to be ejected, upstream and downstream bed levels of canal, size of tunnels, and the thickness of diaphragm. The lower side of the upstream end of the diaphragm is bell-mouthed. The canal bed is depressed below the ejector to facilitate further ejection of sediment. The ejector spans the entire width of the canal and is divided into a number of main tunnels which, in turn, are subdivided with turning vanes which gradually converge so as to accelerate the escaping flow. The width of ejector tunnels may be varied in order to keep the discharges in all the tunnels to be approximately the same. Generally, 10 to 20 per cent of the full supply discharge of the canal is adequate to remove the desired size and amount of sediment as well as for flushing individual channels of the ejector. The tunnel dimensions at the entry and exit should be such that the resulting flow velocities would be adequate to carry the sediments of the desired size. In addition, the sub-tunnels should be contracted such that the exit velocities further increase by 10-15 per cent and should be in the range of 2.5 to 6 m/s depending on the size of the sediment to be ejected. The depth of tunnels should be kept about 1.8 to 2.2 m to facilitate inspection and repair. The ejector discharge is controlled by regulator gates. The outflow from the ejector is led to a natural drainage through an outfall channel which is designed to have a self-cleaning velocity. Sufficient drop between the full supply level of the outfall channel at its tail end and the normal high flood level of the natural stream is necessary for efficient functioning of the channel. The present design practice for sediment ejectors is also empirical with tunnel height being kept 20-25% of the depth of flow with an escape discharge of about 20 per cent of the full supply discharge of the canal downstream of the ejector. It is, therefore, essential that the proposed design be always model-tested before construction.

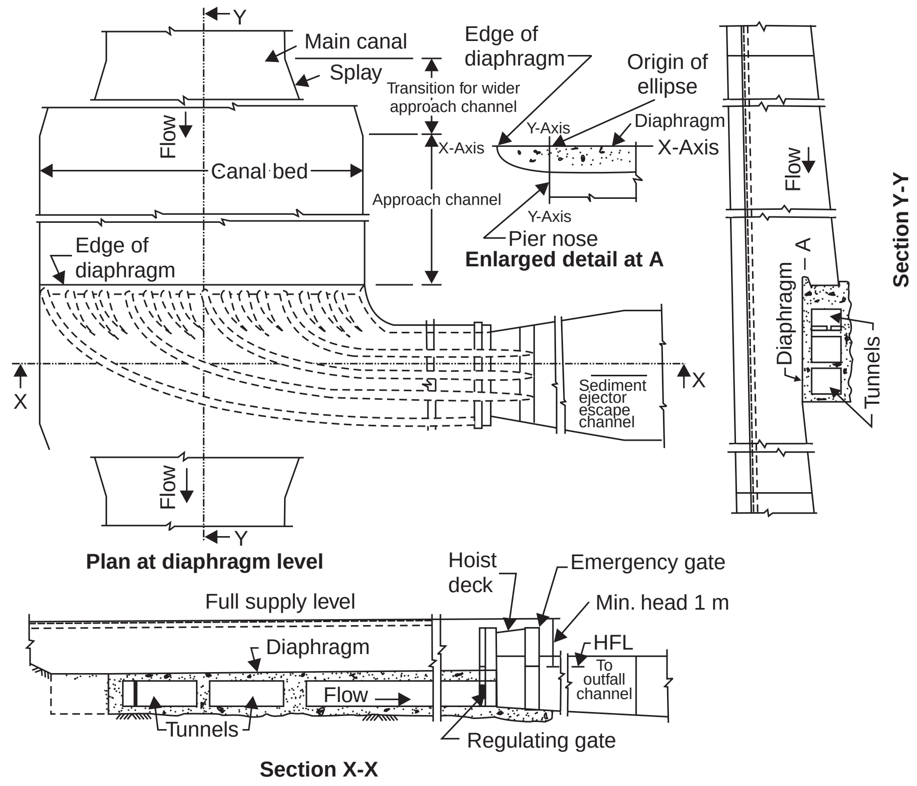


Fig. 13.15 Typical layout of a sediment ejector

The efficiency (E) of the sediment ejector can be defined as (13)

$$E = \frac{I_u - I_d}{I_u} \times 100 \text{ per cent} \tag{13.13}$$

Here, I_u and I_d refer, respectively, to the silt concentration in the canal at the upstream and the downstream of the ejector. A similar definition of efficiency can be used for sediment excluder too.

Recently, Vittal and Shivcharan Rao (14) suggested a rational method to decide on the height of the ejector diaphragm. The method is based primarily on the premises that : (*i*) the suspended load above the diaphragm only passes the ejector and enters the canal downstream of the ejector, and (*ii*) this suspended load (above the diaphragm) should be equal to the total sediment load transport capacity (*i.e.*, the sum of the bed load and suspended load) of the canal downstream of the ejector, if it is to be neither silted nor scoured. In addition, the proposed method also assumed uniform size of sediment and validity of Rouse's equation, Eq. (7.34), for the variation of sediment concentration along a vertical and the logarithmic variation of velocity in sediment-laden flows. Also, river-bed material of coarser size is assumed to have been removed by the sediment excluder. The development of the method is as follows (14).

If X is the ratio of the transport rate of the suspended load above the diaphragm (of height h) to that in the total depth of flow D, one can write (see Art. 7.5.2)

$$X = \frac{q_{sh}}{q_{st}} = \frac{\int_{h/D}^{1.0} \left(\frac{C}{C_a}\right)\left(\frac{u}{u_*}\right) d\,(y/D)}{\int_{2d/D}^{1.0} \left(\frac{C}{C_a}\right)\left(\frac{u}{u_*}\right) d(y/D)} \tag{13.14}$$

in which

$$\frac{C}{C_a} = \left(\left(\frac{D-y}{y}\right)\left(\frac{2d}{D-2d}\right)\right)^Z \tag{13.15}$$

$$\frac{u}{u_*} = \frac{2.3}{k} \log \frac{y}{y'} \tag{13.16}$$

where

u = the velocity at a height y above the bed,

C = the sediment concentration at y,

C_a = the reference concentration at $y = a = 2d$,

d = the size of the sediment,

$Z = w\,u_*/k$,

w = fall velocity of the sediment particles,

u_* = the shear velocity,

k = von Karman's constant, and

$y' = d/30$.

Combining Eqs. (13.14–13.16) and then rearranging the terms,

$$X = \frac{\int_{h/D}^{1.0} \left[\left(\frac{1-\frac{y}{D}}{y/D}\right)\left(\frac{2d/D}{1-\frac{2d}{D}}\right)\right]^Z \frac{2.3}{k} \log\left(\left(\frac{y}{D}\right)\left(\frac{D}{d}\right)(30)\right) d\,(y/D)}{\int_{2d/D}^{1.0} \left[\left(\frac{1-\frac{y}{D}}{y/D}\right)\left(\frac{2d/D}{1-\frac{2d}{D}}\right)\right]^Z \frac{2.3}{k} \log\left(\left(\frac{y}{D}\right)\left(\frac{D}{d}\right)(30)\right) d\,(y/D)} \tag{13.17}$$

Equation (13.17), therefore, suggests that X should depend on h/D, Z and D/d. On substituting the values of h/D (between 0 and 1), Z (between 0.03125 and 4.0) and D/d (between 500 and 10,000) in the final combined equation, they obtained variation of X with h/D and Z (as the third variable) as shown in Fig. 13.16. The parameter D/d did not seem to affect the relationship of Fig. 13.16.

The value of X can also be obtained indirectly as follows (14): By definition, the total sediment transport rate (Q_{st}) equals the sum of the bed load transport rate, Q_b and suspended load transport rate, Q_s.

$$Q_{st} = Q_b + Q_s \tag{13.18}$$

$$Q_{st_1} = Q_{b_1} + Q_{s_1} \tag{13.19}$$

and

$$Q_{st_2} = Q_{b_2} + Q_{s_2} \tag{13.20}$$

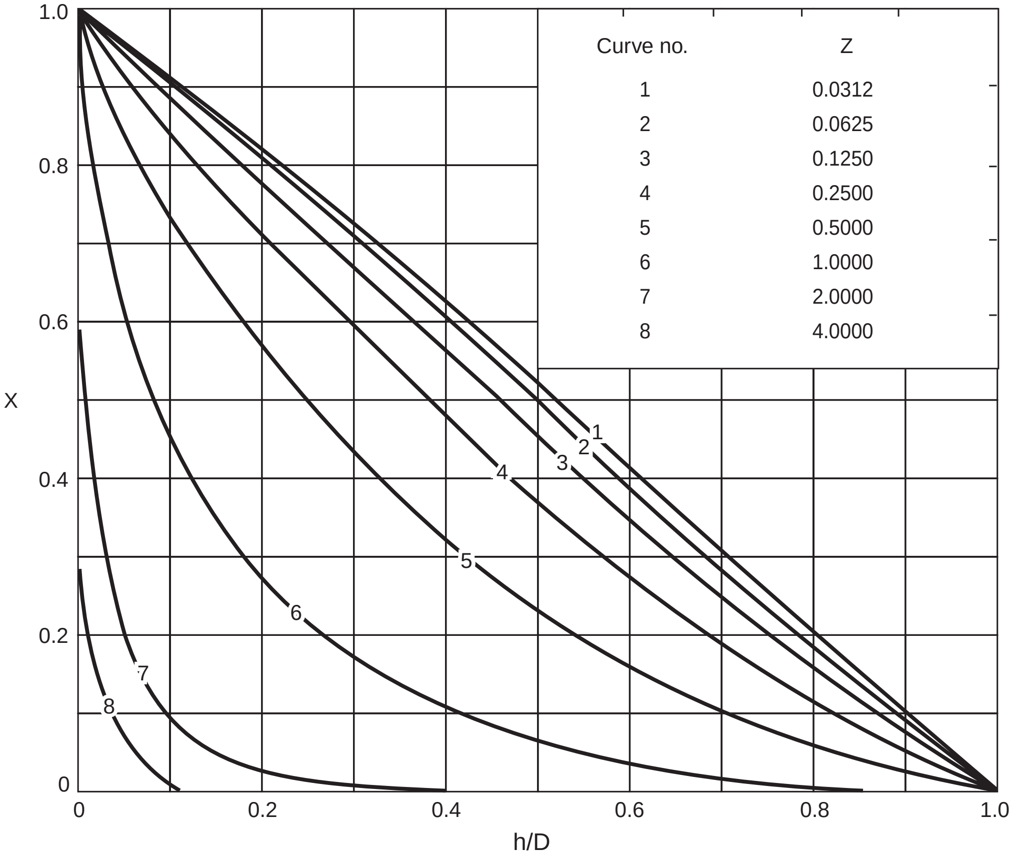


Fig. 13.16 Variation of X with h/D and Z for the design of sediment ejector

Here, suffix 1 and 2 represent canal sections upstream and downstream of the ejector. For non-silting and non-scouring conditions in the canal downstream of the ejector, one can write

$$B_1 q_{sh} = Q_{st_2} \tag{13.21}$$

On dividing Eq. (13.19) with Q_{st_2},

$$\frac{Q_{st_1}}{Q_{st_2}} = \frac{Q_{b_1}}{Q_{st_2}} + \frac{Q_{s_1}}{Q_{st_2}}$$

$$\frac{B_1 q_{st_1}}{B_2 q_{st_2}} = \frac{B_1 q_{b_1}}{B_2 q_{st_2}} + \frac{B_1 q_{s_1}}{B_1 q_{sh}}$$

$$\frac{q_{st_1}}{q_{st_2}} = \frac{q_{b_1}}{q_{st_2}} + \frac{1}{X}\frac{B_2}{B_1} \tag{13.22}$$

Due to the disturbance at the diaphragm entrance, more sediment enters the ejector and, therefore, q_{st_2} is reduced to $K\, q_{st_2}$ with K being less than 1. There is no information about the variation of K. Equation (13.22) can now be rewritten as follows:

$$\frac{q_{st_1}}{Kq_{st_2}} = \frac{q_{b_1}}{Kq_{st_2}} + \frac{1}{X}\frac{B_2}{B_1} \tag{13.23}$$

Additional water discharge (*i.e.*, escape discharge Q_e) is required in the canal between the canal head regulator and the ejector so as to carry the sediment through ejector ducts into the escape channel without affecting the full supply discharge of the canal downstream of the ejector. The ratio of the escape discharge Q_e (= $Q_2 - Q_1$) to the discharge Q_1 in the canal upstream of the ejector can be expressed as

$$\frac{Q_e}{Q_1} = \frac{\int_0^{h/D}\left(\frac{u}{u_*}\right) d\,(y/D)}{\int_0^{1}\left(\frac{u}{u_*}\right) d\,(y/D)} \tag{13.24}$$

On substituting the value of u/u_* (Eq. 13.16) in Eq. (13.24), one can obtain the variation of Q_e/Q_1 with h/D and D/d. Here, again, it was observed by Vittal and Shivcharan Rao that Q_e/Q_1 is not dependent on D/d and

$$\frac{Q_e}{Q_1} \simeq \frac{h}{D} \tag{13.25}$$

Using Fig. 13.16 and Eqs. (13.23 and 13.25), one can determine the diaphragm height h and the escape discharge Q_e and the canal discharge Q_1 for given data (d, Q_2, B_2, D_2, slope S_2 and the sediment load upstream of the ejector). The steps for their computation are as follows:

1. Assume canal discharge upstream of the ejector, Q_1 to be higher than Q_2 by, say 20%.

$$Q_1 = 1.2\, Q_2$$

2. Design the upstream canal for known Q_1 to carry the known amount of sediment load of known size (see Art. 8.5.2). Thus, determine D_1, B_1 and S_1.
3. Compute $u_* = \sqrt{g\, D_1\, S_1}$ and $Z = \dfrac{w}{u_* k}$
4. Calculate the bed loads q_{b_1} and q_{b_2} using Meyer-Peter equation and also the suspended loads q_{s_1} and q_{s_2} using Rouse's equation and, hence, q_{st_1} and q_{st_2}.
5. Assuming a suitable value of K (less than 1, say 0.9), determine X using Eq. (13.23).
6. Obtain h/D for computed values of X (step 5) and Z (step 3).
7. Obtain Q_e/Q_1 from Eq. (13.25) and, therefore, Q_e for the assumed value of Q_1.
8. New (revised) value of Q_1 is $Q_2 + Q_e$.

If the revised value of Q_1 matches with the assumed value of Q_1 in step 1, the values of h (step 5) and Q_e (step 7) can be accepted. Otherwise, repeat steps 2 to 8 with the new value of Q_1.

13.11.3. Settling Basin

In case of canals carrying fine sediment, the variation of sediment concentration will be almost uniform, *i.e.*, the sediment will be moving more as suspended load than bed load. In such situations, the height of the ejector platform will have to be raised. This would result in much larger escape discharge that may not be desirable. An effective way of removing fine sediment from flowing water in a canal is by means of a settling basin in which flow velocity is reduced considerably by expanding the cross-sectional area of flow over the length of the settling basin, Fig. 13.17. The reduction in velocity, accompanied with reduction in bed shear and turbulence, stops the movement of the bed material, and also causes the suspended material to deposit on the bed of the basin. The material on the bed of the settling basin is suitably removed. The settling basin, a costly proposition, is suitable for power channels carrying fine sediment which may damage the turbine blades. The design of such a settling basin involves estimation of the dimensions of the settling basin and suitable method for removing the material from the bed of the settling basin. For known size (and, hence, fall velocity w) of the sediment particle and the depth of flow D, the time required for the particle on the water surface to settle on the bed of the basin would be (D/w) and if the particle moves with a horizontal velocity of flow U, the required length of the basin should be equal to or more than UD/w. To account for the reduction in fall velocity due to turbulence, the length of the basin so obtained may be increased by about 20%.

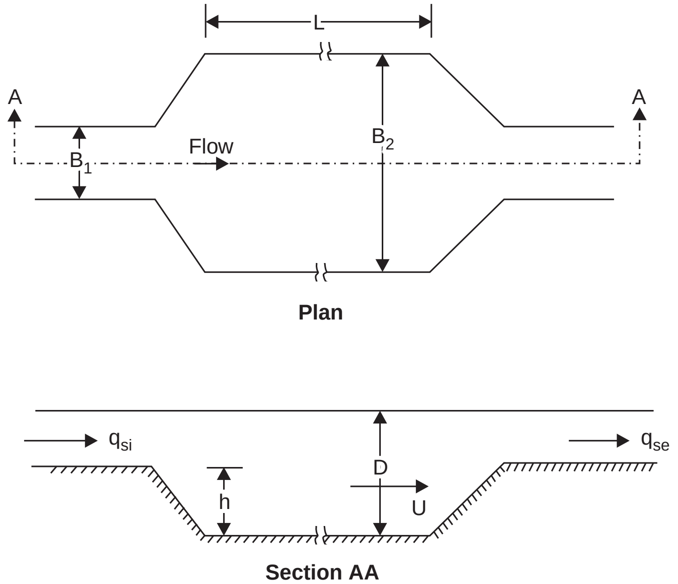


Fig. 13.17 Settling basin - definition sketch

The efficiency of removal of sediment η by a settling basin is defined as

$$\eta = \frac{q_{si} - q_{se}}{q_{si}} = 1 - \frac{q_{se}}{q_{si}} \tag{13.26}$$

Here, q_{si} and q_{se} are, respectively, the amount of sediment of a given size entering and leaving the settling basin in a unit time. Dobbins (15) obtained an analytical solution for the estimation of η by assuming : (*i*) the longitudinal concentration gradient zone, (*ii*) uniform velocity distribution for flow in the basin, and (*iii*) invariant diffusion coefficient over the flow section in the basin. Camp (16) presented Dobbins analytical solution in the form of a graph, Fig. 13.18, between η and (w/u_*) for various values of wL/UD. Here, u_* is the shear velocity ($= \sqrt{gDS}$ in which S is the slope of the settling basin). Using Manning's equation,

$$U = \frac{1}{n} D^{2/3} S^{1/2}$$

one obtains $$\sqrt{DS} = \frac{Un}{D^{1/6}}$$

and, therefore, $$u_* = (Un/D^{1\backslash 6})\sqrt{g}$$

Hence, $$\frac{w}{u_*} = \frac{wD^{1/6}}{Un\sqrt{g}}$$

Here, n is the Manning's roughness coefficient.

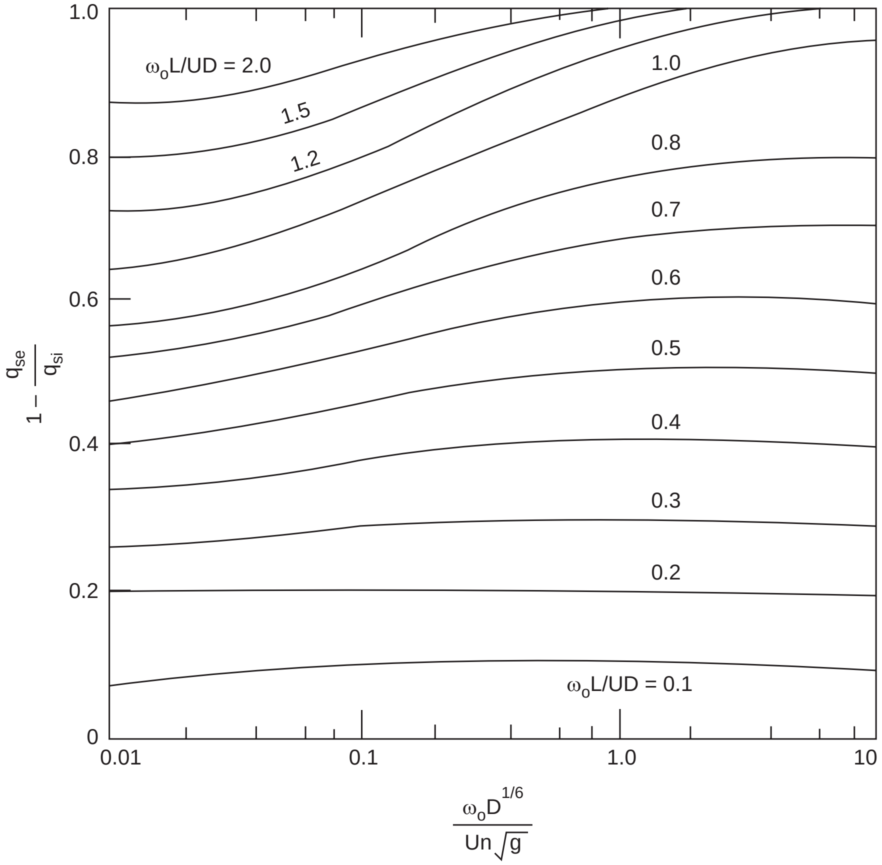


Fig. 13.18 Sediment removal efficiency for settling basin

Sumer (17) assumed logarithmic velocity distribution for flow in the settling basin and also considered suitable variation of the diffusion coefficient and thus analysed the settling of a sediment particle to finally obtain,

$$\eta = 1 - e^{-\left(\frac{k\lambda}{6}\right)\left(\frac{Lu_*}{UD}\right)} \tag{13.27}$$

Here, k = 0.4 and λ is a parameter that depends on β (= $\frac{w}{ku_*}$), Fig. 13.19.

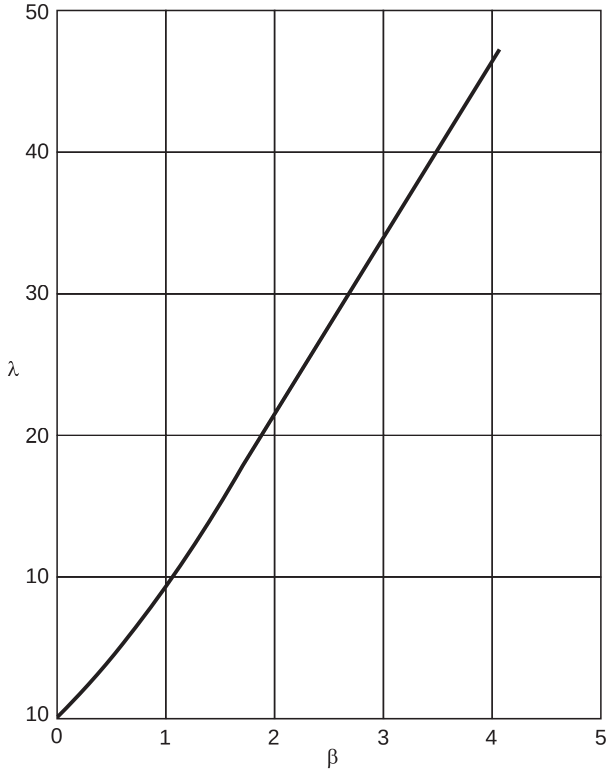


Fig. 13.19 Relation between λ and β

Based on dimensional analysis and experimental investigation, Garde *et al*. (18) obtained

$$\eta = \eta_o(1 - e^{-KL/D}) \tag{13.28}$$

Here, η_o and k are related to w/u_* as shown in Fig. 13.20. The nature of the relationships of Eqs. (13.27) and (13.28) is such that the efficiency reaches 100 per cent only asymptocally.

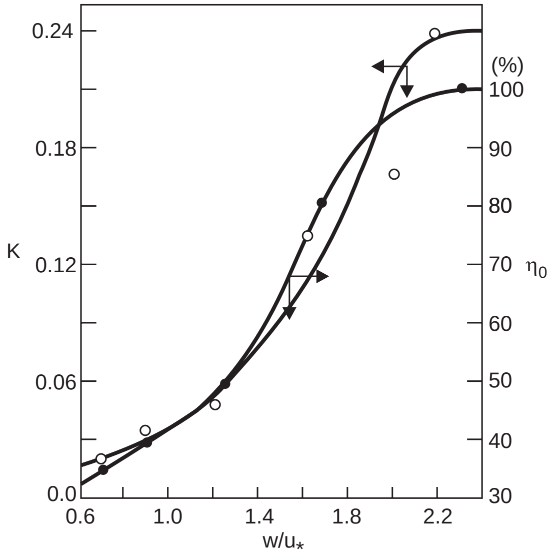


Fig. 13.20 Variation of K and η_0 with $w/u*$

These methods can be used suitably for estimating the efficiency of sediment removal, η for sediment size d in a given settling basin. However, for designing a suitable settling basin to achieve the desired efficiency of sediment removal, η for sediment size d and given flow rate Q, one needs to select suitable combinations of L, B_2, and D for the settling basin and obtain the values of η for each of these combinations. The one which gives η higher than the desired value of η can be selected. At times, these different methods would give very different results and one should use the results judiciously.

The performance of a settling basin is adversely affected if the flow conditions in the basin are relatively turbulent or there is some amount of 'short-circuiting' of flow on account of separation due to expansion in cross-section of flow. Therefore, a suitable expanding transition with two (or more) splitter plates at the entrance of the basin and a contracting transition at the outlet end of the basin are always provided.

Further, a suitable provision to remove the deposited material from the bed of the basin is to be made. For this, one can have two settling basins parallel to each other so that while the material from one basin is removed, the other is in operation. Alternatively, the bed of the basin may be divided into suitable number of hopper-shaped chambers and a suitable pipe outlet in each of these chambers is provided. The deposited sediment may be flushed away through these outlets.

Example 13.3 Estimate the efficiency of removal of sediment of size 0.05 mm (fall velocity $= 2 \times 10^{-3}$ m/s) by a settling basin (length = 40 m, width = 12 m and depth of flow = 3 m and Manning's roughness coefficient, $n = 0.014$) provided in a power channel carrying a discharge, Q of 1.41 m^3/s.

Solution:

$$U = \frac{Q}{DB_2} = \frac{1.41}{3 \times 12} = 0.039 \text{ m/s}$$

$$u_* = \frac{Un\sqrt{g}}{D^{1/6}} = \frac{0.039 \times 0.014 \times \sqrt{9.81}}{(3)^{1/6}} = 1.424 \times 10^{-3} \text{ m/s}$$

$$\frac{w}{u_*} = \frac{2 \times 10^{-3}}{1.424 \times 10^{-3}} = 1.4$$

$$\frac{wL}{UD} = \frac{2 \times 10^{-3} \times 40}{0.039 \times 3} = 0.684$$

Using Camp – Dobbins methods, Fig. 13.18,

$$\eta = 0.66 = 66\%$$

For Sumer's method, $\beta = \dfrac{w}{ku_*} = \dfrac{2 \times 10^{-3}}{0.4 \times 1.424 \times 10^{-3}} = 3.5$

From Fig. 13.19, $\lambda = 38$.

Therefore, $\left(\dfrac{k\lambda}{6}\right)\left(\dfrac{Lu_*}{UD}\right) = \left(\dfrac{0.4 \times 38}{6}\right)\left(\dfrac{40 \times 1.424 \times 10^{-3}}{0.039 \times 3}\right) = 1.23$

$\therefore$ $\eta = 1 - e^{-1.23} = 0.70 = 70\%$

Thus, Camp-Dobbin's method and Sumer's method give matching results.

For $\dfrac{w}{u_*}$ = 1.4, Fig. 13.20 gives

$$K = 0.08 \text{ and } \eta_o = 57\%$$

Therefore, Eq. (13.28) gives

$$\eta = \eta_o\,(1 - e^{-KL/D}) = 57\,(1 - e^{-0.08 \times 40/3}) = 37.38\%$$

Thus the method proposed by Garde *et al.* gives relatively smaller value of η for the given problem compared to the value of η obtained by either Camp-Dobbins method or Sumer's method.

13.12. RIVER TRAINING FOR CANAL HEADWORKS

River training structures for canal headworks are required for the following purposes (13):

(*i*) To prevent outflanking of the structure,

(*ii*) To minimise possible cross-flow through the barrage or weir which may endanger the structure and protection works.

(*iii*) To prevent flooding of the riverine lands upstream of the barrages and weirs, and

(*iv*) To provide favourable curvature of flow at the head regulator from the consideration of entry of sediment into the canal.

The following types of river training structures are usually provided for weirs (13):

(*i*) Guide banks,

(*ii*) Approach embankments,

(*iii*) Afflux embankments, and

(*iv*) Groynes or spurs.

The purpose of guide banks is to narrow down and restrict the course of a river so that the river flows centrally through the weir constructed across it without damaging the structure and its approaches. The alignment of guide banks should be determined such that the pattern of flow at the head regulator induces favourable curvature of flow minimising sediment entry into the canal system (19). More details of guide banks have been included in Chapter 12 and a typical sketch of river training structures provided at weirs and salient details of guide banks are given in Figs. 13.21 and 13.22. While constructing weirs on alluvial rivers, the natural waterway is restricted from economic and flow considerations, and the unbridged width of river is blocked by means of approach embankments. Depending upon the distance between the guide bank and the extent of the alluvial belt, the river may form either one or two meander loops (Fig. 13.21). The approach embankments, aligned with the weir axis, should extend up to a point beyond the range of the worst anticipated loop (19).

Afflux embankments are earthen embankments which extend from both the abutments (or the approach embankments) and are connected on the upstream to the ground above the affluxed highest flood level (or to flood embankments, if existing).

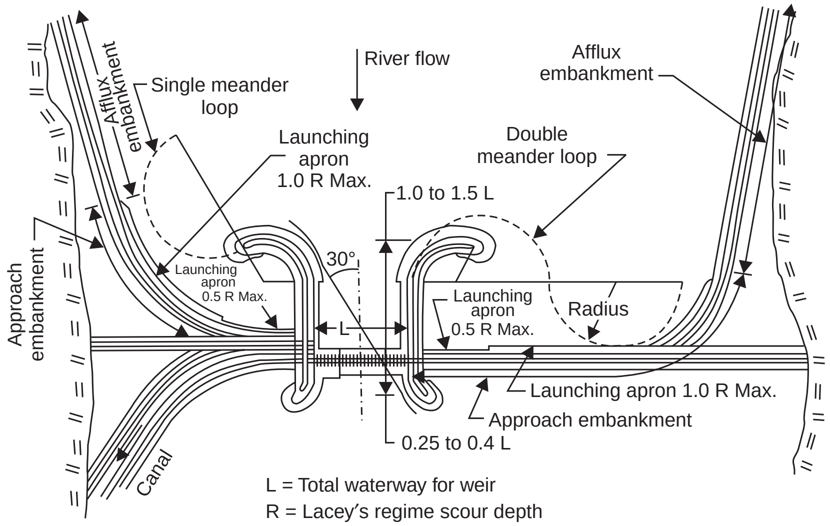


Fig. 13.21 Typical layout of river training structures for canal headworks

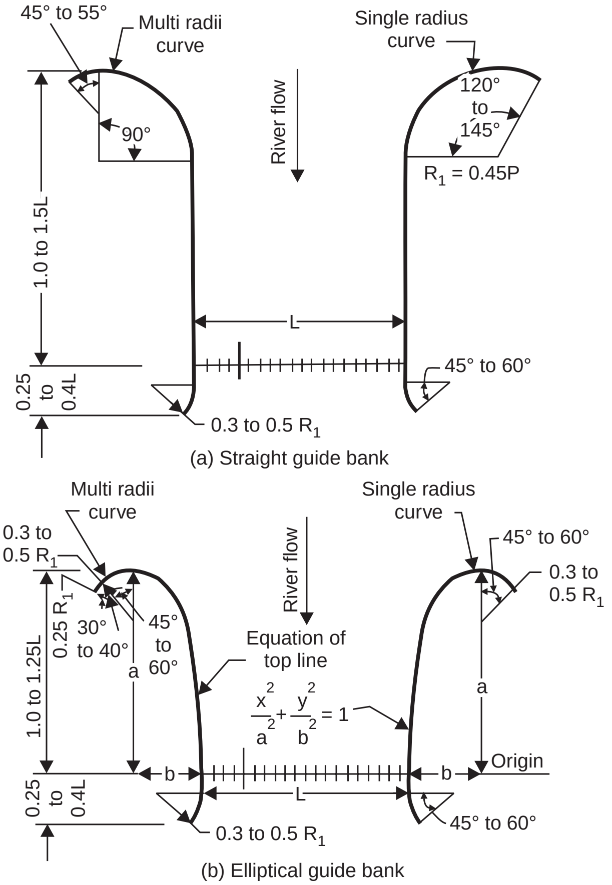


Fig. 13.22 Geometrical shape of guide banks

A top width of about 6 to 9 m and freeboard of 1.0 to 1.5 m above the highest flood level (for a 1-in-500 years flood) are usually provided for these river training works (19). Besides, stone pitching, launching aprons, *etc*. are also provided in the usual manner as described in Chapter 12.

EXERCISES

13.1 Differentiate between a weir and a barrage. Describe the functions of different parts of canal headworks.

13.2 A barrage is to be constructed across a river having a high flood discharge of 7000 m^3/s. The average bed level of the river is 290.00 m and the HFL before construction is 293.80 m. The canal taking off from the river has a full supply discharge of 200 m^3/s. Propose suitable values for

(*i*) Crest levels and waterway of undersluice and weir portions.

(*ii*) Bottom levels of the upstream and downstream sheet piles for weir portion from scour considerations. Also design the downstream launching apron for the undersluices.

Adopt Lacey's silt factor as unity and permissible afflux as 1.0 m.

13.3 A river carries a high flood discharge of 16000 m^3/s with its average bed level at 200.0 m. A canal carrying 200 m^3/s is to take off from the headworks. The full supply level of the canal at its head is 203.0 m. The high flood level before construction is 205.7 m and Lacey's silt factor is equal to unity. Fix suitable values for the waterway and crest levels of weir, undersluices and canal head regulator. Make provision for a silt excluder. Assume suitably any other data, if required.

13.4 Determine the heights of gates and hoisting deck levels for the undersluice and barrage portions and also for the canal head regulator of a diversion headworks. The relevant data are as follows:

River bed level	= 256.20 m
High flood level	= 260.60 m
Afflux	= 0.65 m
FSL of canal	= 258.40 m
Discharge intensity in the barrage portion at high flood	= 15.00 m^2/s
Discharge intensity in the undersluice portion at high flood	= 18.00 m^2/s

Provision of silt excluder is envisaged. A freeboard of 0.3 m may be assumed for gates.

13.5. A barrage is to be constructed across a river whose high flood discharge is 8000 m^3/s and whose minimum bed level is 290.0 m. A canal with full supply discharge of 200 m^3/s would take off from the right bank of the headworks at an angle of 90°. The bed level and the full supply level of the canal are 290.50 and 292.5 m respectively. The stage-discharge curve for the river at the barrage site is given by

$$\text{Stage (in metres)} = 290 + 0.75\, Q - 0.03\, Q^2$$

where, Q is the discharge in 1000 m^3/s. The median size of the river bed material is 0.60 mm.

Design the following:

(*i*) Weir and undersluice portions including floor thickness, sheet piles, launching apron, etc.,

(*ii*) Waterway and crest level of canal head regulator,

(*iii*) Guide banks, and

(*iv*) Silt excluder.

Prepare plan layout of headworks and also the longitudinal sections through weir and undersluice portions. Assume suitably any other data, if required.

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14

PLANNING OF WATER RESOURCE PROJECTS

14.1. GENERAL

The main aim of all water resource development is to improve the economic and environmental conditions for human living. A water resource project may serve one or more purposes and, accordingly, can be either single-purpose or multipurpose. In most cases, a project would be of a dual or multipurpose type. As such, the entire project needs to be investigated as a unit before the design requirement of a single component, such as a dam, can be finalised. A water resource project may serve one or more of the following purposes:

(*i*) Irrigation,

(*ii*) Power development,

(*iii*) Flood control,

(*iv*) Industrial water supply,

(*v*) Domestic and municipal water supply,

(*vi*) Recreation,

(*vii*) Fish and wild life preservation and promotion, and

(*viii*) Navigation.

In almost every water resource project, dam and reservoir are key components of the project. *Dams* impound water, divert water from a stream, or raise the water level. In exceptional cases, dams may be constructed to impound water-borne sediments and water having a damaging chemical quality. Dams contribute immensely in reducing poverty and impacts of floods and droughts besides rejuvenating rivers in dry season. Dams also enable recharge of ground water and growth of more biomass. A *reservoir* is a fresh-water body created or enlarged by the building of dams, barriers or excavations (1).

It is seldom that a water resource project consists of only a dam and reservoir facility. In a flood control project, levees and other channel control works, besides the dam and reservoir, are usually desirable. Water resource projects for power development and water supply (for irrigation, domestic, municipal, and industrial purposes) have a combination of project components to accomplish the desired objectives. Therefore, dams must be planned, designed, and constructed to operate efficiently and harmoniously with other components of the project to achieve maximum benefits at minimum cost. The economic, environmental and social feasibilities, and justification of dam must be examined in combination with those of other project components, and the total project must be evaluated and judged for its feasibility. If the evaluation of a project proposal does not show justification for its construction, it may be dropped or, alternatively, revised and updated with possible justification at a later time. A water resource project should be planned bearing in mind probable physical, economic, and environmental effects.

14.2. PHYSICAL FACTORS

Except for flood control projects, availability of sufficient water is essential for all types of water resource projects. In flood control projects, the sudden excess of water is the problem. The source of water is the surface runoff resulting from weather phenomena which are understood only in a general way. Weather conditions can be predicted only as seasonal probabilities. Weather predictions for shorter periods (a few hours or days) can, however, be made with more reliability. Historical measurements of stream flows and rainfall are considered the best available means for forecasting stream flow supplies for water resource projects.

At sites where no measurements or only a few measurements have been made, reliable correlation methods are used to estimate streamflow statistics. There is always some risk involved in building a project either too large or too small at sites of meagre stream flow measurements. In such situations, alternatives of staged development or other means of adjusting the project size and scope may have to be considered.

A flood occurring once in 100 years or less may cause enormous damage. Therefore, stream gauging records of 10, 20 or 30 years, though useful to some extent, are inadequate for flood control projects and spillway design for large dams. Besides, actual measurement of peak flood flows is difficult even if the stream is being gauged. Some other methods of estimating the magnitude of peak floods are invariably used for the planning of such works. Computation of the stream flow based on high water marks and flood channel dimensions is one such method. Alternatively, stream flow (or runoff) estimation can be based on actual measurements of amount and duration of high rainfall at rain gauge stations in the catchment area upstream of the dam site. The latter method considers factors such as principles of precipitation as affected by stream characteristics in the region, and the catchment characteristics (location, shape, vegetative cover, and geological structure). Extremely large floods are also extremely infrequent floods. Hence, the planner's judgement is crucial in deciding the size of the flood to be controlled by the project.

Two main factors which determine the site of a water resource project are the areas needing water and the location where water supply is available for development. For economic reasons, the water source must be near the place of use so as to save on cost of conveyance. Also, the source should be at higher elevation than the service area to avoid pumping. In case of projects where water is stored only for the purpose of flood control, there is no conveyance cost involved.

One can build a dam almost anywhere if one spends enough money. But, there is obvious advantage in having a dam site in a narrow section of a stream channel where sufficiently strong and impervious foundation (rock or consolidated material) is available. The abutments must be of sufficient height and be strong and impervious. Further, the dam site should not be located on or very close to an active earthquake fault. The dam site must have suitable site for spillway (a structure which releases surplus water after the reservoir has been filled up to its maximum capacity) which can be made part of the main dam only in case of a concrete dam. A dam requires a very large quantity of construction material (cement, aggregates, impervious and pervious soils, rocks, *etc.*) which should be available within economical hauling distance of the dam site. An easily accessible site is preferred as it involves least expenditure on communication works required for the transport of construction machinery, power house equipment construction material, and so on, to the dam site. The value of the land and property which would be submerged by the proposed reservoir should be less than the expected benefits from the project.

The area upstream of the dam site would constitute the reservoir component of the project. For economy in dam height, a reservoir site should be wide and on a mildly sloping stream in order to have a long and wide reservoir in proportion to the height of the dam. The reservoir must not be sited on excessively leaky formations. The site with the possibility of landslides, rock-slides or rockfalls into the reservoir area (which reduce the storage capacity of the reservoir) must be avoided. The site should not be, as far as possible, on valuable land being used for some other purposes, such as agriculture, forestry, communication, and habitation by people. Sites with mineral deposits in and around the reservoir area should also be avoided. As far as possible, a reservoir should not be provided on a stream carrying large sediment loads which would eventually get deposited in the reservoir, thereby reducing its useful storage capacity. However, all streams carry some amount of sediment. Hence, part of the total reservoir storage is reserved for the accumulation of sediment which is likely to enter the reservoir during its intended economic life. Possibilities of constructing sedimentation basins a short distance upstream of the reservoir and/or providing catchment protection and management against sediment erosion must also be explored.

14.3. ECONOMIC CONSIDERATIONS

The cost of a water resource project includes capital investment for constructing the project facilities and the annual or recurring expenditure for operation and maintenance (including replacement) of the project. The capital cost includes the costs of planning, investigations, designs, and construction besides the cost of acquiring rights to the use of water, litigations, and rehabilitation of the affected people. The capital cost also includes the interest on the money invested during construction and up to the start of the project. The benefits likely to be received from a water resource project are widely distributed. As such, the investments on the project cannot be compared with the benefits in terms of monetary units. However, the benefits are expressed, as far as possible, in terms of monetary units and the investment and operational costs are thus compared with the benefits.

It is difficult to quantify some types of project benefits. For example, in an irrigation project, the benefits extend beyond the farmer through a chain of related activities to the people of the area. Social benefits (such as protection against loss of life by floods), recreational benefits, *etc*. are also difficult to estimate in monetary terms. However, benefits of municipal and industrial water services and hydroelectric power generation can be easily estimated by working out the cost of producing the same results by another reasonable alternative arrangement or by determining the market value of the product. Benefits from a flood control project can be estimated by working out the reduction in flood damages in agricultural, residential, commercial, industrial, and such other activities. The value of the land protected from floods increases and this fact should also be included in the benefits of a flood control project. Other possible benefits from a water resource project may be in the from of a fishery enhancement, water quality improvement (in downstream flows from storage releases during dry seasons), and navigation improvement on large rivers (due to storage releases during low flow seasons). Construction of a water resource project provides employment to people of the locality and is vital in areas of persistent unemployment.

Because of uncertainties involved in the estimation of project benefits, the computed benefit-cost ratio is generally not considered as the sole criterion for determining the economic viability of a project. Nevertheless, such computations do provide a logical basis for arriving at meaningful decisions on the size of the project, inclusion and exclusion of different project functions, the priority of the project, and so on. Other considerations such as social needs, repayment potential, and environmental aspects are also examined in determining the worth of a proposal for water resource development.

14.4. ENVIRONMENTAL EFFECTS

A well-planned water resource project should be desirable from economic, social as well as environmental considerations. It should, however, be noted that some of the project components, notably dams and reservoirs, cause adverse environmental effects in the regions of their direct influence. While trying to achieve major project objectives of a water resource project, the planner must examine alternative plans of dams and reservoirs to minimise adverse environmental effects.

Environment is best defined as all external conditions which affect the existence of all living beings. Different living beings affect one another, and the environmental requirements of different living beings are interrelated.

Besides, it is generally not possible to evaluate environmental effects in economic terms. In case of pollution of water and air, however, it is possible to estimate economic loss to some degree. In addition, it is difficult to assign a degree of importance to various environment conditions likely to be judged differently by different persons depending upon their own viewpoint. For example, the people of a hilly region will have a different viewpoint regarding the siting of a dam from those living in the plains where land is inundated during floods and wells go dry during drought.

The environmental effects which directly affect the livelihood and well-being of people are of prime concern. Other environmental effects on various other living beings are also of concern to man but only to the extent to which the existence of the living beings is important to man's living conditions. The beneficial environmental effects include land use improvements by irrigation, flood protection, improved water supplies for domestic and municipal uses, power supplies without consumption of fuel, water quality improvement, fishery improvement, recreational improvement, and health improvement. Various adverse environmental effects may be caused due to construction and operation of dams and reservoirs. Some of these can be mitigated by taking suitable steps while others are unavoidable. These have been tabulated in Table 14.1. The environmental check-list (Table 1.14) provides a comprehensive guide to the areas of environmental concern which should be considered in the planning, design, operation, and management of water resource projects.

Table 14.1 Adverse effects of dams and reservoirs on environment (1)

Potential adverse effect	*Mitigation method or effect*	*Probable degree or importance of adverse effect*
Land use for reservoir		
Loss of fish and aquatic habitat	Changes of species	New species may be less desirable than original species
Loss of wildlife habitat	Improve other areas for habitat	Full mitigation probably not possible
Loss of future access to mineral deposits	None	Is of importance only if mineral deposits exist

(*Contd.*)

Loss of mountain valley areas	None	Important only in extremely mountainous areas
Inundation of historical or archaeological sites	Possibly by a museum	Varies with each individual site
Inundation of exceptional geological formations	Usually not possible	Varies with each individual site
Alteration of downstream flows		
Reduction of fish and aquatic habitat	Maintain regulated flows	Full mitigation possible, but frequently not acceptable because of large sacrifice of project accomplishments
Reduction of stream flushing flows	Release occasional flushing flows	Mitigation method not proven to be worthwhile; Degree of environmental effect depends upon specific stream situation
Changes of water quality	Selective level reservoir outlets; water aeration, if needed	Somewhat limited experience with selective level outlets indicates good prospects of full mitigation
Interference with fish and wildlife migrations		
Blocking anadromous fish runs	Fish hatcheries	Usually capable of full mitigation
Blocking animal migration routes	None practical	Importance depends upon the specific site
Landscape appearance		
Excavation and waste disposal sites	Project expenditures required to landscape sites	Satisfactory mitigation usually possible without excessive expenditure
Reservoir banks below maximum waterline	Minor areas may be developed for beaches	Degree of impact depends upon the specific reservoir site
Abandoned construction facilities	Construction clean-up	Full mitigation possible; important only if not done
Erosion scars from construction roads	Principally by care of drainage	Adverse effects can be reduced but not entirely eliminated with in reasonable cost
Reservoir clearing waste disposal	Controlled burning; marketing maximum amounts of wood products	Temporary effect, usually minor, but not entirely avoidable

14.5. SELECTION OF A PROJECT PLAN

Planning may be defined as the systematic consideration of a project from the original statement of purpose through the evaluation of alternatives to the final decision on a course of action. Planning of water resource project begins with some definite idea about its main purpose. It is usually economical to have a multipurpose rather than a single-purpose project. From economic considerations, the best project plan is the one for which the ratio of combined project benefits and the total project cost is maximum. The time required to construct a dam and then to first fill the reservoir before the start of the project operation is usually very large (several years) and, hence, the interest on the investment up to the start of the project operation should also be added to the investment costs. The cost of a dam and other major project features and also the benefits for at least three different sizes (the smallest, the largest and an intermediate) of the project are worked out. Using these computations, size-benefit and size-cost curves for different possible functions are prepared. A proper analysis of all this information would yield the size and functions of the project which would result in maximum benefit-cost ratio.

Generally, the needs for water services, power, and flood control in any given region continue to grow due to the increasing population. Therefore, it appears to be uneconomical to build large and costly projects far in advance of their needs. As such, physical and design conditions permitting, a project can also be constructed in stages. Because of the growing concern for environmental conditions, it is essential to take into account the environmental effects of alternative plans. Usually, there is an improvement in the environmental conditions due to the availability of water service and flood protection facilities. However, there are some adverse environmental effects of water resource projects which affect (*i*) scenic beauty, and (*ii*) wildlife (both land and aquatic species). These effects, however, cannot be measured. A planner can, therefore, only select an alternative with more favourable or less unfavourable effects.

In making a choice of suitable alternatives, some kind of compromise is always made. These compromises may be in the form of fixing stream flows, acquisition of land to be used as wildlife tracts, siting project features to the advantage of scenic views, and providing access to areas having enjoyment potential. The following method (1) is suggested for this purpose.

Alternative plans of the proposed water resource project, having different amounts of environmental impact but accomplishing other objectives of the project, are prepared. The first step would be to make an inventory of the existing conditions of various important environmental qualities of the water resource system under consideration. These environmental factors may be ranked in order of their importance. The second step in the preparation of alternative plans would be to estimate the future environmental conditions without the project development. These conditions may be the same as the existing ones, or may be degraded or improved. The third step would be to prepare an "optimum economic water resource project plan alternative" without considering environmental impacts except those which are positively controlling environmental impacts. Similarly, an "optimum environmental water resource project plan alternative" would be prepared wherein an attempt would be made to minimise all adverse environmental impacts and still achieve some of the project objectives. If the second alternative results in significantly reduced accomplishments of the project objectives or greatly increased cost compared to that of the first alternative, the second alternative is discarded in favour of a third alternative plan. The third alternative plan would be so prepared that it would reflect a compromise between the two extreme alternatives and seek to avoid or minimise the important adverse environmental impacts while accomplishing all or most of the project objectives of the first alternative.

The role of the planner of a water resource project is to select the best of all possible alternatives. Various methods of optimization, collectively called systems analysis, are, therefore, obvious tools for this purpose. Because of large number of constraints involved, one has to often make several simplifying assumptions in order to obtain the best possible alternative. Besides, deficiencies of the input data will make determination of the true optimum a difficult task. Nevertheless, systems analysis is still the best method of determining the best possible alternative out of several feasible alternatives.

14.6. INVESTIGATIONS

The basic data, usually required for planning of dams and reservoirs, can be grouped in the following categories (1):

(*i*) *Hydrologic data:* Stream flows, flood flows, evaporation, sedimentation, water quality, water rights, and tail-water curves.

(*ii*) *Geological data:* Reservoir sites, dam sites, and construction materials.

(*iii*) *Topographic surveys:* Catchments, reservoir sites, dam sites, and borrow areas.

(*iv*) *Legal data:* Water rights.

(*v*) *Reservoir site cost data:* Land acquisition, clearing, and relocations.

(*vi*) *Environmental factors:* Fish and wildlife, recreation, scenic, historical, and archaeological.

(*vii*) *Economic data:* Economic base for area benefited, crop data, land classification, and market data for various purposes.

The desirable quality of these data would depend on the level of investigations. Investigations for a water resource project are generally carried out in three separate steps (or levels or stages): reconnaissance (or preliminary), feasibility, and pre-construction.

14.6.1. Reconnaissance (or Preliminary) Investigations

The main purpose of such investigations is to screen out the poorer alternatives and to decide the types and amounts of more expensive and time-consuming data (such as stream flow records, topographic mapping, and so on) which need to be collected for making feasibility investigations of the remaining selectable alternatives. A reconnaissance survey will identify the scope of a project plan with respect to its geographical location, project functions, approximate size of its various components, likely problem areas, and time and cost of conducting feasibility investigations.

A complete reconnaissance investigation is, in fact, a preliminary version of a feasibility investigation carried out in a rather short time with less accuracy. It considers all the physical, engineering, economic, environmental, and social aspects related to the project. It is usually conducted with the available data. Collection of some new data, if considered necessary for reconnaissance, is made by preliminary surveys. These may include a simple cross-section (instead of detailed topography) of a stream at dam site, surface investigations of geological conditions at dam site, subsurface explorations for foundation quality at dam site, quality and quantity of available construction materials, and so forth. Preliminary designs are made by using short-cut mehods (using curves, tables, and previous experiences). Cost and benefits of the project are also estimated.

Based on the results of preliminary investigations of alternative project plans, a selection of feasible project plans is made for subsequent feasibility investigations.

14.6.2. Feasibility Investigation

The aim of the feasibility investigation is to ascertain the soundness and justification, or lack of these, of different alternative plans chosen after carrying out preliminary investigations. The analyses need to be of high accuracy and dependability so that the reliability of results, on the basis of which the final selection of the project plan is made, may not be questioned. It should, however, be noted that the feasibility investigation does not mean the end of the project planning. Some minor changes are always required to be made for various reasons during final designs before construction, during construction, and even during project operation.

The first step in the feasibility investigation is to collect or update the basic data of different types. The accuracy and reliability levels of these data must be consistent with the degree of accuracy required for feasibility justifications. The basic data for dams and reservoirs include topographic surveys of sites, information on stream flow and design flood, land costs, reservoir clearing costs, communication facilities, climatic conditions affecting construction, fishery and wildlife to be preserved, construction material, foundation conditions of dam site and reservoir area, availability of trained manpower, and important environmental and other considerations. The facilities and appurtenances necessary for the functioning of the project must be specified, and considered while making feasibility cost estimates.

On the basis of the feasibility investigation, a provisional selection of the site and size of the project is made. Besides, the functions of the project are also decided. The final report prepared on the basis of the feasibility investigations is submitted to the approving and funding authorities of the project.

14.6.3. Pre-construction Investigations

The final adoption of provisionally selected project site and its size and functions begins after the project has been approved and funded for construction. It is essential that final designs consider any new information which might have been obtained or received during the time interval between feasibility investigation and the final design. For example, an extreme low runoff season or a flood of large magnitude might have occurred during this intervening period and this may necessitate changes in the estimates of the critical dry year project water supplies or of the frequency of occurrence of a flood of given magnitude.

For pre-construction planning, more detailed and accurate topographic maps and additional geological investigations are usually necessary to reduce uncertainties about foundation conditions and construction material. This is also true of other basic data needed for planning of dams and reservoirs.

14.7. CHOICE OF DAMS

Most of the dams can be grouped into one of the following two categories:

(*i*) Embankment dams, and

(*ii*) Concrete dams.

Embankment dams include earth-fill dams and rock-fill dams. Concrete dams include gravity dams, arch dams and buttress dams. Preliminary designs and estimates will usually be required for different types of dams before one can decide the suitability or otherwise of one type of dam in comparison to other types. The cost of construction is the most important factor to be considered while making the final selection of the type of dam. Besides, the characteristics of each type of dam, as related to the physical features of the site and its adaptation to the purposes of the dam, as well as safety, and other relevant limitations are also to be considered for selecting the best type of dam for a particular site. The following are the important factors which affect the choice of the type of dam:

(*i*) Topography,

(*ii*) Geology and foundation conditions,

(*iii*) Material available, and

(*iv*) Size and location of spillway.

Topography of the site dictates the first choice of the type of dam. A concrete dam would be the obvious choice for narrow stream flowing between high and rocky abutments (*i.e.,* deep gorges). Broad valleys in plains would suggest an embankment dam with a separate spillway.

Geological and hydrogeological characteristics of the strata which are to carry the weight of the dam determine the foundation conditions. Any type of dam can be constructed on solid rock foundations. Well-compacted gravel foundations are suitable for concrete gravity dams of small height, earth-fill, and rock-fill dams. However, effective cutoffs are required to check the foundation seepage. Silt or fine sand foundations can support concrete dams of small height and earth-fill dams. Problems of settlement, piping, and the foundation seepage are associated with this type of foundation. Non-uniform foundations containing different types of strata will usually require special treatment before any type of dam is constructed on such foundations.

If the construction materials to be used in large quantity for the construction of the dam are available in sufficient quantity within a reasonable distance from the site, the cost of the dam will be considerably reduced due to saving on transportation. If suitable soils for the construction of an earth-fill dam are locally available in nearby borrowpits, choice of an earth-fill dam would be the most economical. The availability of sand and gravel (for concrete) near the dam site would reduce the cost of a concrete dam.

Spillway is a major part of any dam and its size, type, and the natural restrictions in its location will affect the selection of the type of dam. Spillway requirements are decided by the runoff and streamflow characteristics. As such, spillway on dams across streams of large flood potential can become the dominant part of the dam and put the selection of the type of dam to a secondary position. For large spillways, it may be desirable to combine the spillway and dam into one structure. This is possible only in concrete dams. Embankment dams are based on more conservative design assumptions and, hence, spillway is generally not constructed as part of the embankment. On the other hand, excavated material from a separate spillway can be advantageously used for the construction of an embankment dam.

14.8. PLANNING OF RESERVOIRS

One major consideration in the development of any surface water resource project is the structural stability of the reservoir which should be capable of containing safely the projected volumes of water for use throughout its life time. The main factors to be considered for this are as follows (1):

(*i*) Rim stability,

(*ii*) Water-holding capability,

(*iii*) Loss of reservoir water,

(*iv*) Bank storage,

(*v*) Seismicity, and

(*vi*) Sedimentation.

Rim stability and water-holding capability are interrelated. Rim failure can be caused due to either the sliding or the erosion of a segment of the reservoir rim. Seepage of water is mainly responsible for such failures. Major slides into a reservoir would, obviously, reduce reservoir capacity considerably. Similarly, snow avalanches and masses of ice falling from hanging glaciers can cause serious problems. Besides reducing the capacity of the reservoir, a rapidly moving slide may also generate waves. A dam may be overtopped due to the resulting wave action or rise of the water surface on account of a major slide into the reservoir. If the reservoir site is likely to be affected by the slides and cannot be abandoned, some restraining steps in reservoir operation should be taken to avoid serious failure. These steps could be in the form of limiting the filling and drawdown rates or imposing the maximum allowable water surface at a level lower than the maximum normal water surface. Alternatively, installation of drains to relieve water pressure along likely slip surfaces, some form of impervious lining, and pinning the unstable mass of its parent formation by rock bolting can be resorted to for preventing slides. Stabilisation of the unstable mass can also be achieved by strengthening or replacing weak material. Grouting is the most common remedy for strengthening such weak masses. It may be desirable to plan the steps to be taken to mitigate the effects of potential slide after it has occurred in spite of all preventive steps.

Reservoir water loss either to the atmosphere or to the ground can be a controlling factor in the selection of a site for a conservation reservoir. For a flood control reservoir, water loss is of concern only if it relates to the safety of the project. The lining of the surface through which seepage is expected is one of the preventive measures to reduce the reservoir water loss to the ground. At times, a blanket of impervious material extending from the heel of the dam is required. This too serves to control the seepage from the reservoir.

Loss of reservoir water to the atmosphere occurs due to direct evaporation from the reservoir surface. The evaporation losses are affected by the climate of the region, shape of the reservoir, wind conditions, humidity, and temperature. From considerations of evaporation, a reservoir site having a small surface area to volume ratio will be better than a saucer-shaped reservoir of equal capacity. Evaporation-retardant chemicals increase the surface tension of water by forming a monomolecular film and thus reduce evaporation.

Bank storage is the water which spreads out from a body of water, filling interstices of the surrounding earth and rock mass. This water is assumed to remain in the surrounding mass and does not continue to move to ultimately join the ground water or surface water as seepage water does. The bank storage is not mitigable. It must, however, be estimated for feasibility investigations and measured during reservoir operation for providing guidelines for reservoir regulation.

It appears that there is some effect of reservoir impoundment on the increased seismic activity of an area in which a large reservoir (having a storage capacity of more than 12×10^8 m^3 behind a dam higher than 90 m) has been constructed (1). However, there have been large reservoirs without increasing the seismic activity of the region. The increased seismic activity is attributed to the changes in the normal effective stresses in the underlying rock because of the increased pore pressure. The transmission of the hydrostatic pressure through discontinuities in the underlying rock can have a triggering effect where a critical state of stress already exists. The relationship between the reservoir impoundment and the earthquake relationship is not fully understood. Hence, it is necessary that every large reservoir site be subjected to detailed geologic, geodetic, and seismic studies for feasibility decision. These observations must be continued during the reservoir operations too in order to better understand the relationship between reservoir impoundment and seismic activity.

The streams bringing water to the reservoir bring sediments too. The sediment gets deposited in the reservoir due to the reduced stream velocity. The capacity of the reservoir is reduced on account of sediment deposition in the reservoir. Usually, a portion of the reservoir storage is reserved for the storage of the sediment. The life of a reservoir is predicted on the basis of the amount of sediment delivered to it, the reservoir size, and its ability to retain the sediment. Sediment deposition at the initial stage may be beneficial in the sense that it may have the effect of a natural blanket resulting in reduced seepage loss. Measures to minimise sediment depostion in reservoirs include catchment protection through a vegetative management programme to prevent soil erosion, silt detention basins at inlets of smaller reservoirs, and low level outlets in dams to provide flushing action for removal of sediment from the reservoir. Of the various measures, the catchment protection is the most effective and also the costliest.

EXERCISES

14.1 Discuss the physical, economic, and environmental considerations which influence the planning of a water resource project.

14.2 What are the main factors which affect the selection of a dam?

14.3 Write a note on the planning of reservoirs.

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15

EMBANKMENT DAMS

15.1. GENERAL

Embankment dams are water impounding structures composed of natural fragmental materials (such as soil and rock) and consist of discrete particles which maintain their individual identities and have spaces between them. These materials derive strength from their position, internal friction, and mutual attraction of their particles. Unlike cemented materials, these fragmental materials form a relatively flexible structure which can deform slightly to conform to the foundation deflection without causing failure.

Embankment dams have been in existence for many centuries. The earliest forms of these dams were made naturally by landslides and rockfalls which cut off streams and formed natural dams. A 300-m natural dam of this type was created by a landslide which occurred in 1840 on the upper reaches of the Indus river (1). This dam, however, burst just after six months of its formation resulting in great loss of life and property in the valley.

Man-made tanks (or reservoirs) constructed in the early days of civilisation are found in the southern part of India and Sri Lanka. These tanks have been constructed by building earthen embankments. One such earthen embankment 17.6 km long, 21.34 m high, and containing about 13 million cubic metres of earth material was completed in 504 BC (2).

Till around 1925, the methods of design of an embankment dam were based on thumb rules and the heights of such dams rarely exceeded 30 m. The recent developments in soil mechanics have, however, made it possible to design an embankment dam with more confidence. This has resulted in much higher embankment dams such as Beas (116 m) and Ramganga (125 m) dams of India, Goschenenalp dam (156 m) in Switzerland, Oroville dam (224 m) in the USA, Mica Greek dam (235 m) in Canada, and Nurek dam (300 m) in the erstwhile USSR.

Conditions favouring the selection of an embankment dam are as follows (3):

(*i*) Significant thickness of soil deposits overlying bedrock,

(*ii*) Weak or soft bedrock which would not be able to resist high stresses from a concrete dam,

(*iii*) Abutments of either deep soil deposits or weak rock,

(*iv*) Availability of a suitable location for a spillway, and

(*v*) Availability of sufficient and suitable soils from required excavation or nearby borrow areas.

Embankment dams are mainly of two types:

(*i*) Earth-fill or earth dams, and

(*ii*) Rock-fill or earth-rock dams.

The bulk of the mass in an earth-fill dam consists of soil, while in the rock-fill dam it consists of rock material. The design principles for the two types of embankment dams are similar. Earth dams are further divided into the following types:

(*i*) Homogeneous earth dam, and

(*ii*) Zoned earth dam.

Homogeneous earth dams are constructed entirely or almost entirely of one type of earth material. A zoned earth dam, however, contains materials of different kinds in different parts of the embankment. A homogeneous earth dam is usually built when only one type of material is economically available and/or the height of the dam is not very large. A homogeneous earth dam of height exceeding about 6 to 8 m should always have some type of drain (Fig. 15.1) constructed of material more pervious than the embankment soil (4). Such drains reduce pore pressures in the downstream portion of the dam and thus increase the stability of the downstream slope. Besides, the drains control the outgoing seepage water in such a manner that it does not carry away embankment soil, *i.e.*, "piping" does not develop. Such a dam is also categorised as homogeneous (sometimes 'modified homogeneous') dam (Fig. 15.1). Some of the benefits of a zoned earth dam can be achieved in a homogeneous earth dam by either selective placement of soil or using different construction methods in different parts of the embankment and thus creating zones of different characteristics.

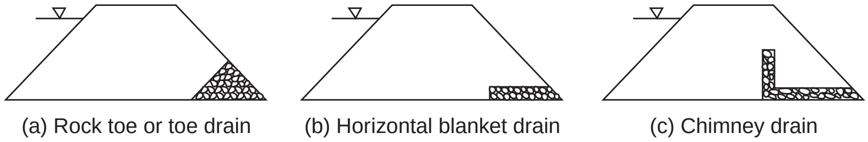


Fig. 15.1 Typical sections of homogeneous earth dam

The most common type of an earth dam usually adopted is the zoned earth dam as it leads to an economic and more stable design of the dam. In a zoned earth dam (Fig. 15.2), there is a central impervious core which is flanked by zones of more pervious material. The pervious zones, also known as shells, enclose, support, and protect the impervious core. The upstream shell provides stability against rapid drawdowns of reservoir while the downstream shell acts as a drain to control the line of seepage and provides stability to the dam during its construction

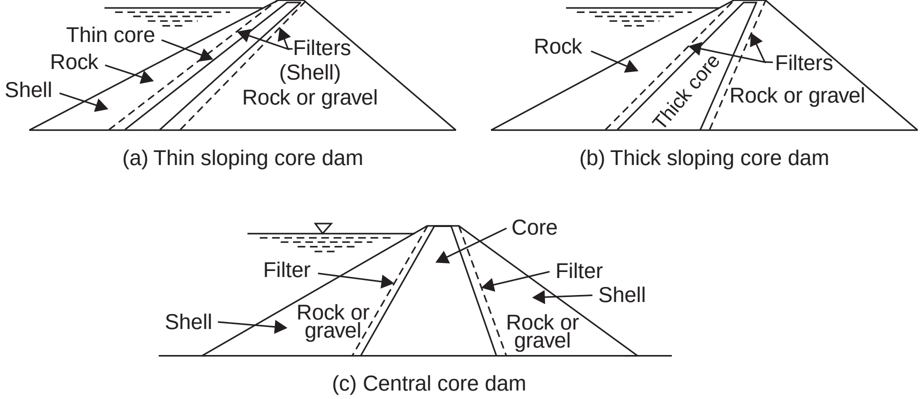


Fig. 15.2 Typical sections of zoned earth dam

and operation. The central core provides imperviousness to the embankment and reduces the seepage. The maximum width of the impervious core will be governed by stability and seepage criteria and also by the availability of the material. An earth dam with a sufficiently thick impervious core of strong material with pervious outer shells can have relatively steeper embankment slopes limited only by the foundation and embankment characteristics. However, a thin core dam is usually more economical and more easily constructed because of lesser amount of fine-grained soil to be handled. Core widths of 30 to 50% of the water head are usually adequate for any type of soil and any dam height while core widths of 15 to 20% of water head are thin and considered satisfactory, if adequately designed and constructed filter layers are provided (4). Core widths of less than 10% of water head should not be used as far as possible.

The impervious core can be placed either as a vertical core or as an upstream sloping core, each of which has some advantages over the other. A vertical core results in higher pressure on the contact between the core and foundation which, in turn, reduces the possibility of leakage along the contact. Besides, for a given quantity of impervious material, the vertical core will have greater thickness. The main advantage of upstream sloping core is that the main downstream shell can be constructed first and the core placed later – an advantageous feature in areas which have short periods of dry weather suitable for building a core of fine-grained soil. Besides, foundation grouting can be carried out while the downstream embankment is being constructed.

A rock-fill dam (Fig. 15.3) is an embankment which uses large-sized rock pieces to provide stability and an impervious membrane to provide watertightness (5). Materials used for the membrane are earth, concrete, steel, asphalt, and wood. The impervious membrane can be placed either on the upstream face of the dam or as a core inside the embankment. The upstream face of the dam is, however, more suitable for placing the impervious membrane due to the following reasons (4):

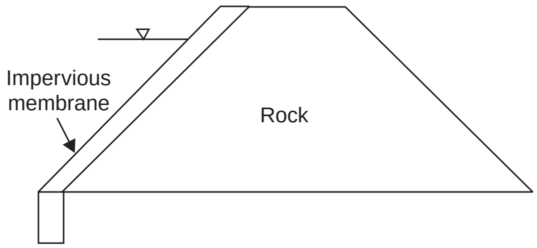


Fig. 15.3 Typical section of a rock-fill dam

(*i*) The upstream impervious membrane, with a suitable drain behind it, prevents seepage from entering the embankment. This reduces pore pressures and prevents the embankment mass from being submerged. Both these effects result in greater stability of the embankment.

(*ii*) The upstream impervious membrane is accessible for inspection and repair.

(*iii*) The upstream impervious membrane also serves a secondary function of wave protection.

(*iv*) The upstream impervious membrane can be built after completion of the embankment. This would permit initial settlement of the embankment without affecting the membrane adversely.

15.2. DESIGN CONSIDERATIONS

The design of an embankment dam is based on analytical considerations as well as on experience. The main steps in the design of an embankment dam are as follows (4):

(*i*) A thorough exploration of the foundation and abutments.

(*ii*) Evaluation of the quantities and characteristics of all the embankment construction materials available within a reasonable distance of the dam site.

(*iii*) A study of all the factors which may influence the design.

(*iv*) The selection of trial designs.

(*v*) Analysis of the safety of the trial designs.

(*vi*) The modification of the designs to satisfy the minimum stability requirements.

(*vii*) The preparation of detailed cost estimates.

(*viii*) The final selection of the design which seems to offer the best combination of economy, safety, and convenience in construction.

The steps (*iii*) and (*v*) have been discussed in the following sections.

15.2.1. Factors Influencing the Design of an Embankment Dam

(i) Materials Available for Construction

One of the main advantages of an embankment dam is the availability of construction material free of charge at or near the dam site. Depending upon the type of material available, the designed embankment may either be a homogeneous earth dam (when the soil available is impervious), a zoned earth dam (when both pervious and impervious soils are available) or a rock-fill dam (if rock is available and impervious material is not). The design may also incorporate use of materials from required excavation (for spillway construction) for reasons of economy.

(ii) Foundation Characteristics

An embankment dam can be constructed on almost any kind of foundation. Foundation characteristics mainly affect the foundation treatment which, in some cases, may be the most difficult and important part of the design and construction of an embankment dam. Besides, the embankment dimensions would be considerably influenced. For example, a softer foundation would necessitate an embankment with flatter slopes, broader cross-section, a larger freeboard (to mitigate the effects of embankment settlement), considerations for differential settlement cracks, and measures for control of underseepage to avoid the danger of piping.

(iii) Climate

It is generally difficult to handle fine-grained soils during the rainy season and control the construction moisture content of the fine-grained soils in arid regions. As such, if the construction of the embankment has to be carried out during the rainy season, it is advisable to have sloping core embankment. Similarly, in arid regions, one extra year may be required for constructing a small reservoir for storing flood runoff for the purpose of construction of the dam.

(iv) Shape and Size of Valleys

A dam site with broad valleys and gently sloping abutments may not affect the design of an embankment. However, narrow valleys and steep abutments may necessitate special design provisions. For example, because of the limited working space in a narrow valley, a simpler design requiring few special construction provisions is preferable. If the construction and maintenance of haul roads on the abutments at different elevations become difficult and costly, one may have to design a rock-fill embankment which can be constructed by dumping rock in high lifts from relatively few haul roads.

(v) River Diversion

If a river diversion scheme is to be implemented by the construction engineer or the contractor, it increases the problem of the designer who must envisage all possible ways of river diversion and make his design adaptable to each of these ways. On major rivers, however, it may be advisable to specify the river diversion scheme and design the embankment accordingly. In a narrow valley, the river is diverted through a tunnel or conduit. In wider valleys, parts of the embankment on the two abutments are constructed while letting the river flow through the central region of the valley. This central part of the embankment is constructed only at the end and is known as the 'closure' section. The construction of the closure section is carried out rapidly to prevent overtopping of the dam and, hence, special design details (*viz*., providing extra filter drains, different designs for different embankment sections in order to use the material available on the two abutments, and so on) and construction details (such as compacting the closure section at higher water content, keeping a reserve of borrow material to achieve a rapid construction rate for closure, *etc*.) are specified. If coffer dams of large volumes are used for diversion purposes, it would be economical to incorporate these into the dam embankment, if possible.

(vi) Probable Wave Action

The severity of the wave action and the amount of protection needed for the upstream face of the embankment mainly depends on the wind velocity and the length of the reservoir. The waves drive repeatedly against the embankment and, thus, cause the embankment erosion. A layer of dumped rock riprap is considered the most effective and economical wave protection.

(vii) Time Available for Construction

The design of an embankment is dependent on the time available for construction. A shorter construction period, in case of high dams, may result in higher pore pressures requiring relatively flatter slopes. When construction time is limited, it may not be possible to use the material from the required excavation, or it may be that only a part of it can be used. Similarly, underseepage measures would also be affected by the time available for construction. Handling of fine-grained soils requires considerable time and, therefore, it may be desirable to provide a manufactured impervious membrane to save time.

(viii) Function of the Reservoir

The function of the reservoir determines the allowable water loss due to seepage through the embankment and foundation. Accordingly, the embankment section may be relatively more impervious (for conservation reservoirs) or relatively more pervious (for flood control reservoirs). In hydroelectric projects, the upstream face of the dam will be subjected to a "sudden drawdown" condition which may necessitate the provision of a flatter upstream slope.

(ix) Earthquake Activity

In regions of seismic activity, the designer may have to adopt more conservative design features such as better filters, downstream drains of larger capacity, thicker cores of more piping resistant materials, flatter side slopes, longer construction time, and so on.

15.2.2. General Design Criteria for Embankment Dams

The major causes of failure of an embankment dam are overtopping, piping, and earth slides in a portion of the embankment and its foundation (due to insufficient shear strength). Of these, overtopping is the most common cause of complete and catastrophic failure of an embankment dam. The design of an embankment dam must meet the following safety requirements (6, 3):

(*i*) There is no danger of overtopping. For this purpose, spillway of adequate capacity and sufficient freeboard must be provided.

(*ii*) The seepage line is well within the downstream face so that horizontal piping may not occur.

(*iii*) The upstream and downstream slopes are flat enough to be stable with the materials used for the construction of embankment for all conditions during construction, operation, and sudden drawdown.

(*iv*) The shear stress induced in the foundation is less than the shear strength of the foundation material. For this purpose, the embankment slopes should be sufficiently flat.

(*v*) The upstream and downstream faces are properly protected against wave action and the action of rain water, respectively.

(*vi*) There should not be any possibility of free passage of water through the embankment.

(*vii*) Foundation seepage should not result in piping at the downstream toe of the dam.

(*viii*) The top of the dam must be high enough to allow for the settlement of the dam and its foundation.

(*ix*) The foundations, abutments, and embankment must be stable for all conditions of operation (steady seepage and sudden drawdown) and construction.

15.2.3. Freeboard

All embankment dams must have sufficient extra vertical distance between the crest of the embankment dam and the still water surface in the reservoir. This distance is termed the *freeboard* (7) and must be such that the effects of wave action, wave run-up, wind set-up, earthquake, and settlement of embankment and foundation do not result in overtopping of the dam. Normal freeboard is measured with respect to the full reservoir level while the minimum freeboard is measured with respect to the maximum water level in the reservoir (7). An adequate freeboard is the best guarantee against the failure of an embankment dam due to overtopping.

Wave action for freeboard computations is best represented by the wave height and wave length both of which depend on fetch and wind velocity. *Fetch* is defined as the maximum straight line distance over open water on which the wind blows (7). Effective fetch is weighted average fetch of water spread, covered by 45° angle on either side of trial fetch, assuming the wind to be completely non-effective beyond this area. The effective fetch is calculated (7) by drawing fifteen radials on the reservoir contour map at an interval of 6° from a selected point located on the periphery at which fetch is required to be determined (Fig. 15.4). The central radial is drawn in the direction of wind. It should be noted that after the construction of reservoir, the funnelling action of the valley may direct wind towards the dam. As such, the wind direction may preferably be assumed along the maximum fetch line. Each radial runs the full length of the water surface at a given pool elevation. Effective fetch f_e is calculated from

$$f_e = \frac{\sum R_i \cos \alpha_i}{\sum \cos \alpha_i} \tag{15.1}$$

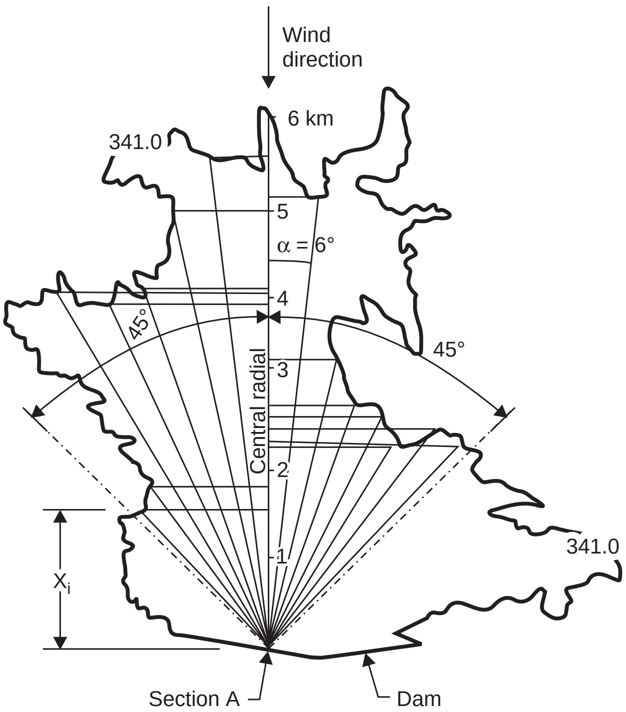


Fig. 15.4 Radials for computations of effective fetch

Here, R_i is the effective length of the i^{th} radial and α_i is the angle between the i^{th} radial and the central radial. Such values of effective fetches are calculated for 2 or 3 different trial fetches and the maximum value of these effective fetches is used for further computations of wave height H_s and the wave period T_s which are, respectively, given as (7)

$$\frac{gH_s}{V^2} = 0.0026 \left[\frac{gf_e}{V^2}\right]^{0.47} \tag{15.2}$$

and

$$\frac{gT_s}{V} = 0.45 \left[\frac{gf_e}{V^2}\right]^{0.28} \tag{15.3}$$

The wave length, L_s (in metres) is obtained from

$$L_s = 1.56\ T_s^2 \tag{15.4}$$

Here, H_s and T_s are the wave height (in metres) and wave period (in seconds) of a significant wave, g the acceleration due to gravity (m/s^2), V the wind velocity (m/s) over the water surface, and f_e is the effective fetch (in metres).

The wind velocities over the water surface are higher than the wind velocities over the land surface and the difference between the two depends upon the fetch and the surrounding terrain conditions. The ratio of the wind velocity over the water surface to the wind velocity over the land surface is given in Table 15.1. For computation of minimum freeboard, the wind velocity is taken as half to two-thirds the wind velocity adopted for calculating normal freeboard (7). For calculation of the normal and minimum freeboard, the wave length and design wave height (H_0) are taken as L_s and 1.67 H_s, respectively (7).

Table 15.1 Ratio of the wind velocity over water surface to the wind velocity over land surface (7)

Effective fetch (km)	1	2	4	6	8	10 and above
The ratio	1.1	1.16	1.24	1.27	1.3	1.31

Wave run-up is the vertical difference between the maximum elevation attained by wave run-up on a slope and the water elevation on the slope excluding wave action. The wave run-up, R for a smooth surface can be obtained from Fig. 15.5. Wave run-up on a rough surface would be less and, hence, the values of R, obtained from Fig. 15.5, are multiplied by a correction factor obtained from Table 15.2.

Table 15.2 Surface roughness correction factor for wave run-up (7)

Type of pitching	*Recommended correction factor*
Cement concrete surface	1.00
Flexible brick pitching	0.80
Hand-placed riprap	
(*i*) laid flat	0.75
(*ii*) laid with projections	0.60
Dumped riprap	0.50

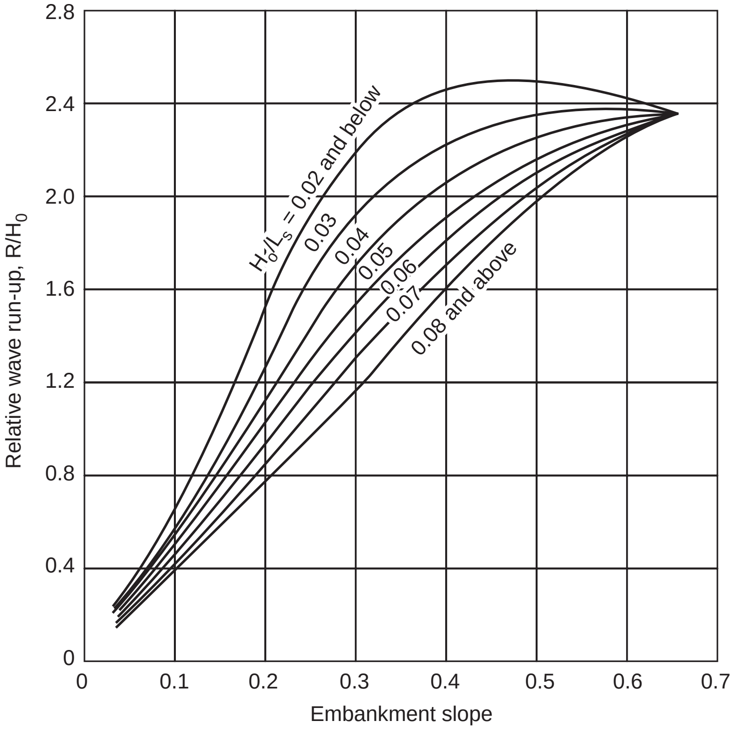


Fig. 15.5 Relative run-up of waves

If the wave run-up is less than the design wave height H_0, the freeboard is governed by the design wave height H_0 (7).

Wind set-up is the result of piling up of the water on one end of the reservoir on account of the horizontal driving force of the blowing wind. The magnitude of the rise of water surface above the still water surface is called the wind set-up or the wind tide (7).

Consider sections 1 and 2, Fig. 15.6, in a reservoir. Let the distance between these sections be dF. Let the depth at section 1 be D. The wind exerts shear stress τ_w on the water surface (assumed horizontal) as a result of which the depth at section 2 is $D + dS$. The shear on the bed of the reservoir is, say τ_b. Applying momentum principle to the control volume between sections 1 and 2, one gets

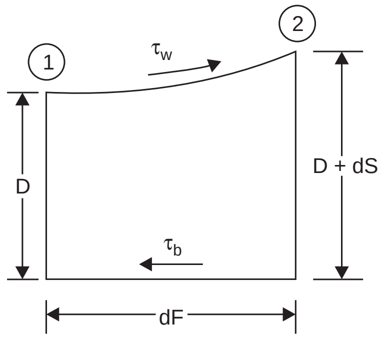


Fig. 15.6 Wind set-up

$$\tau_w\, dF - \tau_b\, dF + \frac{1}{2}\rho g D^2 - \frac{1}{2}\rho g\,(D + dS)^2 = \rho Q\,(U_2 - U_1)$$

Here, ρ is the mass density of water, Q the discharge, and U_1 and U_2 are the velocities of flow at sections 1 and 2, respectively. Since the depth in the reservoir is large, the velocities U_1 and U_2 are negligible and, therefore, the shear stress τ_b too is negligible. If one drops the term containing square of dS (as dS is small), the above equation reduces to

$$\tau_w\, dF = \rho g D dS$$

$$\therefore \qquad dS = \frac{\tau_w dF}{\rho g D}$$

Since, τ_w is proportional to the square of the wind velocity V, the wind set-up S can be written as

$$S \propto \frac{V^2 F}{D}$$

In practice, the wind set-up, S is calculated from the Zuider Zee formula (7):

$$S = \frac{V^2 F}{62{,}000 D} \qquad (15.5)$$

where, V is the velocity of the wind over the water surface (in km/hr), F the fetch in km, and D is the average depth of water in metres along the maximum fetch line.

The freeboard is thus the sum of the height (or wave run-up) and the wind set-up. In order to account for the uncertain effects of seiches (periodic undulations in the reservoir water surface believed to be on account of earthquake, intermittent wind, varying atmospheric pressures, and irregular inflow and outflow of water), and vertical settlement of the embankement and foundation, an additional margin of safety is always added to the computed freeboard. The freeboard (normal as well as minimum) should not, however, be less than 2 m (7). The elevation of the top of the embankment dam is obtained by adding normal freeboard to the full reservoir level and the minimum freeboard to the maximum water level in the reservoir. Obviously, the higher of the two elevations is to be adopted as the elevation of the top of the dam.

Example 15.1 Compute freeboard and the level of top of the dam for the following data:

Full reservoir level	= 340.00 m
Maximum water level	= 342.20 m
Effective fetch	
for normal freeboard	= 3.66 km
for minimum freeboard	= 4.00 km
Wind velocity over land for normal freeboard	= 150 km/hr
Average depth of reservoir	
for normal freeboard	= 29.0 m
for minimum freeboard	= 31.2 m
Embankment slope	= 2.5 (H) : 1(V)

The upstream face is covered with hand-placed stone pitching.

Solution:

Quantity	*For normal freeboard*	*For minimum freeboard*	*Remarks*
Effective fetch (km)	3.66	4.00	
Wind velocity over land (km/h)	150.00	75.00**	
Wind coefficient	1.226	1.240	From Table 15.1
Wind velocity over water,			
V(km/hr)	183.90	93.00	
V(m/s)	51.083	25.83	
Significant wave height, H_s (m)	2.37	1.20	From Eq. (15.2)
Wave period, T_s (s)	4.88	3.71	From Eq. (15.3)
Wave length, L_s (m)	37.15	21.47	From Eq. (15.4)
Design wave height, H_0 (m)	3.96	2.00	$H_0 = 1.67\ H_s$,
Wave steepness, H_0/L_s	0.1066	0.093	
Relative wave run-up, R/H_0	1.6	1.6	From Fig. 15.5
Wave run-up, R (m)	6.336	3.20	
Surface roughness correction factor for wave run-up	0.75	0.75	From Table 15.2
Corrected wave run-up (m)	4.752	2.4	
Wind set up, S (m)	0.069	0.018	From Eq. (15.5)
Freeboard required (m)	4.752 + 0.069 = 4.821	2.4 + 0.018* = 2.418	
Elevation of top of dam (m)	340.0 + 4.821 = 344.821	342.2 + 2.418 = 344.618	

** Assuming wind velocity for minimum freeboard to be half the wind velocity for normal freeboard (7).

* Wave run-up is higher than design wave height (H_0)

Therefore, adopt elevation of top of the dam as 344.821 or, say, 345.00 m.

15.3. ESTIMATION AND CONTROL OF SEEPAGE

15.3.1. Seepage Analysis

The theory of flow through porous media is utilised for the estimation of seepage through an embankment dam and its foundation. As discussed in Sec. 9.3, the governing equation for two-dimensional seepage, as would occur in an embankment dam and its foundation, is the Laplace equation

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = 0 \tag{15.6}$$

Here, h is the seepage head. Equation (15.6) is valid for homogeneous, isotropic, and incompressible soil which is fully saturated with incompressible water. Equation (15.6) can be solved by graphical, analytical, numerical, or some other suitable methods, such as analogue methods. The graphical solution of Eq. (15.6) involves drawing of flownet which has been discussed in Sec. 9.3.

Most of the seepage problems related to embankment dams can be analysed by drawing flownets for sections with single permeability. For example, if the outer shells of a dam are many times pervious than the core, the analysis of the seepage conditions in the core alone may be adequate for such cases. However, in many seepage problems one has to analyse seepage through sections of different permeabilities. For such conditions, a basic deflection rule must be followed in passing from a soil of one permeability to a soil of different permeability. Seepage water needs less energy to flow through a region of relatively higher permeability. Therefore, when water flows from a region of high permeability to one of lower permeability, the flow takes place in such a manner that it remains in the more permeable region for the greatest possible distance. In other words, water seeks the easiest path to travel in order to conserve its energy. Another way of appreciating the seepage behaviour in sections of different permeabilities is that, other factors being the same, smaller area of flow cross-section is needed in the higher permeability region.

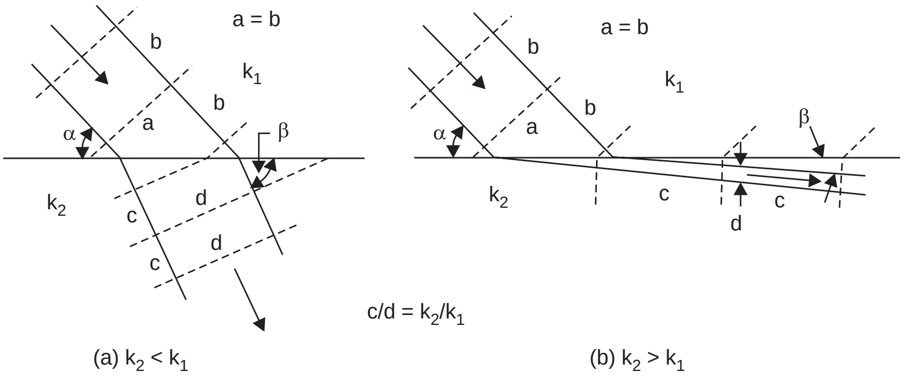


Fig. 15.7 Change in shape of flownet squares on account of regions of different permeability

In seepage through porous medium, the hydraulic gradient is the measure of the rate of energy loss. One would, therefore, expect steep hydraulic gradients in zones of low permeability.

Figure 15.7 shows the deflection of flow lines when they cross boundaries between soils of different permeabilities. The flow lines bend in accordance with the following relationship (8):

$$\frac{\tan\beta}{\tan\alpha} = \frac{K_1}{K_2} \tag{15.7}$$

Also, the areas formed by the intersecting flow and equipotential lines either elongate or shorten according to the following relationship (8):

$$\frac{c}{d} = \frac{K_2}{K_1} \tag{15.8}$$

While drawing flownets for sections with different permeabilities, one must regularly measure the lengths and widths of the figures to ensure that Eq. (15.8) is satisfied.

At times, the compacted embankments and natural soil deposits are stratified rendering them more permeable in the horizontal direction than in the vertical direction. For this anisotropic condition, the velocity components, Eq. (9.42), for two-dimensional flow will be

$$u = -K_x \frac{\partial h}{\partial x} \text{ and } v = -K_y \frac{\partial h}{\partial y} \tag{15.9}$$

Thus, for two-dimensional flow, the continuity equation, Eq. (9.43), with $\partial w/\partial z = 0$ and combined with Eq. (15.9), yields

$$K_x \frac{\partial^2 h}{\partial x^2} + K_y \frac{\partial^2 h}{\partial y^2} = 0 \tag{15.10}$$

Equation (15.10) can, alternatively, be written as

$$\frac{\partial^2 h}{\partial x_t^2} + \frac{\partial^2 h}{\partial y^2} = 0 \tag{15.11}$$

where, $$x_t = x\sqrt{K_y/K_x}$$

Equation (15.11) is the familiar Laplace equation with transformed coordinate system involving x_t and y, and governs the flow in anisotropic seepage condition. To draw a flownet for the anisotropic condition, one needs only to shrink the dimensions of the given cross-section in the direction of greater permeability (8). Having drawn the flownet for the transformed section, it is then reconstructed on the cross-section drawn to the original scale. Obviously, the flownet on the original cross-section would not be composed of squares but of rectangles elongated in the direction of greater permeability.

The effective permeability $\overline{K}$ can be determined by comparing the discharge Δq through any one figure of the transformed section and the corresponding figure of the original section (Fig. 15.8). If Δh is the drop in head between adjacent equipotential lines, then

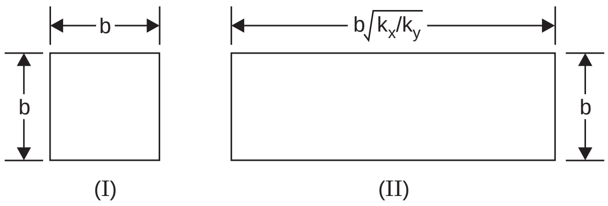


Fig. 15.8 Comparison of flownet figures in the (I) transformed and (II) original section

$$\Delta q = \overline{K} \frac{\Delta h}{b} b = K_x \frac{\Delta h}{b\sqrt{K_x / K_y}} b$$

$$\therefore \qquad \overline{K} = \sqrt{K_x K_y} \tag{15.12}$$

One can also determine the seepage quantity using Eq. (9.46) which is rewritten as

$$q = Kh \frac{N_f}{N_d} \tag{15.13}$$

Equation (15.13) is valid for the isotropic condition but can also be used for the anisotropic condition by replacing K with effective permeability $\overline{K}$ [Eq. (15.12)]. It should be noted that the shape factor (*i.e.,* N_f/N_d) remains the same for both the original and transformed sections.

The major difficulty in the seepage analysis of an embankment dam is that the topmost streamline, *i.e.,* the seepage line or the phreatic line, is not known. The seepage line is defined as the line above which there is no hydrostatic pressure and below which there is hydrostatic pressure (6). If the embankment is composed of coarse material, the capillary effects are negligible and the seepage line is practically the line of saturation. But, in case of an embankment of fine-grained soil, there is saturation without hydrostatic pressure and also a negligible flow occurs in the capillary fringe above the seepage line. The prediction of the seepage line helps in drawing the flownent and determines the piping potential. In case of a homogeneous earth dam founded on impervious foundation, the seepage line cuts the downstream face above the base of the dam unless, of course, special drainage measures are adopted.

The equipotential lines must intersect the seepage line at equal vertical intervals (Fig. 15.9). This requirement permits graphical determination of the seepage line simultaneously while a flownet is being drawn. Alternatively, one can determine the seepage line using Kozeny's solution according to which the equation of seepage line for an embankment with parabolic upstream face and downstream horizontal drain, shown in Fig. 15.10, is expressed as

$$y^2 - y_0^2 + 2xy_0 = 0 \tag{15.14}$$

where, $$y_0 = 2a_0 = \sqrt{d^2 + h^2} - d \tag{15.15}$$

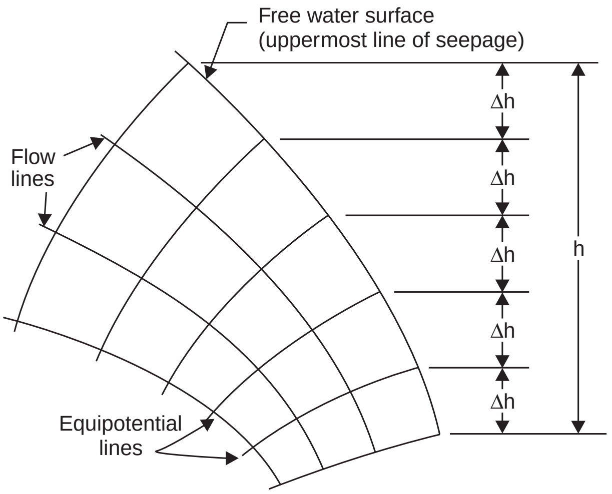


Fig. 15.9 General conditions for line of seepage

Further, the seepage discharge per unit length of embankemnt, q, through the embankment, as per Kozeny's solution, is given as

$$q = Ky_0 = K\left[\sqrt{d^2 + h^2} - d\right] \tag{15.16}$$

where, K is the coefficient of permeability, and other symbols are as explained in Fig. 15.10.

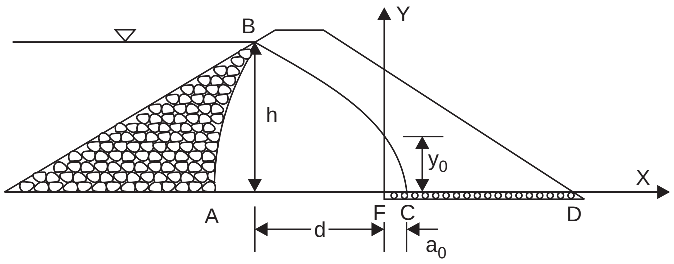


Fig. 15.10 Embankment dam with Kozeny's conditions

It should be noted that the location of seepage line and the point at which it cuts the downstream face (the downstream drain in Fig. 15.10) are dependent only on the cross-section of the dam and are not affected by the coefficient of permeability of the embankment material. Further, actual embankment dams do not have parabolic upstream faces though many of them have downstream horizontal drain. The seepage line follows a parabola, Eq. (15.14), with some departures at the entry and the exit of common types of embankment dams.

15.3.2. Casagrande's Solution for Common Embankment Dams

Casagrande (9) has described important conditions which must be met by flownets at the points of entry, discharge, and transfer across boundaries between dissimilar soils. These conditions have been given in Fig. 15.11. Casagrande (9) has also extended Kozeny's solution for common embankment dams with usual upstream face and the downstream drain other than the horizontal drain.

Casagrande obtained accurate solutions for different embankment sections by drawing flownet and compared the seepage lines with Kozeny's parabolic seepage line (also called 'base parabola'). He, thus, found that in the central portion, the seepage line coincided with the base parabola. Further, the base parabola meets the water surface at the corrected entry point B which lies on the water surface, and is at a distance of 0.3 times the horizontal projection of the water-covered upstream face, from the junction of the water surface with the upstream face (Fig. 15.12). This means that $EB = 0.3\, EG$. The actual seepage line, however, starts from E and is at right angles to the upstream face which is an equipotential surface. It, then, takes a reverse curvature and meets the base parabola tangentially.

When the downstream drain is other than a horizontal drain (in which case angle of the discharge face, $\alpha = 180°$), the actual seepage line departs from the base parabola in the exit region also. For α less than 90°, the seepage line meets the discharge face, *i.e.,* the downstream slope, tangentially at C. For $90° < \alpha < 180°$, the seepage line becomes vertical at the discharge face. Comparing his graphical solutions with the corresponding base parabola, Casagrande obtained a graphical relation between the ratio $\Delta a/(a + \Delta a)$ and the angle of discharge face α (Fig. 15.13). Here, a is the distance from the focus F, along the discharge face, to the point where the seepage line meets the discharge face while $(a + \Delta a)$ is the corresponding distance for the base parabola. Using this graph, Δa and, hence, the corrected exit point C can be determined. The base parabola in the exit region is, therefore, suitably modified so that it meets the discharge face at C.

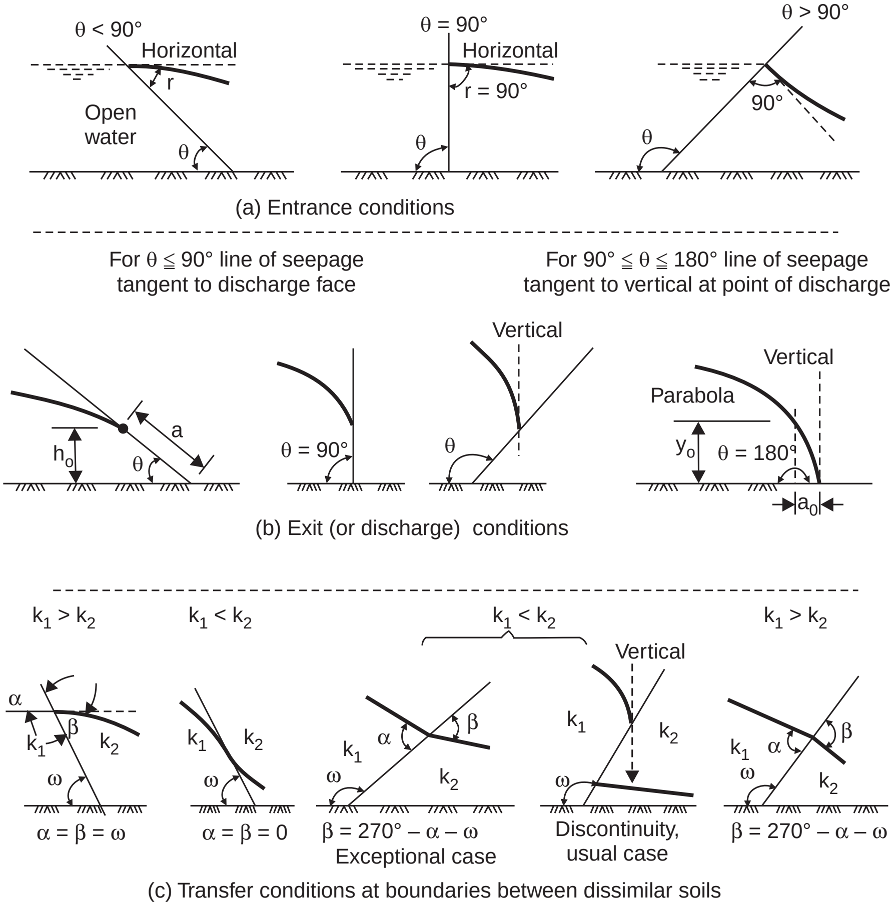


Fig. 15.11 Different conditions for seepage line

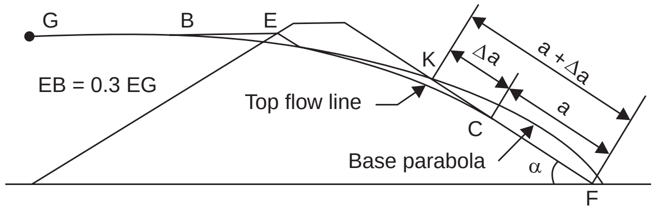
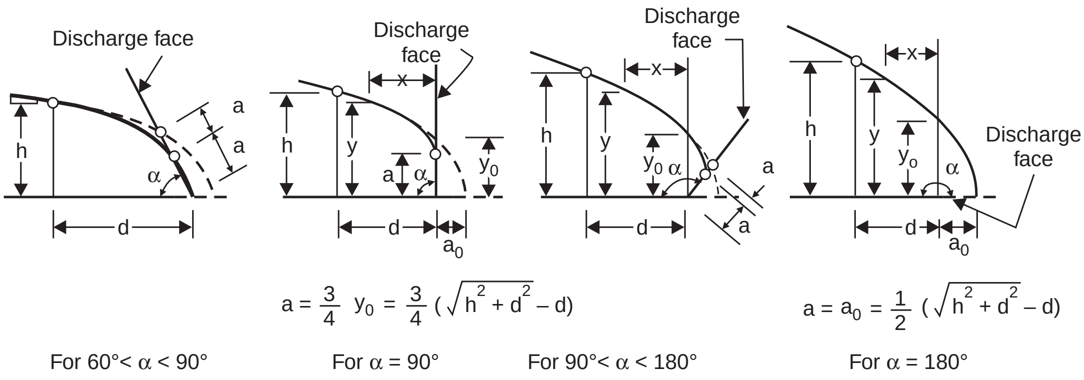
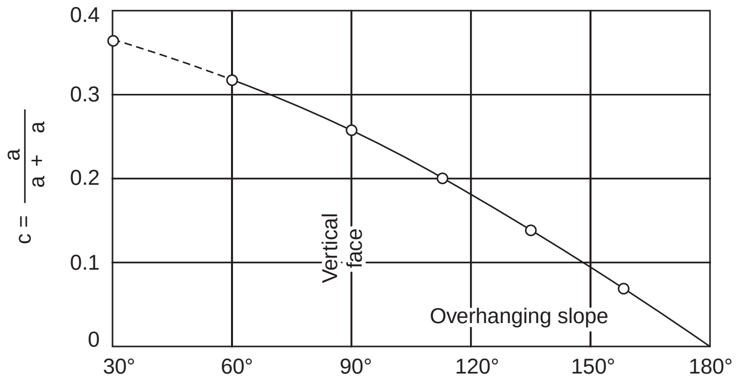


Fig. 15.12 A Casagrande's solution

The seepage quantity can be determined by using Kozeny's equation [Eq.(15.16)] with d being replaced by the horizontal distance between the corrected entry point B and focus F of the base parabola. Alternatively, one can draw the flownet and use Eq.(15.13).



(a) Diagrams for determining a + a



(a) Diagram for determining Δa and a

Fig. 15.13 Correction at the discharge point of base parabola

Example 15.2 Calculate the seepage through an earth dam [Fig. 15.14 (*a*)] resting on an impervious foundation. The relevant data are as follows:

Height of the dam	= 60.0 m
Upstream slope	= 2.75 : 1 (*H* : *V*)
Downstream slope	= 2.50 : 1 (*H* : *V*)
Freeboard	= 2.5 m
Crest width	= 8.0 m
Length of drainage blanket	= 120.0 m

Coefficient of permeability of the embankment material

in *x*-direction = 4×10^{-7} m/s

in *y*-direction = 1×10^{-7} m/s

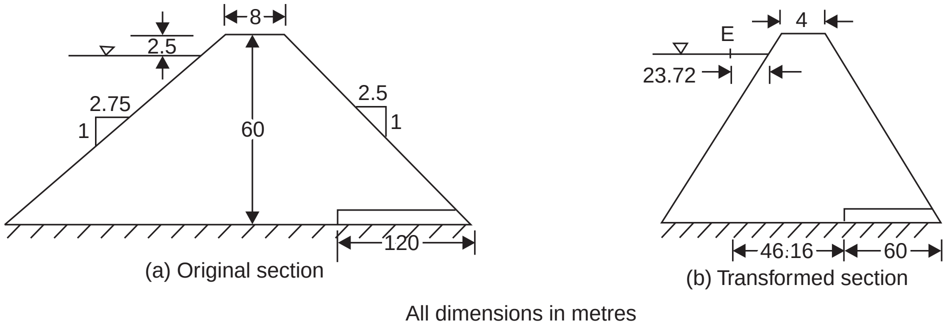


Fig. 15.14 Original and transformed sections for Example 15.2

Solution: Since it is a case of anisotropic permeability, the original section needs to be transformed through the transformation

$$x_t = x\sqrt{K_y/K_x} = x\sqrt{1\times 10^{-7}/4\times 10^{-7}} = x/2$$

With respect to the transformed section [Fig. 15.14 (*b*)]

$$d = 46.16 \text{ m and } h = 57.5 \text{ m}$$

and $$\overline{K} = \sqrt{K_x K_y} = \sqrt{4\times 10^{-7}\times 1\times 10^{-7}} = 2\times 10^{-7} \text{ m/s}$$

From Eq. (15.16)

$$\therefore \quad q = \overline{K}\,[\sqrt{d^2+h^2} - d]$$

$$= 2\times 10^{-7}\,[\sqrt{46.16^2+57.5^2} - 46.16]$$

$$= 55.2\times 10^{-7} \text{ m}^3\text{/s/m}$$

15.3.3. Methods of Controlling Seepage through Embankment and its Foundation

Seepage through an embankment dam and its foundation causes not only the loss of water but, if uncontrolled, also results in piping in the downstream portion of the embankment and foundation. Besides, the stability of slopes is also affected by seepage forces. Piping is the progressive erosion of embankment material due to leaks which develop through the embankment or foundation. Leaks in an embankment are usually caused because of poor construction resulting in insufficiently-compacted or pervious-layered embankment, inferior compaction adjacent to concrete outlet pipes or other structures, and weak bond between the embankment and the foundation or abutments. Piping is caused due to the erosive forces of seepage water tending to move soil particles along with the seepage water. When the forces resisting erosion (*viz*., cohesion, the interlocking stresses, the weight of soil particles as well as the action of the downstream filters, if present) are less than the erosive forces of the seepage water, the soil particles are washed away and piping begins. Earth slides (or sloughing) are related to piping. Sloughing begins when a small amount of material at the downstream toe has eroded and caused a small slump (or slide) leaving behind steeper slope which eventually gets saturated due to seepage water and slumps again. This may continue and cause complete failure of the embankment.

In order to check these effects of seepage, some measures must be taken to control the seepage in embankment dams so that the seepage line is well within the downstream face and the seepage water is suitably collected and disposed of. The methods of seepage control for embankment dams can be grouped into two broad categories:

(*i*) Methods which prevent or reduce the seepage, such as complete vertical barriers (*viz*., rolled-earth cutoffs, steel sheet piles, and concrete walls), grout curtains, upstream impervious blanket, and thin sloping membrane.

(*ii*) Methods which control the seepage water that has entered, such as embankment zoning, horizontal blanket drains, chimney drain, partially penetrating toe drains, and relief wells.

Usually, a combination of these methods is used.

(*i*) *Rolled-Earth Cutoff*

Rolled-earth cutoff [Fig. 15.15 (*a* and *b*)] provides an effective barrier for controlling seepage through pervious foundation of moderate thickness (up to about 25 m) over an impermeable bedrock formation. As can be seen from Fig. 15.15 (*c*), a cutoff penetrating up to 80% of the pervious depth would reduce seepage quantity only by about 50%. Therefore, to be effective, cutoffs must penetrate the full depth of the pervious foundation as shown in Fig. 15.15 (*b*). Such cutoffs are also advantageous in providing full-scale exploration trenches exposing all soil strata, permitting treatment of the exposed bed rock when necessary and increasing the stability of the dam because of effective replacement of the large mass of foundation soil with stronger material. The major difficulty in the construction of a rolled-earth cutoff is the dewatering of the excavated trench and keeping it dewatered until its backfilling.

At times, the foundation is such that the average permeability of the foundation soil decreases with depth below the surface or there is a single continuous impervious layer (over other pervious layers) to which the cutoff can be connected. In such circumstances a partial vertical cutoff [Fig. 15.15 (*a*)] may be useful. Such partial cutoffs extending to a depth of 2 to 3 m should be specified for sites which, otherwise, may not need a seepage barrier. This provision would serve the purpose of continuous excavation through the upper layers of soil so as to better understand the subsoil conditions which, at times, may suggest the need for further excavation or provision of other suitable measures.

(*ii*) *Slurry Trench Method*

In deep alluvial deposits, provision of a rolled-earth cutoff may be expensive and difficult. In such situations, a deep narrow trench is kept open by filling it with a thick slurry of clay and bentonite. This trench, known as slurry trench, is backfilled with different types of soil, cement, and bentonite mixtures or unreinforced or reinforced concrete to act as a seepage barrier.

(*iii*) *Sheet Piling*

Interlocking steel sheet piling and interlocking wood are also used to construct thin cutoffs through deep alluvial foundations of embankment dams. Such thin cutoffs make the best under seepage barrier when alternative cutoff methods are costly and time-consuming. However, small openings in cutoffs and gaps at the bottom or top can result in large amount of seepage. Studies have indicated that a cutoff with 5 per cent open area at one point reduces the seepage by 60 per cent, whereas a cutoff with the same amount of open space equally divided among four openings is only 30 per cent efficient (8). Another drawback of sheet piling is the damage to itself by boulders during the driving process. Sheet piling is more effective in a homogeneous foundation than in a stratified foundation. Sheet piling is less frequently chosen because of its relatively higher cost and inherent leakage through interlocks between individual piles.

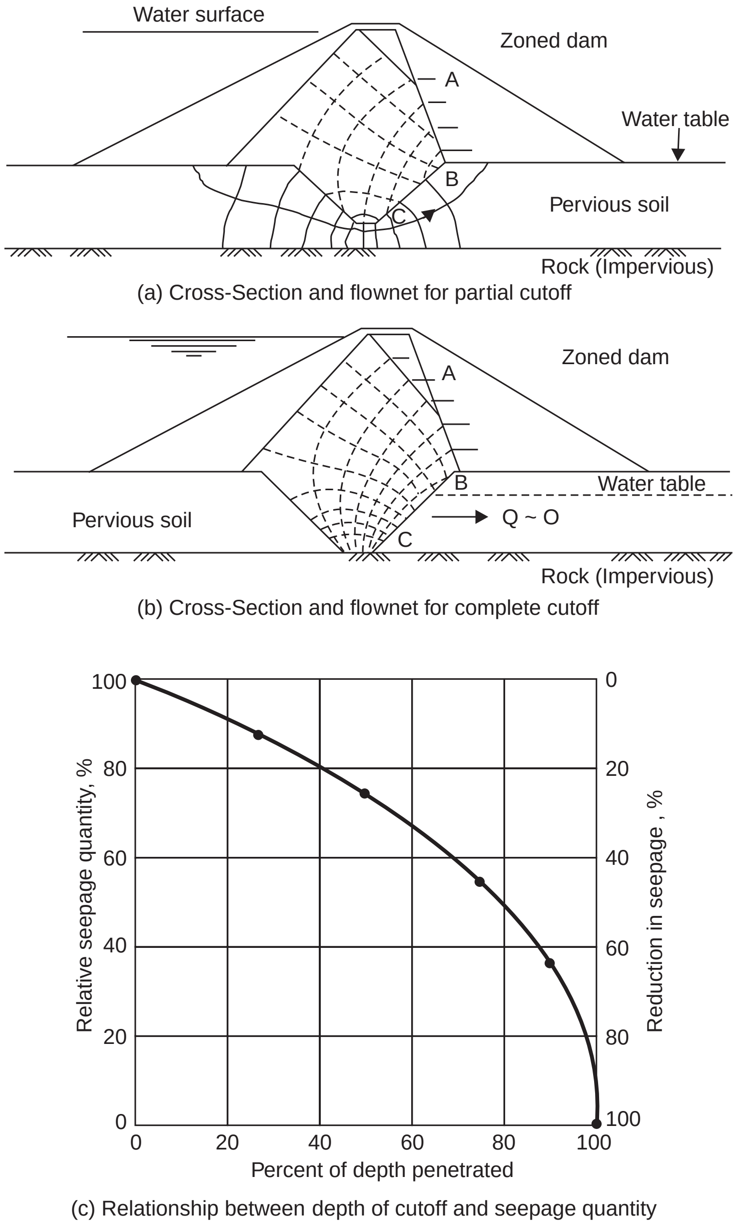


Fig. 15.15 Rolled-earth cutoff

(*iv*) *Concrete Cutoff Walls*

Concrete cutoff walls of 1.5 to 2.0 m thickness can be easily constructed by backfilling the dewatered trenches with unreinforced concrete. The trenches are braced and sheeted to keep them open. Another new method of constructing a concrete cutoff wall is by installing a continuous row of overlapping concrete piers with the help of special drilling rigs or other equipment (4).

(*v*) *Grouting*

Grouting is the process by which fluid cement pastes are pumped through small diameter drilled holes into crevices and joints in rocks for strengthening dam foundation. After setting, the cement paste forms an impervious barrier (or grout curtain) to the seepage water. This method is also used for creating grout curtain in alluvial foundations of embankment dams. However, it is difficult to force cement grout into pores of sizes smaller than about 1.0 mm. To overcome this difficulty, chemical grouts are used in place of cement grouts. Grouting of alluvial deposits is also difficult because of: (*i*) the necessity of keeping the hole open with casing, (*ii*) the impossibility of using packers, and (*iii*) the lack of a proper injection technique. Chemical grouts, used until recently, were costly and not much effective either. The "Soletanche" method (4), however, enables effective grouting of alluvial deposits. In this method, a row of holes (125 to 250 mm diameter) at a spacing of about 3.0 m are drilled through the alluvium up to bedrock by using drilling mud and suitable drilling bits. Grout pipes (diameter about 60 mm) having circumferential rows of holes at vertical intervals of about 300 mm are set to bedrock in the drilled hole. The circumferential holes were covered with 100 mm long, tightly fitting rubber sleeves. The annular space between the grout pipe and the drilled hole is filled with a brittle clay-cement grout (Fig. 15.16). Grouting of the alluvium is then carried out from the bottom up through each of the circumferential holes in the grout pipe. The grout mix is pumped into each circumferential hole under pressure through a smaller interior grout pipe which has a set of two rubber packers spaced 300 mm apart. In this way the grout pressure can be confined to the short length of the grout pipe opposite each of the circumferential holes and the rubber sleeve.

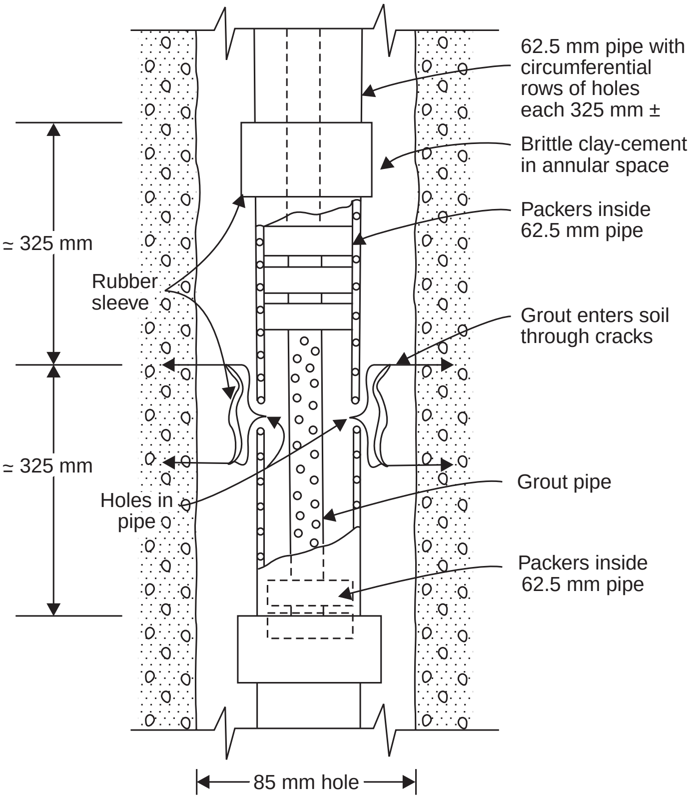


Fig. 15.16 Grouting of alluvial deposits

Grout mix is thus forced out of the rubber sleeve which expands slightly. The mix cracks the thin cylinder of clay-cement grout surrounding the pipe. The grout mix then enters the soil through these cracks. The grout mix is usually made up of clay, cement, and water in varying proportions and a predetermined amount (usually 50% of soil volume or 175% of soil voids) of the mix is pumped at each elevation.

Grout curtains may reduce seepage by significant amounts but their effect on hydrostatic pressure is relatively poor (8).

(*vi*) *Horizontal Upstream Impervious Blanket*

When the foundation soil is homogeneous and extends to large depths, a horizontal upstream impervious blanket (Fig. 15.17) offers an effective method of reducing the underseepage by increasing the length of the seepage path. Such blankets must be constructed using impervious soil in the same manner as the impervious core of the dam. At sites where a natural surface blanket of impervious soil already exists, it may be necessary only to scarify the upper surface and recompact it with proper water content and also fill the gaps to make a continuous impervious seal. The thickness and length of such blankets depend on the permeability of the blanketing material, the stratification and thickness of the pervious foundation, and the reservoir depth. Thickness ranging from 0.6 to 3 m is generally used. If the blanket material is very impervious compared to the natural foundation soil so that the seepage through the blanket is negligible, the length of the blanket can be related to the reduction in underseepage. Considering Fig. 15.17, the underseepage Q in the absence of the blanket is given as

$$Q = K_f \frac{H}{x_d} Z_f \tag{15.17}$$

in which K_f is the coefficient of permeability of the foundation soil, and Z_f is the thickness of the foundation. If the provision of the blanket of length x reduces the underseepage to pQ, then

$$pQ = K_f \frac{H}{x_d + x} Z_f \tag{15.18}$$

Combining Eqs. (15.17) and (15.18)

$$p = \frac{x_d}{x_d + x} \tag{15.19}$$

or $$x = x_d \frac{(1 - p)}{p} \tag{15.20}$$

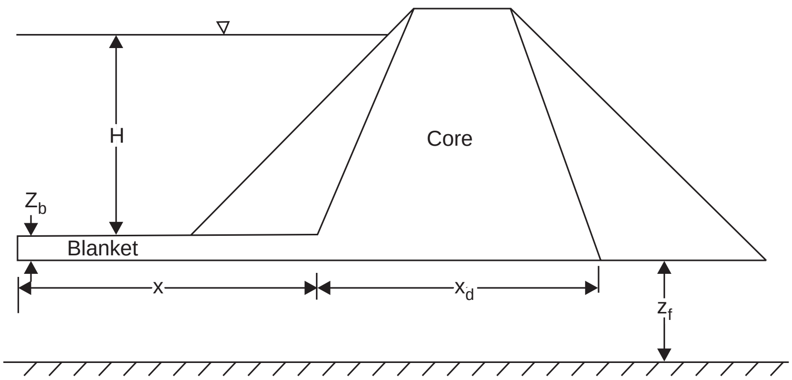


Fig. 15.17 Horizontal upstream impervious blanket

It may, however, not always be possible to have an impervious blanket through which the seepage can be considered negligible. In such cases, there is an optimum length of the blanket and there is no significant reduction in the underseepage with further increase in the length of the blanket. The effectiveness of such blankets (Fig. 15.18) is analysed by using the following Bennett's fundamental differential equation (10):

$$\frac{d^2h}{dx^2} = a^2h \tag{15.21}$$

where, h is the head loss through the blanket which equals the difference of the head on the two sides of the blanket under which percolation takes place and a is given as:

$$a^2 = \frac{K_b}{K_f Z_f Z_b} \tag{15.22}$$

Here, K_f and K_b are coefficients of permeability of the foundation, and the blanket material, respectively. For a blanket of constant thickness Z_b and infinite horizontal extent, Bennett (10) obtained

$$x_r = \frac{1}{a} \tag{15.23}$$

where, x_r is termed the effective length of the blanket and is defined as the length of the prism of foundation which will be able to carry the actual foundation seepage under the same head loss h_0 (up to the end of the blanket) with a linear hydraulic gradient, *i.e.,*

$$q_f = K_f \frac{h_0}{x_r} Z_f \tag{15.24}$$

For the case of blanket of constant thickness and finite length (*i.e.,* the part of the foundation seepage enters from the reservoir bed upstream of the blanket), Bennett (10) obtained

$$x_r = \frac{e^{2ax} - 1}{a\,(e^{2ax} + 1)} = \frac{1}{a} \tan h\,(ax) \tag{15.25}$$

Here, x_r represents the effective length of the blanket up to a distance x downstream from its upstream end ($x = 0$). If L is the total length of the blanket, then its effective length is given as

$$x_r = \frac{1}{a} \tan h\,(aL) = \frac{e^{2aL} - 1}{a\,(e^{2aL} + 1)} \tag{15.26}$$

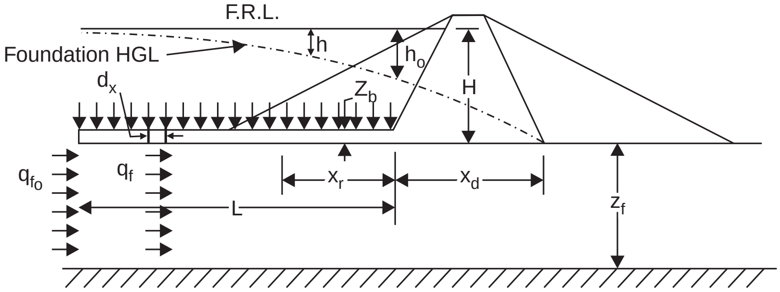


Fig. 15.18 Definition sketch for Bennett's analysis

Having determined the effective length of the blanket using a suitable equation [Eq. (15.23) for infinite length and Eq. (15.26) for finite length], one can determine the reduction in the foundation seepage as follows:

If x_d is the base width of the impervious core of the dam, the foundation seepage is given as

$$q_{f_1} = K_f \frac{H}{x_d} Z_f \tag{15.27}$$

When an impervious blanket, whose effective length is x_r, is available, the foundation seepage is given as

$$q_{f_2} = K_f \frac{H}{x_r + x_d} Z_f \tag{15.28}$$

Here, H is the total head. Also, the loss of head up to the end of the blanket will be equal to

$$\frac{x_r}{x_r + x_d} H$$

The optimum length of an upstream impervious blanket x_o is given (10) by the expression $ax_o = \sqrt{2}$. Length of the blanket beyond x_o will increase its effective length x_r only marginally.

(viii) Zoning

Most often embankment dams and particularly earth dams have an internal impervious core surrounded on both sides by outer sections called shell. The core provides water tightness to the embankment. The core can either be a thin sloping core, [Fig. 15.2 (*a*)], or a somewhat thicker sloping core, [Fig. 15.2 (*b*)] or even a thick central core [Fig. 15.2 (*c*)]. Impervious core dissipates hydrostatic energy rapidly but increases the chances of piping due to the presence of large hydraulic gradients. Therefore, at the boundaries of the core and shells, filters must always be used. Whenever sufficient quantity of highly impermeable soils are readily available, a thick central core type embankment [Fig. 15.2 (*c*)] should be constructed. A thick core provides wide zone for energy dissipation, relatively low hydraulic gradients, and longer contacts with foundations. Dams having extremely narrow cores induce large hydraulic gradients within the embankment.

(ix) Downstream Drains

Dams with heights of more than about 6 m should always be provided with some type of downstream drain which is constructed using material many times more pervious than the embankment soil. Such drains reduce the pore water pressures in the downstream shell and, thus, increase the stability of the downstream slope against sliding. These downstream drains could be in the form of toe drains, [Fig. 15.1 (*a*)], horizontal blanket drains [Fig. 15.1 (*b*)], chimney drains [Fig. 15.1 (*c*)], strip drains [Fig. 15.19] or partially penetrating toe drains [Fig. 15.20] depending upon the height of the dam, the cost and availability of previous material, and the permeability of the foundation.

Horizontal Blanket Drains: These drains [Fig. 15.1 (*b*)] are widely used for dams of low to moderate heights and are highly effective only on relatively uniform foundations and in non-stratified embankments. Such drains collect seepage from the embankment as well as the downstream portion of the foundation and accelerate the consolidation of the foundation. Such drains are, however, not suitable for stratified embankments. These drains usually extend from the downstream toe to the upstream ranging in length from 25 to 100 per cent of the projected length of the downstream slope. The thickness of such drains should always be more than 1 m and should be sufficient to be able to carry the maximum anticipated seepage with a conservative factor of safety. The drain would consist of relatively coarse particles and should, therefore, be surrounded by suitable filter layers to prevent the migration of finer material of embankment or foundation. At sites where pervious material is available in small amount, the general effect of horizontal blanket can still be obtained by constructing strip drain (Fig. 15.19).

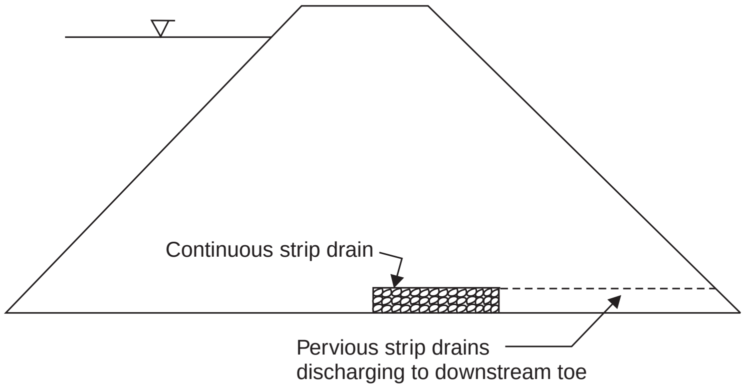


Fig. 15.19 Strip drain

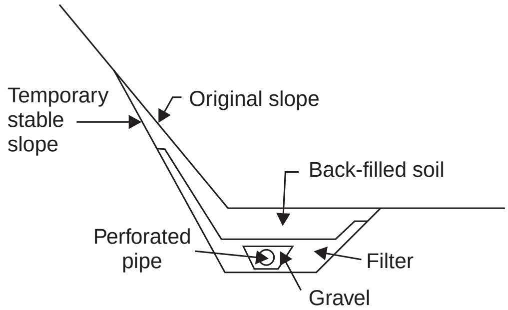


Fig. 15.20 Partially penetrating toe drain

Toe Drains: If sufficient quantity of boulder or quarried rock is available, a toe drain (or rock-toe) [Fig. 15.1 (*a*)] of height about 0.25 to 0.35 times the height of the embankment will be very effective in controlling seepage from stratified embankments. The rock-toe also protects the lower part of the downstream slope from tail-water erosion. Generally, the inner slope of rock-toe is provided at 1 : 1.

Chimney Drains: Chimney drains [Fig. 15.1 (*c*)] combine the advantages of horizontal blanket drains as well as toe drains. These drains can completely intercept embankment seepage irrespective of the extent of embankment stratification. Besides, these drains keep the seepage line well below the downstream face. For effective functioning, these drains and their outlets must have sufficient permeability to discharge seepage water without building up excessive head.

Partially Penetrating Toe Drains: If undesirable foundation seepage conditions are likely to develop during the operation of a dam, a partially penetrating toe drain (Fig. 15.20) should be constructed so as to improve the seepage conditions. The perforated pipes are connected to gravity outlets. These drains will not be effective when the foundation is such that the drains are separated from underlying pervious strata by impervious layers. In such cases, relief wells are more suitable.

(*x*) *Relief Wells*

Relief wells (Fig. 15.21) are provided when pervious strata of embankment foundation are too deep to be penetrated by rolled-earth cutoffs or toe drains. Relief wells can penetrate most pervious water-bearing strata and relieve uplift pressures effectively. Spacing between the relief wells should be small enough to lower the water pressure to the desired safe level. Relief wells must penetrate through the complete depth of the pervious foundation, if possible. A relief well should preferably have an interior perforated pipe (*i.e.,* the well screen) with a minimum diameter of 15 cm or larger, if heavy foundation seepage is anticipated. The annular space surrounding the well screen is backfilled with graded filter. However, near the surface, the annular space is backfilled with impervious soil or concrete so that upward flow of water outside the relief well pipe may not occur.

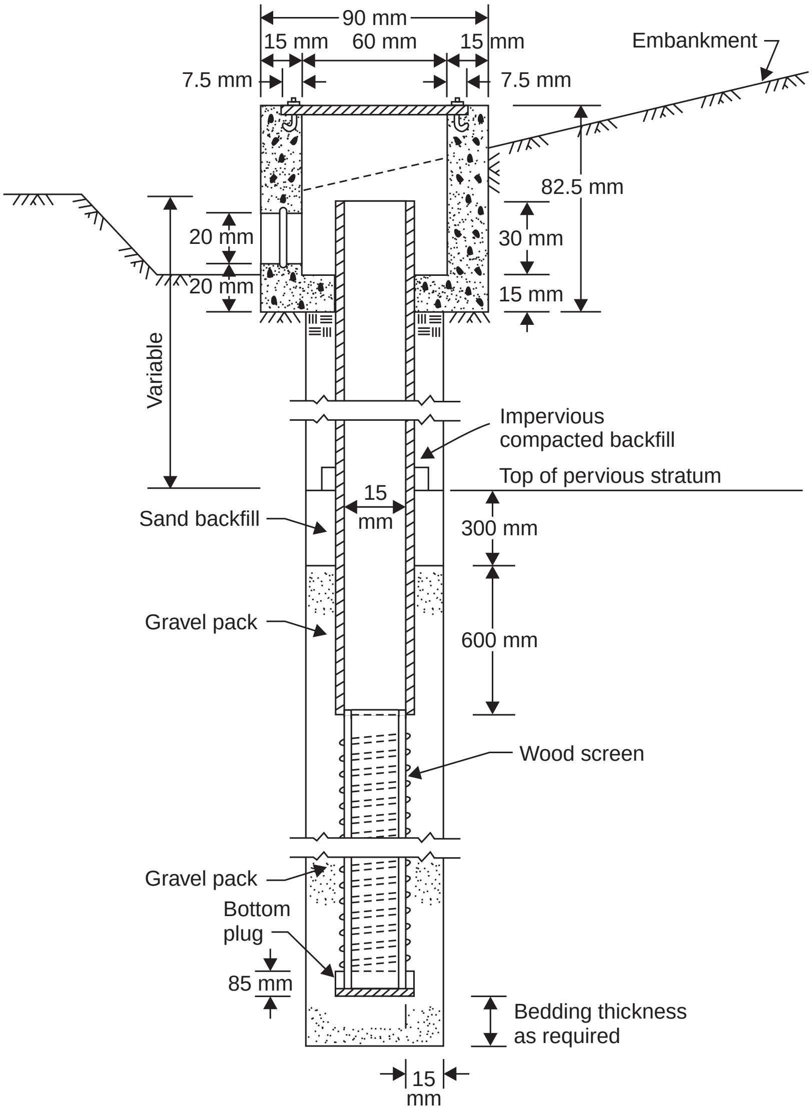


Fig. 15.21 Gravael-packed relief well screen (4)

Example 15.3 Using Bennett's solution for upstream impervious blanket, find the discharge through the foundation of an earth dam having a central core (Fig. 15.22) for the following data:

Water depth over upstream blanket	= 30.0 m
Crest width	= 8.0 m
Freeboard	= 2.0 m
Upstream and downstream slopes of the core	= 1 : 1
Thickness of the impervious blanket	= 2.0 m
Length of the blanket (connected to the core)	= 80.0 m
Depth of pervious foundation	= 30.0 m
Coefficient of permeability of	
(*i*) foundation soil	= 2×10^{-4} m/s
(*ii*) core and blanket soil	= 2×10^{-6} m/s

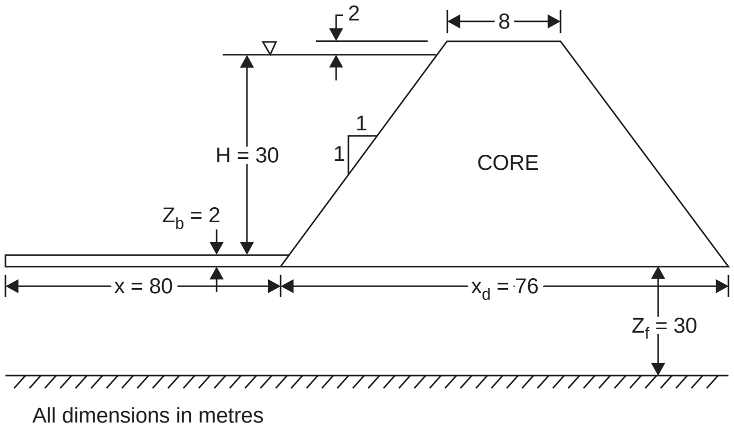


Fig. 15.22 Sketch for Example 15.3

Solution: Using Eq. (15.22)

$$a = \sqrt{\frac{K_b}{K_f Z_f Z_b}} = \sqrt{\frac{2 \times 10^{-6}}{2 \times 10^{-4} \times 30 \times 2}} = 0.0129$$

$\therefore$ Effective length of blanket,

$$x_r = \frac{1}{a} \tan h(ax)$$

$$= \frac{1}{0.0129} \tan h(0.0129 \times 80) = 60.0 \text{ m}$$

From Eq. (15.28),
$$q_f = K_f Z_f \frac{H}{x_r + x_d} = 2 \times 10^{-4} \times 30 \times \frac{30}{60 + 76}$$

$$= 1.324 \times 10^{-3} \text{ m}^3\text{/s/m}$$

15.3.4. Filters

In a homogeneous embankment, the individual soil particles subjected to seepage forces cannot move because they are held in place by the neighbouring soil particles. But, at the boundaries between soils of different sizes, the finer soil particles may be washed into the void spaces of the coarser material. To prevent such migration of soil particles, it should always be ensured that the relative gradation of adjacent soil zones meet established filter criteria. If the difference in size of soils of the fine and coarse zones of an embankment is too large to satisfy the filter criteria, zones of intermediate gradation, known as filters must always be provided in between the fine zone and coarse zone.

There are two main conflicting requirements of a filter layer:

(*i*) the filter layer must be more pervious than the protected soil so that the filter acts as a drain, and

(*ii*) the size of the particles used in the filter layer must be small enough to prevent migration of the particles of the protected soil into the voids of the filter layer.

Further, the filter layer should be sufficiently thick to provide good distribution of all particle sizes throughout the filter (11). The following rules are widely used for the design of filter layer (4, 11):

(*i*) $$\frac{D_{15} \text{ of the filter}}{D_{15} \text{ of the protected soil}} \geq 5$$

(*ii*) $$\frac{D_{15} \text{ of the filter}}{D_{85} \text{ of the protected soil}} \leq 5$$

(*iii*) $$\frac{D_{50} \text{ of the filter}}{D_{50} \text{ of the protected soil}} < 25$$

(*iv*) The gradation curve of the filter should have approximately the same shape as the gradation curve of the protected soil.

(*v*) Where the protected soil contains a large percentage of gravels, the filter should be designed on the basis of the gradation curve of the portion of the material which is finer than the particles passing one-inch sieve.

(*vi*) Filters should not contain more than about 5% of fines passing no. 200 sieve, and the fines should be cohesionless.

Here, D_{15}, D_{50}, and D_{85} represent the particle sizes which are, respectively, coarser than the finest 15, 50, and 85 per cent of the soil, by weight. These filter criteria are based on studies with non-cohesive soils and take into consideration only the grain size of the protected soil. As such, these rules may be conservative, particularly for clays which can resist piping action because of cohesion.

Theoretically, the required thickness of a horizontal filter is very small—about 15 cm for sand and 30 cm for gravel. But, from practical considerations, a minimum thickness of 1.0 m is desirable (11). For vertical or inclined filters, the minimum width of filters is about 2 to 3 m for convenience in construction.

15.4. STABILITY ANALYSIS OF EMBANKMENT DAMS

Quantitative assessment of the stability of slopes is very important in the design of embankment dams. Prior to 1935, the side slopes of embankment dams were selected purely on the basis of past experiences and preferences of designers. During the last fifty years there has been considerable improvement in the understanding of soil shear strength, laboratory methods of soil testing, and the methods of stability analysis. As a result of all these developments, it is now possible to quantitatively assess the stability of slopes of an embankment dam.

15.4.1. Critical Conditions for the Stability of an Embankment Dam

The following conditions are considered critical for the stability of an embankment dam (12).

(i) During Construction With or Without Partial Pool

When an embankment is constructed, water is added to soil during compaction. This increases pore pressures and may cause failure of both upstream (if there is no water in the reservoir) and downstream slopes. During construction, the reservoir is usually empty and, hence, both the upstream and downstream slopes need to be analysed. The magnitude and the distribution of the construction pore pressures depend primarily on the construction moisture content in the embankment, natural moisture content in the foundation, soil properties, construction rate, dam height, and internal drainage. Since high construction pore pressures exist only during the first few years of the life of the dam (when reservoir may not have been filled completely), more conservative and expensive design is avoided and, instead, suitable steps are taken to reduce the construction pore pressures. These steps include:

(*i*) compacting impervious core at an average moisture content which is 1–3 per cent below the optimum moisture content,

(*ii*) making impervious core section thinner,

(*iii*) providing internal drains within impervious core section to accelerate pore pressure dissipation,

(*iv*) accepting a lower factor of safety on the plea that a slide during construction would not cause catastrophic failure and only delay the completion of the dam, and

(*v*) having longer construction period so that some pore pressure is dissipated from the already-constructed fill before laying another layer.

For analysing the stability of slopes during construction, one needs information about pore pressure. As per USBR's simple approach, the pore pressure head during construction equals, approximately, 1.25 times the height of the fill. Alternatively, one can measure pore pressures in sealed laboratory specimens which have been compacted and subjected to increasing stresses simulating the field conditions in respect of the water content, densities and increasing stresses, and use them for analysis. However, there can be some difficulties in simulation. If pore pressures are allowed to dissipate during construction by causing break in construction, the pore pressures are estimated (12) by using the following Bishop's method which uses Hilf's equation.

Bishop's method requires the knowledge of volume change – effective stress relationship [$(\Delta V/V)$ v/s σ'] for the compacted fill. This is determined experimentally by testing representative specimens. The volume change – pore pressure relationship [$(\Delta V/V)$ v/s Δu] is obtained for each stage of construction using Boyle's law and Henry's law (for solubility of air in water).

Any sample of compacted fill having a volume of V_o , Fig. 15.23 includes volume of soil grains V_s and volume of voids V_v which equals the sum of the volume of water V_w and volume of air V_a occupying the voids. The volume of air dissolved per unit volume of water is defined as Henry's coefficient of solubility, H whose approximate value is 0.02. The degree of saturation S_o is the ratio of volume of water and volume of voids. Thus,

$$
\begin{aligned}
\text{Initial volume of free air} \quad &= V_a + \text{dissolved air} \\
\text{(at initial pressure } p_o\text{)} \quad &= (V_v - V_w) + HV_w \\
&= V_v - V_v S_o + HS_oV_v \\
&= V_v(1 - S_o + S_oH)
\end{aligned}
$$

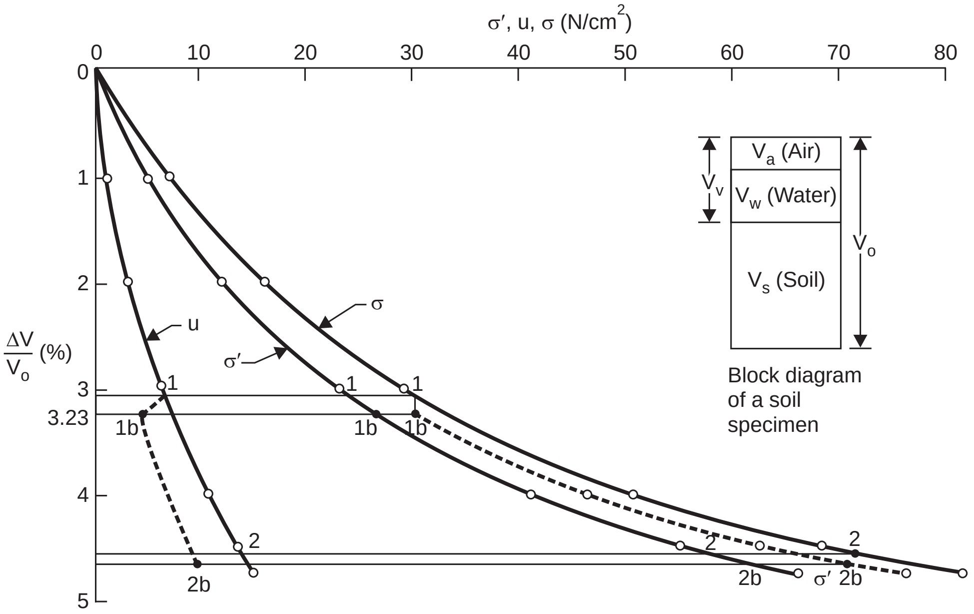


Fig. 15.23 Variation of u, σ' and σ with $\Delta V/V_o$

At a new pressure p, the volume of air, using Boyle's law, is, therefore, given as

$$\frac{p_o}{p}\,V_v(1-S_o+S_oH)$$

Therefore, change in volume of air, $\Delta V = (p_o/p)\,V_v\,(1-S_o+S_oH) - V_v\,(1-S_o+S_oH)$

$$p\,\{\Delta V + V_v\,(1-S_o+S_oH)\} = p_oV_v\,(1-S_o+S_oH)$$

$$\frac{p_o}{p} = 1+\frac{\Delta V/V_v}{1-S_o+S_oH}$$

$$= 1+\frac{\Delta V/n_oV_o}{1-S_o+S_oH}$$ in which n_o is the initial porosity of soil.

$$1-\frac{p_o}{p} = -\frac{\Delta V/V_o}{n_o\,(1-S_o+S_oH)}$$

$$p-p_o = -\frac{(\Delta V/V_o)\,p}{n_o\,(1-S_o+S_oH)}$$

$$= -\frac{(\Delta V/V_o)\,p_o}{n_o\,(1-S_o+S_oH)\,(p_o/p)}$$

$$= -\frac{(\Delta V/V_o)\,p_o}{n_o\,(1-S_o+S_oH)\left(1+\dfrac{\Delta V/V_o}{n_o\,(1-S_o+S_oH)}\right)}$$

$$= -\frac{(\Delta V/V_o)\,p_o}{\dfrac{\Delta V}{V_o}+n_o\,(1-S_o+S_oH)}$$

If the initial pressure p_o is the atmospheric pressure, p_a then change in pore pressure, $\Delta u = p - p_o = p - p_a$.

Therefore, $$\Delta u = -\frac{\left(\dfrac{\Delta V}{V_o}\right)p_a}{\dfrac{\Delta V}{V_o} + n_o\,(1 - S_o + S_o H)} \qquad (15.29)$$

Equation (15.29) is known as Hilf's equation. This equation is applicable under the following conditions:

(*i*) There is only vertical compression and no lateral bulging of the fill.

(*ii*) Pressures in pore water and pore air are the same. Strictly speaking, Hilf's equation gives pore air pressure (say, U_a) whereas the effective stress depends on pore water pressure (say, U_w) which equals the sum of U_a and capillary pressure U_c which is negative for unsaturated condition and zero for saturated condition. Since earth fills are usually placed at 80 to 90 per cent degree of saturation, the capillary pressure is usually small and, therefore, neglected.

(*iii*) Decrease in volume of fill is due to compression of pore air and dissolution of pore air in pore water.

(*iv*) There is no dissipation of pore pressure during construction.

(*v*) Boyle's and Henry's laws are applicable.

Further, when all the air in pores has gone into solution,

$$\Delta V = V_a = V_v - V_w = V_v - V_v S_o = V_v\,(1 - S_o)$$

$$\therefore \qquad \frac{\Delta V}{V_o} = \frac{V_v}{V_o}\,(1 - S_o)$$

$= n_o(1 - S_o)$ with negative sign, of course.

Thus,

$$\Delta u = \frac{n_o\,(1 - S_o)\,p_a}{n_o\,(1 - S_o + S_o H) - (1 - S_o)n_o}$$

$$= \frac{(1 - S_o)p_a}{S_o H} = \frac{(1 - S_o)V_v\;p_a}{HV_w}$$

and, $$\Delta u = \frac{p_a\,V_a}{H\,V_w} \qquad (15.30)$$

when all the air in pores has gone into solution, the soil is fully saturated and any further increase in load ($\Delta\sigma$) results in equal increase in pore pressure (Δu) as there is no air to get compressed.

In order to prepare total stress versus pore pressure relationship for an earthfill being constructed in stages, following steps are followed:

(*i*) Using experimental data, plot σ' against $\Delta V/V_o$, Fig. 15.23.

(*ii*) For different assumed values of $\Delta V/V_o$, compute Δu using Hilf's equation, Eq. (15.29) and the total stress $\sigma = \sigma' + \Delta u$. Carry out these computations till either $\sigma = \rho g H_1$ or

$[\Delta V/V_o] = n_o\,(1 - S_o)$ *i.e.*, when all pore air has gone into solution. Here, H_1 is the height of the dam (or earthfill) at the end of the relevant stage of construction.

(*iii*) Plot σ v/s $\Delta V/V_o$ and Δu v/s $\Delta V/V_o$, Fig. 15.23.

(*iv*) Let the pore pressure Δu at the end of the first stage be dissipated by x per cent before the beginning of the second stage.

Pore pressure at the beginning of the second stage

$$\Delta u_{1b} = \left(1 - \frac{x}{100}\right) \Delta u_1$$

$$\therefore \qquad \sigma'_{1b} = \sigma'_1 + \frac{x}{100}\,\Delta u_1$$

and $$\sigma_{1b} = \sigma'_{1b} + \left(1 - \frac{x}{100}\right) \Delta u_1 = \rho g H_1$$

Also, $$p_{ob} = p_o + \Delta u_{1b} \text{ (in absolute terms)}$$

With absolute pore pressure p_{ob}, Hilf's equation can again be used to compute rise in pore pressure with further loading for the second stage of loading provided that the values of S_o and n_o are recalculated at 1*b*. During the loading stage under undrained conditions, the degree of saturation S can be expressed as follows:

$$S = \frac{\text{Initial volume of water}}{\text{volume of voids}} = \frac{S_o V_v}{V_v + \Delta V} = \frac{S_o}{1 + \dfrac{\Delta V}{V_v}}$$

Since $$\Delta V = \frac{p_o}{p}\,V_v\,(1 - S_o + S_o H) - V_v\,(1 - S_o + S_o H)$$

$$\frac{\Delta V}{V_v} = (1 - S_o + S_o H)\left(\frac{p_o}{p} - 1\right)$$

For the beginning of the second stage, the degree of saturation S_{ob} corresponding to new pore pressure (gauge) p_{ob} is, therefore, expressed as:

$$S_{ob} = \frac{S_o}{1 + \left(\dfrac{p_o}{p_{ob}} - 1\right)(1 - S_o + S_o H)}$$

Similarly, new value of porosity at stage 1*b*,

$$n_{ob} = \frac{\text{New volume of voids}}{\text{Initial volume}}$$

$$= \frac{V_v + \Delta V_{1b}}{V_o} = n_o + \frac{\Delta V_{1b}}{V_o}$$

$$= n_o + \left(\frac{\Delta V}{V_o}\right)_{1b}$$

Here, $\left(\dfrac{\Delta V}{V_o}\right)_{1b}$ is the value of $\Delta V/V_o$ at 1*b* and is negative.

Here, it has been implicity assumed that there is no drainage during the construction (loading stage) and drainage is only during the periods when construction is stopped temporarily for dissipation of pore pressure. However, there will be some drainage during the construction which can be accounted for by applying a suitable dissipation factor at the end of each step of construction.

(*v*) Using now 1*b* as origin [*i.e.,* $\Delta u = 0$ and $(\Delta V/V_o) = 0$], plot Δu v/s $\Delta V/V_o$ and σ v/s $\Delta V/V_o$ using steps (*ii*) and (*iii*) with new values of porosity n_{ob} and degree of saturation, S_{ob} computed in step *iv*.

(*vi*) Likewise, computations can be carried out for further stages of construction, if any.

(*vii*) Plot σ versus u on a separate graph sheet from the above results,

Above steps of computations have been illustrated in the following example.

Example 15.4 A fill is placed at an initial saturation of 80% (S_o) and initial porosity of 37.5% (n_o). The average unit weight of the compacted fill is 20,000 N/m^3. The dam is raised to a height of 15 m in the first stage after which there is a gap during which one-third of pore pressure will have dissipated. In the next stage, the dam is raised to a height of 35 m. Plot u v/s σ. The variation of $\Delta V/V_o$ v/s σ' is as follows:

$\Delta V/V_o$(%)	1	2	3	4	4.5	4.75
σ' (N/cm^2)	5.0	12.5	23	40	54.5	66.0

Solution: During the first stage:

$$\Delta u = -\frac{\left(\dfrac{\Delta V}{V_o}\right)p_o}{\left(\dfrac{\Delta V}{V_o}\right) + n_o\,(1 - S_o + S_o H)}$$

with p_o = atmosheric pressure = 10.3 N/cm^2 and H = 0.02

$$= -\frac{10.3\left(\dfrac{\Delta V}{V_o}\right)}{\dfrac{\Delta V}{V_o} + 0.375\,(1 - 0.80 + 0.80 \times 0.02)} = -\frac{10.3\left(\dfrac{\Delta V}{V_o}\right)}{0.0798 + \left(\dfrac{\Delta V}{V_o}\right)}.$$

Using this equation and the curve $\dfrac{\Delta V}{V_o}$ v/s σ', the following table can be prepared.

$\Delta V/V_o$(%)	Δu	σ'	σ
1	1.48	5.0	6.48
2	3.45	12.5	15.95
3	6.21	23.0	29.21
4	10.35	40.0	50.35
4.5	13.32	54.5	67.82
4.75	15.15	66.0	81.15

At the end of the first stage of construction (1 on Fig. 15.23)

$$\sigma = \gamma H = 20000 \times 15 \text{ N/m}^2 = 300000 \text{ N/m}^2$$

$$\sigma = 30 \text{ N/cm}^2$$

For $\sigma = 30 \text{ N/cm}^2$

$$\frac{\Delta V}{V_o} = 3.06\% \qquad \text{from graph}$$

$\therefore$ $\sigma' = 23.3 \text{ N/cm}^2$ from graph

and $u = 6.7 \text{ N/cm}^2$ from graph

After one-third of pore pressure dissipation (= 2.23 N/cm^2)

$u = 4.47 \text{ N/cm}^2$; $\sigma' = 23.30 + 2.23 = 25.53 \text{ N/cm}^2$;

$\sigma = 30 \text{ N/cm}^2$ and $\Delta V/V_o = 3.23\%$ (from curve)

At the beginning of the 2nd stage of construction, (1b on Fig. 15.23)

$(\Delta V_{1b}/V_o) = 3.23\%$; $u_{1b} = 4.47 \text{ N/cm}^2$, $\sigma_{1b} = 25.53 \text{ N/cm}^2$ and $\sigma_{1b} = 30 \text{ N/cm}^2$

$$p_{ob} = p_o \ (= p_{atm}) + u = 10.3 + 4.47 = 14.77 \text{ N/cm}^2$$

$$S_{ob} = \frac{S_o}{1 + \left(\dfrac{p_o}{p_{ob}} - 1\right)(1 - S_o + S_o H)}$$

$$= \frac{0.80}{1 + \left(\dfrac{10.3}{14.47 \text{ N}} - 1\right)(1 - 0.8 + 0.8 \times 0.02)} = 0.853$$

$$n_{ob} = n_o + \frac{\Delta V_{1b}}{V_o} = 0.375 - 0.0323 \left(\text{as } \frac{\Delta V_{1b}}{V_o} \text{ is} - \text{ve}\right)$$

$$= 0.343$$

Using these values and Hilf's equation, one obtains

$$\Delta u_{12} = -\frac{\left(\dfrac{\Delta V_{12}}{V_o}\right) p_{ob}}{\left(\dfrac{\Delta V_{12}}{V_o}\right) + n_{ob}\,(1 - S_{ob} + S_{ob} H)}$$

$$= -\frac{14.77\left(\dfrac{\Delta V_{12}}{V_o}\right)}{\dfrac{\Delta V_{12}}{V_o} + 0.343\,(1 - 0.853 + 0.853 \times 0.02)}$$

$$= -\frac{14.77\left(\dfrac{\Delta V_{12}}{V_o}\right)}{\dfrac{\Delta V_{12}}{V_o} + 0.056}.$$

One can now compute Δu_{12} and prepare the following table :

$\frac{\Delta V}{V_o}$ (%)	$\frac{\Delta V_{12}}{V_o} = \frac{\Delta V}{V_o} - \frac{\Delta V_{1b}}{V_o} = \frac{\Delta V}{V_o} - 0.0323$	Δu_{12} (w.r.t. 1*b* as origin) (N/cm^2)	Pore pressure (w.r.t. initial pore pr.) $= u_{1b} + \Delta u_{12}$ (N/cm^2)	σ' (N/cm^2)	σ (N/cm^2)
3.23	0	0	4.47	25.53	30.00
4.0	0.0077	2.35	6.82	40.00	46.82
4.5	0.0127	4.33	8.80	54.50	63.30
4.65	0.0142	5.02	9.49	60.61	70.00

The values of Δu_{12} are shown plotted in Fig. 15.23 from 1*b* to 2*b*. Using this figure, a plot of u versus σ can be prepared, Fig. 15.24.

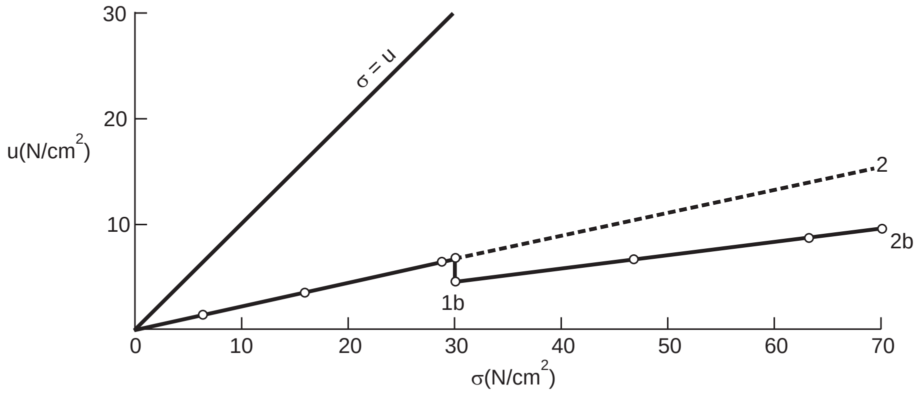


Fig. 15.24 Variation of u with σ

(ii) Partly-filled Reservoir Condition

Soon after the construction of an embankment dam, it takes a few seasons to fill the reservoir. This means that the dam will experience different amounts of pore pressures at different times depending upon the reservoir level. Since the upstream slope would not be fully covered with water during partly-filled reservoir condition, it needs to be examined for stability for various reservoir levels ranging from one-third to two-thirds the height of the full reservoir head, and the minimum factor of safety is worked out.

(iii) Sudden Drawdown Condition

This condition corresponds to lowering of reservoir level at a rate much faster than the rate of subsequent dissipation of pore pressure. The condition results in excess pore pressures and unbalanced seepage forces and may become worse when the materials of the upstream portion of the dam are not freely draining. If the coefficient of permeability of the shell material is less than 10^{-4} cm/s, full pore pressure should be considered (12). If the coefficient of permeability of the shell material is greater than 10^{-2} cm/s, pore pressure may not be considered in the analysis. For other values of the coefficient of permeability of the shell material, pore pressure values may be interpolated (12). These recommendations are based on a drawdown rate of 3 m/month.

The drawdown pore pressure in clay core (Fig. 15.25) can be determined using Bishop's formula (12):

$$U = \rho g\ [h_c + h_r\,(1 - n) - h] \tag{15.32}$$

Here, U is the drawdown pore pressure at any point within the core, ρ the mass density of water, g the acceleration due to gravity, h_c the height of core material at the point under consideration, h_r the height of the shell material at that point, h the drop in the head under steady seepage condition at the point, and n is the specific porosity of the shell material, *i.e.,* the volume of water draining out from the shell per unit volume.

The resisting and driving forces for impermeable material are calculated using the submerged and saturated weight, respectively, if the pore pressures are not otherwise included in the stability analysis (13).

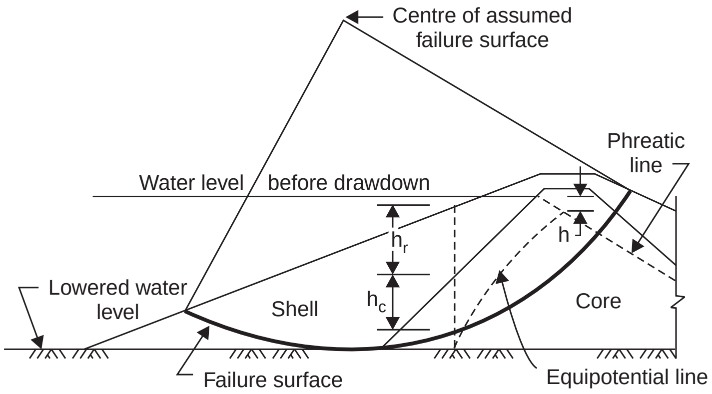


Fig. 15.25 Downstream pore pressure in clay core

(*iv*) *Steady Seepage at Full Reservoir Condition*

This condition develops when the reservoir is full and as such any shear slide would lead to a disastrous failure. Therefore, the stability analysis for this condition must be carried out more conservatively. The stability of the downstream slope should be analysed for this condition using the effective stress method and taking into account pore pressures owing to gravity flow. Pore pressures on account of changes in the embankment volume are not considered in the analysis, because the shear strains imposed on a well-constructed embankment are likely to dilate (*i.e.,* expand the volume) the soil and reduce the pore pressures temporarily (13).

The following unit weights may be used for the calculation of driving and resisting forces when pore pressures are otherwise not included in the stability analysis (13).

Location	*Driving force*	*Resisting force*
Below phreatic surface	Saturated weight	Submerged weight
Above phreatic surface	Moist weight	Moist weight

(*v*) *Steady Seepage with Sustained Rainfall*

If the downstream shell is relatively pervious, the pore pressures can increase appreciably by the penetration of rain water. In such situations, the gravity flownet should be constructed assuming that the dam crest and the downstream slope are sources of supply for seepage water (4) (Fig. 15.26). The downstream slope is analysed for stability assuming that partial saturation occurs due to rainfall. The saturation for the downstream shell is taken as 50 per cent (for coefficient of permeability of the shell less than 10^{-4} cm/s and 0 per cent for coefficient of permeability of the shell more than 10^{-2} cm/s) or a suitable value between 0 and 50 per cent depending upon the value of the coefficient of permeability of the shell material (12).

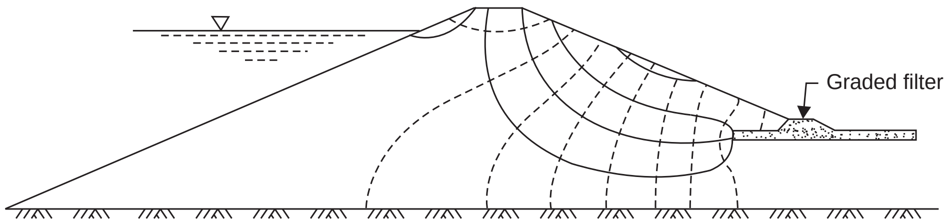


Fig. 15.26 Flownet for the downstream shell of an embankment dam during sustained rainfall

(vi) Earthquake Condition

The shaking of dam during an earthquake may cause most adverse conditions of stability of an embankment dam. For example, due to the shaking of the embankment, cracks within the core may widen causing increased leakage and consequent piping failure. This shaking may also result in the settlement of the crest due to the compression of the foundation and embankment. Acceleration forces acting on the embankment dam during an earthquake may cause shear slide of the slopes of the dam. Both the upstream and downstream slopes must be examined for earthquake effects if the dam is situated in an earthquake-prone region. Besides, other suitable measures to prevent earthquake damage are also adopted. Such measures include provision of graded filter downstream of the core, extra freeboard, very pervious downstream zones for disposing of the maximum anticipated leakage rapidly, flatter slopes near the top of the dam, better foundation treatment, and so on.

15.4.2. Shear Strength of Soils

Soils derive their strength from contact between particles capable of transmitting normal as well as shear forces. The contact between soil particles is mainly due to friction and the corresponding stress between the soil grains is called the effective (or intergranular) stress σ'. Thus, the shear strength of a soil is mainly governed by the effective stress. Besides the effective stress between soil grains, the pore water contained in the void spaces of the soil also exerts pressure which is known as pore pressure, u. The sum of the effective stress and pore pressure acting an any given surface within a compacted earth embankment is called the total stress σ. As the pore water cannot resist shear, all shear stresses are resisted by the soil grains only. The effective stress is approximately equal to the average intergranular force per unit area and cannot be measured directly (14). The total stress is equal to the total force per unit area acting normal to the plane. The pore water affects the physical interaction of soil particles. Soils with inactive surfaces do not absorb water. But, clay particles, formed of silicates which are, frequently, charged electrically, absorb water readily and exhibit plastic, shrinkage, and swelling characteristics. Besides, a change in pore pressure can directly affect the effective stress. The pore water thus influences the shear strength parameters of a soil to a considerable extent.

The shear strength of a soil is fully mobilised when a soil element can only just support the stresses exerted on it. For most soils, the shear strength of any surface, at failure, is approximated by the following Mohr-Coulomb's linear relationship [Fig. 15.27]:

$$s = c' + \sigma' \tan \phi' \tag{15.31}$$

where, s = shear strength of soil (or shear stress at failure) on the surface under consideration,

c' = cohesion,

ϕ' = angle of internal friction (or angle of shearing resistance), and

σ' = effective normal stress acting on the failure surface = $\sigma - u$.

Here, c' and ϕ' are determined using effective stresses.

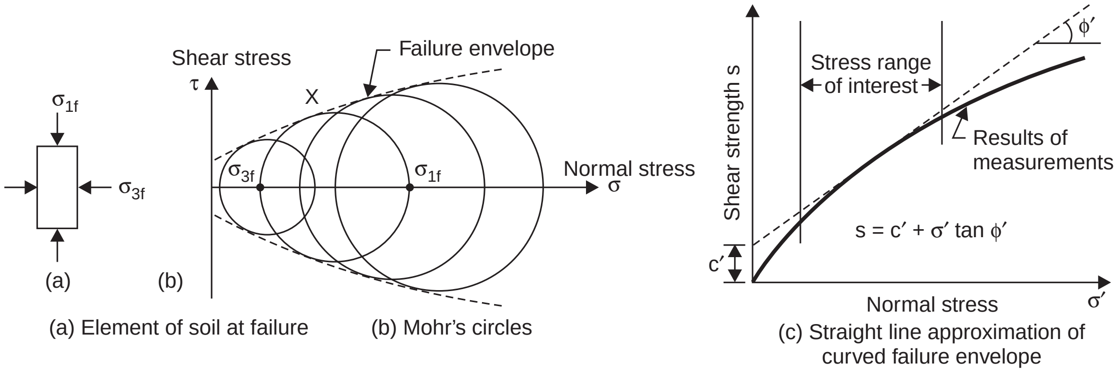


Fig. 15.27 Shear strength of soil

While analysing the stability of an embankment, the designer uses one of the two methods in vogue. These are: (*i*) total stress method, and (*ii*) effective stress method. Laboratory tests (for determining shear strength parameters of a soil) for the total stress method are performed such that the experimental conditions with regard to pore pressure simulate the pore pressure conditions in the embankment at failure and the shear strength of the soil sample is measured in terms of total stress without pore pressure measurements. For the effective stress method, pore pressures on the potential failure surface are estimated and the shear strength is determined in terms of the effective stresses using Eq. (15.31). The total stress method is relatively simpler. But, the effective stress method is more fundamental in nature and, hence, is recommended (12) for the stability analysis of embankment dams.

15.4.3. Laboratory Tests for Shear Strength of Compacted Impervious Soils

There are three types of laboratory tests which are commonly used for the determination of shear strength of compacted impervious soils. These methods differ in the method of consolidating the soil sample before the sample is failed in shear. These methods are as follows:

(*i*) Undrained test,

(*ii*) Consolidated-undrained test, and

(*iii*) Drained test.

In the undrained test (also known as the 'quick', 'unconsolidated-undrained' or '*Q*' test), drainage or dissipation of pore pressure is not allowed at any stage of the test. The relationship between the shear strength and normal pressure, obtained in terms of total stress, is used in the stability analysis of an embankment dam for 'during' and 'immediately after' construction condition. The soil samples are tested at a moisture content and density which would prevail during or immediately after construction. If the moisture content corresponds to saturation level, it will be found that all soil samples (saturated) have the same shear strength if no consolidation (*i.e.,* drainage) is allowed (14).

In consolidated-undrained test (also known as 'consolidated-quick' or '*R*' test) the sample is first allowed to consolidate (with full pore pressure dissipation) under a specified consolidation pressure, and is then failed in shear without permitting drainage. To obtain strength parameters in terms of effective stress (*i.e.,* c' and ϕ'), the pore pressure should be measured. These values are used for effective stress method of stability analysis. If the test values are to be used for the total stress method of stability analysis, the sample should be tested (without pore pressure measurement) at a water content which is anticipated in the dam during the period being analysed. It should often be difficult to determine the water content which would prevail in the dam. Hence, it is usual practice to conduct the tests on saturated samples for total stress method of analysis. The results of these tests would be useful for analysing sudden drawdown condition of impervious zones of embankment and foundation (14). The test results would also be useful for analysing the upstream slope during a partial pool condition and the downstream slope during steady seepage (12).

Drained test (also known as the 'slow' test or '*S*' test) permits drainage and complete dissipation of pore pressure at all stages of the test. The strength parameters are determined in terms of effective stress. The results of this test are to be used for freely-draining soils in which pore pressures do not develop (12).

15.4.4. Shear Strength of Pervious Soils

In embankment sections of clean sand and gravel, the pore pressures develop primarily due to seepage flow. The changes in pore pressure on account of changes in the embankment volume are short-lived and can be neglected. Consequently, the effective stress method of stability analysis is used for sections of pervious sand and gravel. The strength characteristics of sand and gravel are determined by drained tests. Because of the limitations of the size of test equipment, it may not be possible to test gravels of larger size. The strength characteristics of such soil can safely be assumed as those obtained for finer fractions of the same soil (4).

15.4.5. Factor of Safety

In most of the stability analysis methods for embankment dams, it is assumed that a slope might fail by a mass of soil sliding on a failure surface. At the moment of failure, the shear strength of the soil is fully mobilised on the entire failure surface and the overall slope as well as each of its parts are in the static equilibrium. For stable slopes, the shear strength mobilised under equilibrium conditions is less than the available shear strength and this is conventionally expressed in terms of a factor of safety, *F* defined as (14).

$$F = \frac{\text{shear strength available}}{\text{shear strength required for stability}} \tag{15.32}$$

The acceptable value of factor of safety for different conditions of stability of an embankment dam depends on method of analysis, site conditions, size of the dam, functions of the reservoir, and so forth.

15.4.6. Methods of Stability Analysis for Embankment Dams

There are several methods for analysing the stability of embankment dams. Of these, the limit equilibrium methods are the ones most commonly used. In these methods, a number of failure surfaces are analysed to determine their factors of safety. The minimum of these values is taken as the factor of safety for the slope under consideration. The failure surface corresponding to the minimum factor of safety, *F* is termed the "critical failure surface". Obviously, for stable slopes, the value of *F* should be greater than unity. The methods of stability analysis recommended by Bureau of Indian standards are (12): (*i*) the standard method of slices, and (*ii*) the wedge method.

15.4.6.1. Standard Method of Slices

This method (also known as the Swedish method of stability analysis) is the simplest method of stability analysis of embankment dams. It assumes that the forces acting on the sides of a slice do not affect the maximum shear strength which can develop on the bottom of the slice. This method of stability analysis was originally developed only for circular slip surfaces. However, it can be extended to non-circular slip surface also. The procedure for this method is as follows (4):

(*i*) The trial sliding mass (*i.e.,* the soil mass contained within the assumed failure surface) (Fig. 15.28) is divided into a number (usually 5 to 12) of slices which are usually, but not necessarily, of equal width. The width is so chosen that the chord and arc subtended at the bottom of the slice are not much different in length and that the failure surface subtended by each slice passes through material of one type of soil.

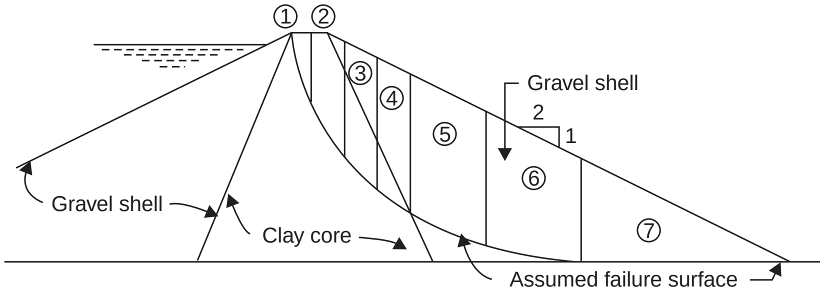


Fig. 15.28 Swedish method of stability analysis

(*ii*) For every slice of the trial sliding mass, the following forces are determined assuming the dam section to be of one unit length:

(*a*) The total weight (W) of the slice which is equal to the area of the slice multiplied by suitable gross unit weight of the soil mass.

(*b*) The force (U) due to pore pressure acting on the slice bottom is equal to the average unit pore pressure u, multiplied by the length of the bottom of the slice *i.e.,* $U = ub/\cos\alpha$ where, b is the width of the slice and α is the angle between the vertical and normal drawn at the centre of the bottom of the slice under consideration.

(*c*) The shear strength (C) due to cohesion for the slice under consideration is $c'\,b/\cos\alpha$ where, c' is the unit cohesion.

(*d*) The normal and tangential components of the total weight W are N ($= W\cos\alpha$) and T ($= W\sin\alpha$).

(*e*) The total shear force, *i.e.*, shear strength S which develops on the bottom of the slice at failure equals $C + (N - U)\tan\phi'$. Here, ϕ' = angle of internal friction in terms of the effective stress.

In addition, there are intergranular forces (E) and forces due to pore pressure (U) acting on both sides of any given slice. While the magnitude and direction of forces due to pore pressures can be estimated, the intergranular forces are not known. To make computational procedure simple, these forces (E_L and U_L) acting on one side of a given slice are assumed to be equal (in magnitude) and opposite (in direction) to the forces acting on the other side of the slice (*i.e.,* E_R and U_R). It should, however, be noted that $\Sigma (E_L - E_R)$ and $\Sigma (U_L - U_R)$ for the entire sliding mass are not zero.

(*iii*) The results of these computations are tabulated and the sums of the forces S and T are determined.

(*iv*) The factor of safety is computed from the relation

$$F = \frac{\Sigma S}{\Sigma T} = \frac{\Sigma\,[C + (N - U) \tan \phi']}{\Sigma T} \tag{15.33}$$

By this method of analysis, one obtains a conservative value of the factor of safety. This is due to complete neglect of the intergranular forces and pore pressures acting on the sides of slices in the computations.

Alternatively the factor of safety for the chosen slip surface is computed using Taylor's "Modified Swedish Method". This method assumes that: (*i*) the directions of the intergranular forces acting on the sides of the slices are parallel to the average exterior slope of the embankment, and (*ii*) an equal proportion of the shear strength available is developed on the bottom of all the slices (4). The computational steps for Taylor's method applied to failure surface of any arbitrary shape (Fig. 15.28) are as follows (4):

(*i*) The trial sliding mass is divided into a suitable number of slice so that their chord length and arc length (subtended at the bottom of the slice) do not differ much and the entire bottom of a slice is within one type of soil material.

(*ii*) For each slice [Fig. 15.29 (*a*)] following forces are computed:

(*a*) The total weight W,

(*b*) The forces due to pore pressure acting on the bottom and sides of the slices, *i.e.,* U_L, U_R, and U_B, and

(*c*) The cohesion force C acting on the bottom of the slice.

(*iii*) For each slice, the known forces W, U_L, U_R and U_B are resolved into a resultant R [Fig. 15.29 (*b*)].

(*iv*) The direction of the intergranular forces acting between the slices is assumed parallel to the average exterior slope of the embankment.

(*v*) Assume a suitable value of factor of safety, say F_D, on the basis of stability analysis carried out by some approximate method, or otherwise, and determine

$$C_D = \frac{C}{F_D}$$

(*vi*) Draw composite force polygon [Fig. 15.29 (*c*)] for the whole trial sliding mass including all the forces acting on the individual slices.

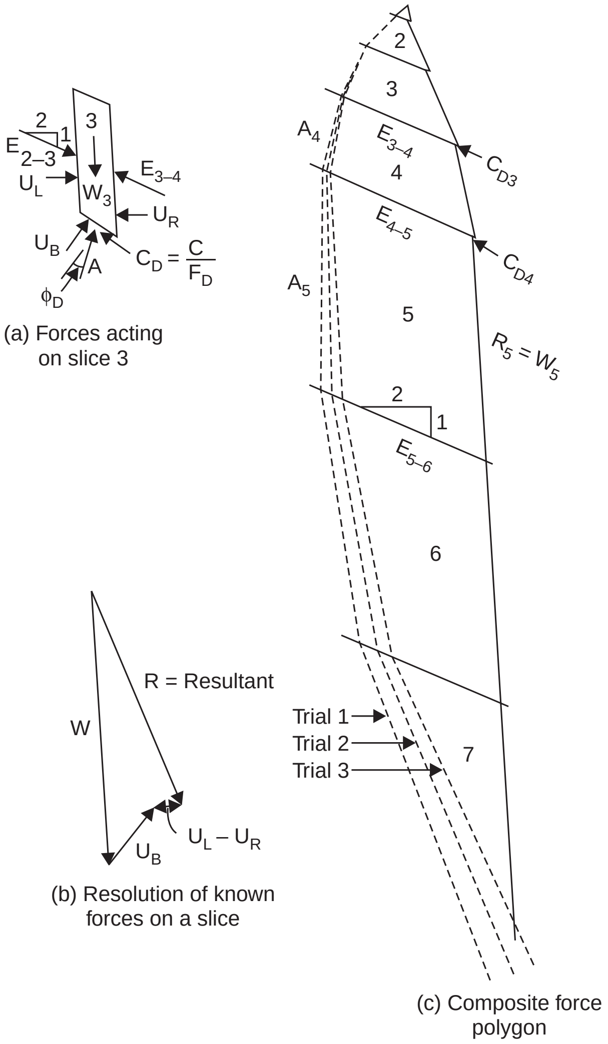


Fig. 15.29 Modified Swedish method of stability analysis

If the force polygon does not close, choose another value of F_D and compute C_D and redraw the composite force polygon. This is continued until one obtains the safety factor which closes the force polygon. This method can be similarly applied to two-wedge as well as three-wedge systems (Fig. 15.30). The modified Swedish method should be used for final stability analysis in all major embankment dams.

15.4.6.2. Wedge (or Sliding Block) Method

This method is used when the slip surface can be approximated by two or three straight lines. Such a situation arises when the slope is underlain by a strong stratum such as rock or there is a weak layer included within or beneath the slope. In such circumstances, an accurate stability analysis can be carried out by dividing the trial sliding mass into two or three blocks of soil and examining the equilibrium of each block. The upper block (or wedge) is called the driving (or active) block and the lower block is called the resisting (or passive) block. In a three-wedge system, the central block is called the sliding block (Fig. 15.30).

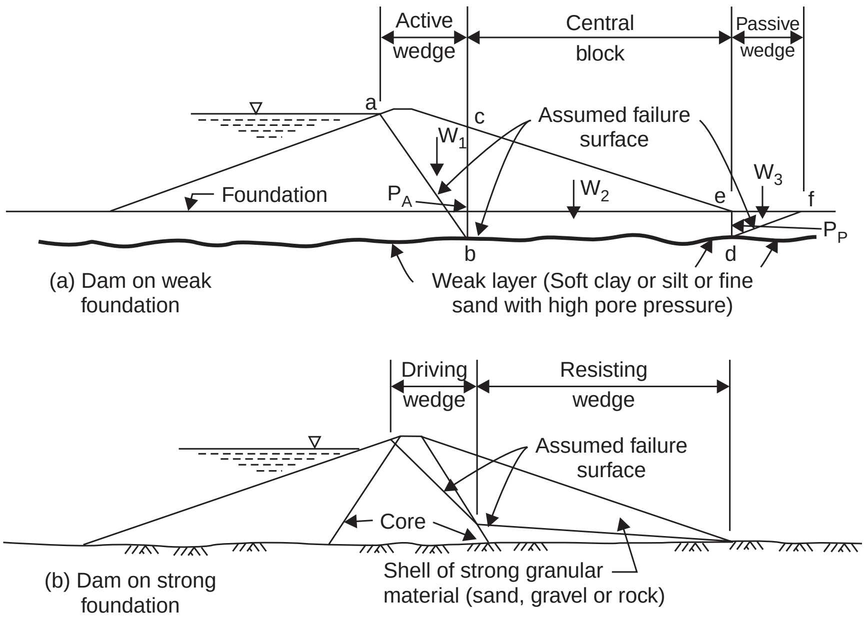


Fig. 15.30 Conditions for applicability of wedge analysis

The factor of safety can be estimated by any of the methods discussed earlier. Alternatively, assuming that the active and passive wedges are at failure and that the total forces on the vertical planes (*bc* and *de*) are horizontal, one can estimate the factor of safety as the ratio of the force P_1 available on *bd* to resist the movement of the central block and the unbalanced force, $P_A - P_P$. This means,

$$F = \frac{C_{bd} + (W_2 - U_{bd}) \tan \phi'_{bd}}{P_A - P_P} \tag{15.34}$$

where, the subscript *bd* is used for the plane *bd* and the subscript 2 is for the central block.

Example 15.5 Determine the factor of safety for the slip surface shown in Fig. 15.31 for sudden drawdown condition with the following properties of the embankment material:

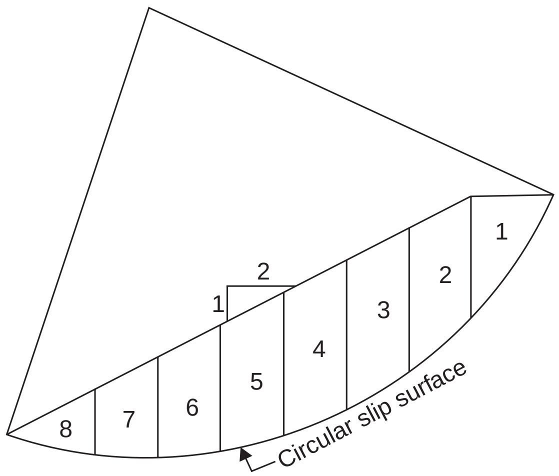


Fig. 15.31 Slip surface for Example 15.5

Saturated weight = 21.0 kN/m^3

Submerged weight = 11.0 kN/m^3

Cohesion = 24.5 kN/m^2

Angle of internal friction, ϕ' = 35°

Angle α, arc length, and area of different slices are given in the first four columns of Table 15.3.

Solution: Since pore pressures are not known, the driving force (T-component) and the resisting force (N-component) are calculated using saturated and submerged weights, respectively, and are shown in Table 15.3.

Table 15.3 Data and solution for Example 15.4

Slice	*α (degrees)*	*Arc length (metres)*	*Area of slice (m^2)*	*T-Component*		*N-Component*	
				Weight, W (kN/m)	*W sin α (kN/m)*	*Weight, W (kN/m)*	*W cos α (kN/m)*
1	54.5	6.70	12.26	257.46	209.60	134.86	78.31
2	41	3.80	19.51	409.71	268.79	214.61	161.97
3	31	3.50	21.37	448.77	231.13	235.07	201.49
4	22	3.35	20.90	438.90	164.42	229.90	213.16
5	13	3.05	19.97	419.37	94.34	219.67	214.04
6	5	3.05	16.72	351.12	30.60	183.92	183.22
7	– 3.5	3.05	12.08	253.68	– 15.49	132.88	132.63
8	– 13	4.30	6.69	140.49	– 31.60	73.59	71.70
		30.80			951.79		1256.52

Using Eq. (15.33)

$$\text{Factor of Safety} = \frac{30.80 \times 24.5 + 1256.52 \tan 35°}{951.79} = 1.72$$

15.4.7. Seismic Considerations in Stability Analysis

Seismic forces reduce the margin of safety of an embankment dam. Therefore, when an embankment dam is located in a seismic region, the stability analysis must also consider earthquake forces. During an earthquake, the ground surface oscillates randomly in different directions. This motion can be represented by horizontal and vertical components. A rigid structure is expected to follow the oscillations of its base in the absence of relative deformation from the base of the structure to its top. The amplitude of the oscillations and the acceleration vary along the height of the structure. An earth dam should be treated as a flexible structure for determining dynamic pressure due to earthquake. However, a simple method to account for earthquake forces in the design of structures is based on seismic coefficients. In this method, basic seismic coefficients or earthquake acceleration coefficients are used. The seismic coefficient is defined as the ratio of earthquake acceleration in a particular direction to the gravitational acceleration. If α_h is the horizontal earthquake acceleration coefficient then the additional inertial force of the soil mass (of the slice under consideration) is taken as $\alpha_h W$ in the horizontal direction. Obviously, a force equal to $\alpha_h W \cos \alpha$ is added to the tangential forces and $\alpha_h W \sin \alpha$

is deducted from the forces acting in the normal direction. The factor of safety, therefore, becomes

$$F = \frac{\Sigma C + \Sigma\,(N - U - \alpha_h W \sin\alpha)\tan\phi'}{\Sigma\,(W\sin\alpha + \alpha_h W\cos\alpha)} \qquad (15.35)$$

This simple way of accounting the seismic effects in the stability analysis is based on the pseudostatic concept in which the dynamic effects of an earthquake are replaced by a static force, and in which limit equilibrium is maintained (6).

15.4.8. Stability of Foundation

The factor of safety against shear for the foundation material is the ratio of the shear strength to shear stress at a location in the foundation where maximum intensity of shear stress occurs. Since the factor of safety corresponds to the maximum shear stress, its acceptable value should only be slightly greater than unity for the foundation of an embankment dam. The magnitude and location of the maximum shear stress can be determined approximately as follows:

The horizontal shear, S under a slope of the dam is equal to the difference between the lateral thrust on a vertical through the upper end of the slope and a vertical through the toe of the slope. Thus, with reference to Fig. 15.32,

$$S = \rho g\,\frac{h_1^2 - h_2^2}{2}\tan^2\left(45 - \frac{\phi_1}{2}\right) \qquad (15.36)$$

where, ρ = average mass density of the soil,

g = acceleration due to gravity,

ϕ_1 = equivalent angle of internal friction, determined by the equation $\rho g h_1 \tan\phi_1 = c + \rho g h_1 \tan\phi$ where, c and ϕ are actual properties of the foundation soil, and h_1 and h_2 are heights, as shown in Fig. 15.32, of the upper and lower ends of the slope, respectively, above a stratum which is much stronger than the overlying foundation material.

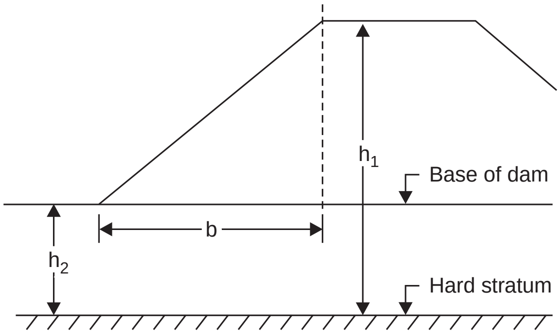


Fig. 15.32 Shear stress in foundation of embankment

The average shear stress s_a is equal to S/b. Here, b is the horizontal length of the slope. Results of photoelastic investigations indicate that the maximum intenstiy of shear equals 1.4 times the average shear s_a, and occurs at a distance of 0.4 b from the upper end of the slope. One can, alternatively, compare average values of shear stress and shear strength and accept a relatively higher factor of safety for the stability of the foundation of an embankment.

15.5. SLOPE PROTECTION

If the upstream slope of an earth dam, retaining a large reservoir, is composed of materials other than cobbles or rocks, it must be protected against damage by wave action. Based on past experience, the methods of upstream slope protection are: (*i*) dumped rock riprap, (*ii*) hand-placed stone pitching, (*iii*) monolithic RCC slab, and (*iv*) asphaltic concrete. Of these four methods, the dumped rock riprap overlying a finer filter layer (or layers) provides an excellent wave protection measure as it is least damaged by post-construction embankment settlement and is an effective dissipator of wave energy. A riprap layer should be designed such that: (*i*) the individual rocks are not moved out of place by the wave forces, and (*ii*) the filter underlying the riprap will not be washed out through the voids in the riprap layer. The filter itself should be able to prevent erosion of the underlying embankment material. The thickness and size of dumped rock riprap, as recommended by Bertram (15), are given in Table 15.4. Table 15.4 also gives the minimum thickness of underlying filter layer as recommended by the US Army Corps of Engineers.

Table 15.4 Thickness and size of dumped rock riprap

Maximum wave height (m)	*Minimum average rock size D_{50} (cm)*	*Layer thickness (cm)*	*Minimum filter layer thickness (cm)*
0-0.6	25	30	15
0.6-1.2	30	45	15
1.2-1.8	38	60	22.5
1.8-2.4	45	75	22.5
2.4-3.0	52	90	30

If the surface of the downstream slope of an earth dam consists of fine-grained soil, considerable erosion may be caused by either surface water runoff during rainstorms or wind during windstorms in arid regions. The erosion of the surface results in deep gullies (as deep as 3 m in severe cases) on the downstream slope at the abutment contacts as well as in the central portion of the dam.

A good cover of grass on the surface of the downstream slope holds the surface soil in place and provides most satisfactory and economical slope protection. However, in very dry areas, it may not be possible to spare enough water to grow and maintain the grass cover. In such situations, dumped rock riprap (used for upstream slope protection) can be used for the downstream slope as well.

15.6. INSTRUMENTATION

Instruments are installed in an embankment dam to measure pore pressures at different locations, and also the settlement and horizontal meovement of the dam. The measurements from these instruments enable the concerned authorities to compare the measured quantities with the corresponding values used by the designer. These measurements also provide a reliable basis for analysing the performance of the dam and for taking suitable steps to overcome a problem which may develop. Besides, these instruments provide valuable data for use in future designs of dams. The instruments installed in an embankment dam are piezometeres (for measuring pore water pressure), and instruments for measuring horizontal movements, foundation settlement, and embankment compressions. Instruments of the second type can be either internal instruments which are installed within the embankment during construction, or surface monuments consisting of concrete-embedded steel rods installed accurarterly along straight lines. The installed instruments should be simple in design with minimum moving parts, and should require no maintenance as most of the instruments would be embedded. The installation procedure should cause least interference in construction activities. The method of observation should also be simple.

15.7. EMBANKMENT CONSTRUCTION

The construction of an embankment mainly consists of the following activities:

(*i*) Excavation of the material,

(*ii*) Hauling the excavated material to the site of the dam,

(*iii*) Mixing the material (either in the borrow pit or the embankment surface) with water to obtain the uniformity of desired water content and other properties,

(*iv*) Spreading the material in layers on the embankment surface, and

(*v*) Compacting the spread material to the desired density.

Over the last fifty years there has been considerable development in larger and faster earth-handling equipment. The rate of embankment construction depends primarily on the amount and types of equipment used and can be of the order of 2000 m^3/day. The types of equipment generally used for the construction of embankment dams are as follows:

(*i*) Excavating equipment, *viz.*, power shovels, drag lines, scrapers, *etc.*,

(*ii*) Hauling equipment, *viz.*, scrapers, truck, belt conveyors, *etc.*,

(*iii*) Spreading equipment, *viz.*, bullozoers, graders, *etc.*,

(*iv*) Compacting equipment, *viz.*, sheepsfoot and rubber-tyred rollers, *etc.*, and

(*vi*) Watering equipment, *viz.*, water trucks, hose, *etc.*

Depending upon the site conditions, some other types of equipment for specific purposes may also be needed.

EXERCISES

15.1 Discuss the factors which influence the design of an embankment dam.

15.2 What are the common causes of failure and corresponding safety measures adotped in an embankment dam?

15.3 Describe different methods of controlling seepage through an embankment dam and its foundation.

15.4 For the earth dam of homogeneous section with a horizontal drain as shown in Fig. 15.33, draw the top flow line and the flownet. Also estimate the discharge per metre length through the body of the dam ($K = 5 \times 10^{-4}$ cm/s).

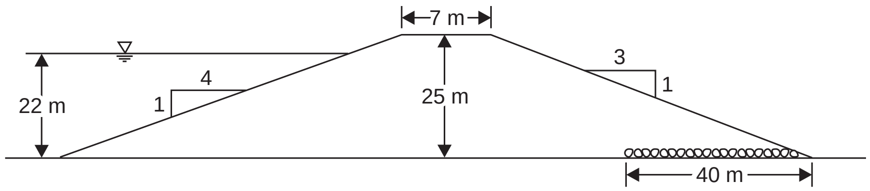


Fig. 15.33 Sketch for Exercises 15.4 and 15.5

15.5 For the embankment section shown in Fig. 15.33, if $K_x = 9 \times 10^{-4}$ cm/s and $K_y = 6 \times 10^{-4}$ cm/s, estimate the discharge through the dam section per metre length.

15.6 Determine the length of blanket to reduce the foundation seepage by 40 per cent for the dam section shown in Fig. 15.34.

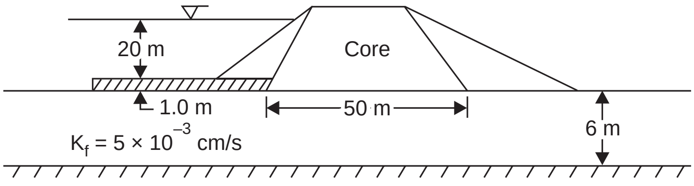


Fig. 15.34 Sketch for Exercise 15.6

15.7 Find the factor of safety against sudden drawdown for the failure surface shown in Fig. 15.35 with the following data:

Angle of internal friction	:	For shell = 32°, for core = 20°
Cohesion	:	For shell = 0, for core = 40 kN/m^2
Saturated weight	:	For core material = 20 kN/m^3
		For shell material = 21 kN/m^3 (not freely draining)

Assume water up to the top of dam under full reservoir condition.

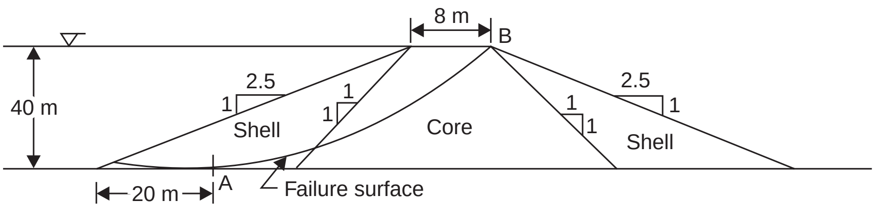


Fig. 15.35 Sketch for Exercise 15.7

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16

GRAVITY DAMS

16.1. GENERAL

A *gravity dam* is a solid concrete or masonry structure which ensures stability against all applied loads by its weight alone without depending on arch or beam action. Such dams are usually straight in plan and approximately triangular in cross-section. Gravity dams are usually classified with reference to their structural height which is the difference in elevation between the top of the dam (*i.e.*, the crown of the roadway, or the level of the walkway if there is no roadway) and the lowest point in the excavated foundation area, exclusive of such features as narrow fault zones (1). Gravity dams up to 100 ft (30.48 m) in height are generally considered as low dams. Dams of height between 100 ft (30.48 m) and 300 ft (91.44m) are designated as medium-height dams. Dams higher than 300 ft (91.44 m) are considered as high dams.

The downstream face of a gravity dam usually has a uniform slope which, if extended, would intersect the vertical upstream face at or near the maximum water level in the reservoir. The upper portion of the dam is made thick enough to accommodate the roadway or other required access as well as to resist the shock of floating objects in the reservoir. The upstream face of a gravity dam is usually kept vertical so that most of its weight is concentrated near the upstream face to resist effectively the tensile stresses due to the reservoir water loading. The thickness of the dam provides resistance to sliding and may, therefore, dictate the slope of the downstream face which is usually in the range of 0.7 to 0.8 (H) : 1(V). The thickness in the lower part of the dam may also be increased by an upstream batter.

When it is not feasible to locate the spillway in the abutment, it may be located on a portion of the dam in which case the section of the dam is modified at the top to accommodate the crest of the spillway and at the toe to accommodate the energy dissipator. The stability requirements of such overflow sections of gravity dams would be different from those of non-overflow gravity dams.

16.2. FORCES ON A GRAVITY DAM

The forces commonly included in the design of a gravity dam are shown in Fig. 16.1. These are as follows (2, 3, 4):

(i) Dead Load

The dead load (W_c) includes the weight of concrete and the weight of appurtenances such as piers, gates, and bridges. All the dead load is assumed to be transmitted vertically to the foundation without transfer by shear between adjacent blocks.

(ii) Reservoir and Tail-water Loads (W_w, W_w', W_1, and W_1')

These are obtained from tail-water curves and range of water surface elevations in reservoir obtained from reservoir operation studies. These studies are based on operating and hydrologic data such as reservoir capacity, storage allocations, stream flow records, flood hydrographs, and reservoir releases for all purposes. In case of low overflow dams, the dynamic effect of the velocity of approach may be significant and should, therefore, be considered. If gates or other control features are used on the crest, they are treated as part of the dam so far as the application of water pressure is concerned. In case of non-overflow gravity dams, the tail-water should be adjusted for any retrogression. Any increase in tail-water pressure due to curvature of flow in the downstream bucket of an overflow type gravity dam should also be considered in the design of gravity dams (4).

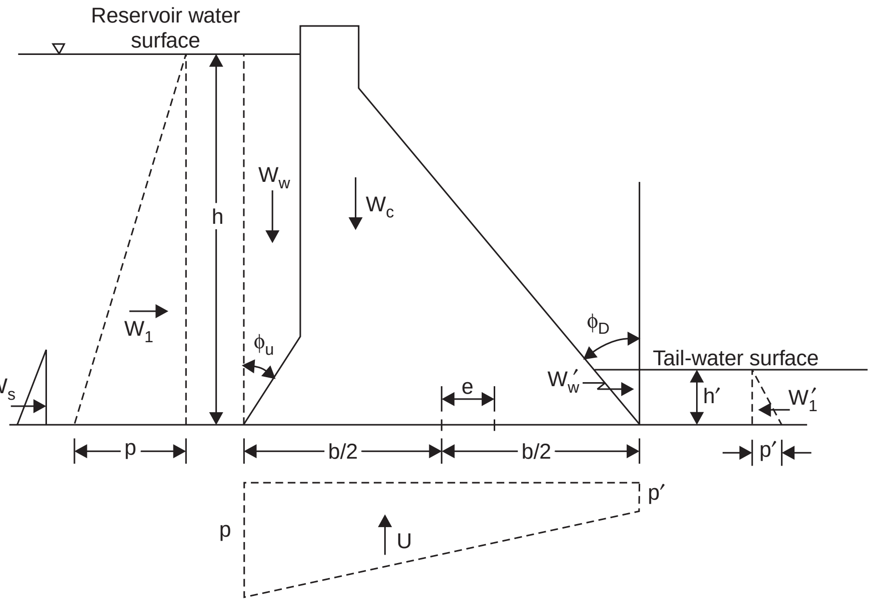


Fig. 16.1 Usual loading combination for a gravity dam

(*iii*) *Uplift Forces*

Uplift forces (U) occur due to internal hydraulic pressures in pores, cracks, and seams within the body of a dam, at the contact between the dam and its foundation, and within the foundation. The distribution of internal hydrostatic pressure along a horizontal section through a gravity dam is assumed to vary linearly from full reservoir pressure at the upstream face to zero or tail-water pressure at the downstream face, and to act over the entire area of the section. The pressure distribution is also adjusted depending upon the size, location, and spacing of internal drains. Experimental and analytical studies indicate that the drains set in from the upstream face at 5 per cent of the maximum reservoir depth and spaced laterally twice that distance will reduce the average pressure at the drains to approximately tail-water pressure plus one-third the difference between reservoir water and tail-water pressures (3) (Fig. 16.2). It is assumed that uplift forces are not affected by earthquakes (2).

(*iv*) *Silt Load*

The construction of a dam across a river carrying sediment invariably results in reservoir sedimentation which causes an additional force (W_s) on the upstream face of the dam. The horizontal silt pressure is assumed equivalent to a hydrostatic load exerted by a fluid with a mass density of 1360 kg/m^3. The vertical silt pressure is assumed equivalent to that exerted by a soil with a wet density of 1925 kg/m^3.

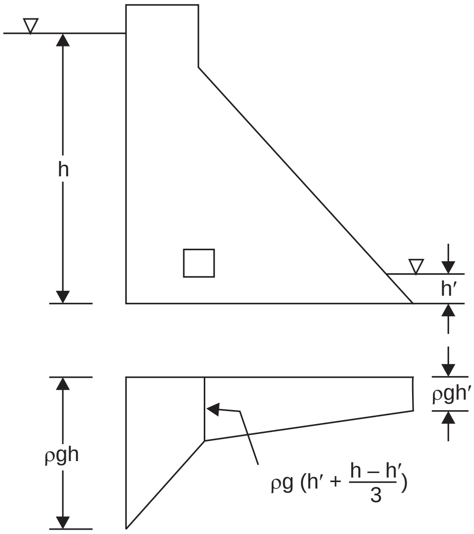


Fig. 16.2 Modification in uplift force due to drain

(*v*) *Ice Pressure*

If the designer anticipates the formation of an ice sheet of appreciable thickness and its remaining on the reservoir water surface for a long duration, the ice pressures must be computed using a suitable method of their estimation. In the absence of such a method, ice pressure may be taken as 250 kPa (250 kN/m^2) applied over the anticipated area of contact of ice with the face of the dam (2).

(*vi*) *Wave Pressure*

The upper portion of a dam is also subjected to the impact of waves. Wave pressure against massive dams of large height is usually of little importance. Wave pressure is related to wave height h_w as follows (2):

(*a*) The maximum wave pressure p_w (in kilopascals) occurs at 0.125 h_w above the still water level and is given by the equation

$$p_w = 24\, h_w \tag{16.1}$$

where, h_w is the height of the wave in metres.

(*b*) The total wave force P_w (in kilonewtons) is given by

$$P_w = 20\, h_w^2 \tag{16.2}$$

and acts at 0.375 h_w above the still water level in the downstream direction.

(*c*) The wave height h_w can be calculated using the following relations:

$$h_w = 0.032\sqrt{VF} + 0.76 - 0.27\, F^{1/4} \text{ for } F < 32 \text{ km} \tag{16.3}$$

$$h_w = 0.032\sqrt{VF} \text{ for } F > 32 \text{ km} \tag{16.4}$$

Here, V is the wind velocity in kilometres per hour and F is the fetch in kilometres.

The height of the wave and the wind set-up decide the freeboard which is the vertical distance between the top of the dam and the still water level. The wind set-up S (in metres) is estimated by the Zuider Zee formula

$$S = \frac{V^2 F}{62{,}000\, D} \tag{16.5}$$

in which, D is the average depth in metres over the fetch distance F.

The minimum freeboard should be equal to wind set-up plus $\frac{4}{3}$ times wave height above the normal pool elevation or above maximum reservoir level corresponding to the design flood, whichever gives higher crest elevation for the dam (2). The freeboard shall not, however, be less than 1.0 m above the mean water level corresponding to the design flood.

(vii) Earthquake

Gravity dams are elastic structures which may be excited to resonate by seismic disturbances. Such dams should be designed so that they remain elastic when subjected to the design earthquake. The design earthquake should be determined considering (*i*) historical records of earthquakes to obtain frequency of occurrence versus magnitude, (*ii*) useful life of the dam, and (*iii*) statistical approach to determine probable occurrence of earthquakes of various magnitudes during the life of the dam. A gravity dam should also be designed to withstand the maximum credible earthquake which is defined as the one having a magnitude usually larger than any historical recorded earthquake (3).

Earthquakes impart random oscillations to the dam which increase the water and silt pressures acting on the dam and also the stresses within the dam. An earthquake movement may take place in any direction. Both horizontal and vertical eathquake loads should be applied in the direction which produces the most unfavourable conditions. For a gravity dam, when the reservoir is full, the most unfavourable direction of earthquake movement is upstream (so that the inertial forces acting downstream may result in resultant force intersecting the base of the dam outside middle-third of the base besides increasing the water load and, therefore, the increased overturning moment) and is downward for vertical earthquake movement as it causes the concrete, and water above the sloping faces of the dam to weigh less resulting in reduced stability of the dam. When the reservoir is empty, more unfavourable is the downstream ground motion causing inertial forces to act upstream so that the resultant may intersect the base of the dam outside middle-third of the base. The effect of earthquake forces depends on (*i*) their magnitude which, in turn, depends on the severity of the earthquake, (*ii*) the mass of the structure and its elasticity, and (*iii*) the earthquake effects on the water load. For estimation of earthquake load, knowledge of earthquake acceleration or intensity, usually expressed in relation to acceleration due to gravity g, is useful. This ratio of earthquake acceleration to gravitational acceleration is termed seismic coefficient and is designated as α_h. The value of seismic coefficient for horizontal as well as vertical earthquake accelerations for different zones of the country are different and can be obtained from the Codes. Considering a structure of mass M moving with an acceleration $\alpha_h\, g$ in the horizontal direction during an earthquake, the horizontal earthquake force acting on the structure, P_e is given as

$$P_e = M\, \alpha_h\, g = \frac{W}{g} \alpha_h g = \alpha_h W$$

where, W is the weight of the structure. The value of α_h has usually been taken as 0.1 in the absence of any other specified value. Similarly, the value of the seismic coefficient in the vertical direction can be taken as 0.05.

The inertia of water in the reservoir also produces a force on the face of the dam during an earthquake. For dams with vertical or sloping upstream face, the variation of horizontal hydrodynamic earthquake pressure with depth is given by the following equations (5):

$$p_e = c_1 \,\alpha_h \,\rho\, gh \tag{16.6}$$

and

$$c_1 = \frac{c_m}{2}\left[\frac{y}{h}\left(2-\frac{y}{h}\right)+\sqrt{\frac{y}{h}\left(2-\frac{y}{h}\right)}\right] \tag{16.7}$$

where, p_e = hydrodynamic earthquake pressure normal to the face,

c_1 = a dimensionless pressure coefficient,

α_h = ratio of horizontal acceleration due to earthquake and the gravitational acceleration, *i.e.,* horizontal acceleration factor,

ρ = mass density of water,

g = acceleration due to gravity.

h = depth of reservoir,

y = vertical distance from the reservoir surface to the elevation under consideration,

and c_m = the maximum value of c_1 for a given slope (Fig. 16.3).

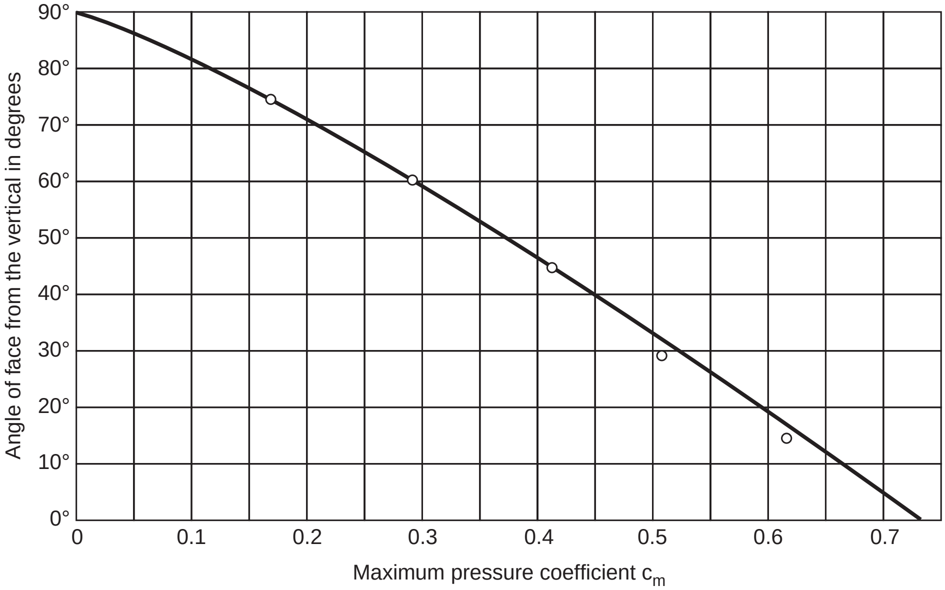


Fig. 16.3 Variation of c_m with inclination of the upstream face

Let V_{pe} (or V_{pe}') represent change in horizontal component of reservoir (or tail-water) load on the face above a section due to horizontal earthquake loads and it is computed for each increment of elevation selected for the study and the totals obtained by summation because of the nonlinear response (1). Likewise, M_{pe} (or M_{pe}') which represents the moment of V_{pe} (or V_{pe}') about the centre of gravity of the section, is computed. The inertia forces for concrete in the dam should be computed for each increment of height, using the average acceleration factor for that increment. The inertia forces to be used while considering an elevation in the dam are the summation of all the incremental forces above that elevation and the total of their moments about the centre of gravity at the elevation being considered. The horizontal concrete inertia force (V_e) and its moment (M_e) can be calculated using Simpson's rule (1). Alternatively, V_{pe} and M_{pe} can be obtained from the following equations (4):

$$V_{pe} = 0.726\, p_e\, y$$

and
$$M_{pe} = 0.299\, p_e\, y^2$$

The effects of vertical accelerations may be determined using the appropriate forces, moments, and the vertical acceleration factor α_v. The forces and moments due to water pressure normal to the faces of the dam and those due to the dead loads should be multiplied by the appropriate acceleration factors to determine the increase (or decrease) caused by the vertical downward (or upward) accelerations. The effect of earthquake on uplift forces is considered negligible.

Dams having upstream face as a combination of vertical and sloping faces are analysed as follows (4):

(*a*) If the height of the vertical portion of the upstream face of a dam is equal to or greater than one-half the total height of the dam, analyse the dam as if it has a vertical upstream face throughout.

(*b*) If the height of the vertical portion of the upstream face of a dam is less than half the total height of the dam, use the pressure which would occur if the upstream face has a constant slope (equal to the slope of the sloping portion of the upstream face) from the water surface elevation to the heel of the dam.

(viii) Other Miscellaneous Loads

In addition to the above-mentioned forces there may be thermal loads and vertical water loading too. If the contraction joints are grouted, the horizontal thrusts, caused by volumetric increases due to rising temperature, will produce load transfer across joints. This load transfer increases the twist effects and the loads at the abutments (3). Vertical water loading is exerted by the weight of the water on sloping upstream and downstream faces of the dam. The vertical component of the water flowing over the spillway is not included in the analysis as water tends to attain the spouting velocity which reduces pressure on the dam. Any negative pressure which may develop on the spillway crest is also neglected. However, any sub-atmospheric pressure developing on the downstream sloping surface of the spillway due to lack of aeration should be considered by treating them as positive load (acting in the downstream direction) applied on the upstream face.

The design of gravity dams must consider most adverse combination of probable load conditions (1). Combinations of loads whose simultaneous occurrence is highly improbable may, however, be excluded. Most load combinations can be categorised as usual, unusual, or extreme. For example, normal design reservoir elevation with appropriate dead loads, uplift, silt, ice, tail-water, and thermal loads corresponding to usual temperature make a typical usual type load combination. Unusual load combination considers maximum design reservoir elevation with the loads of usual type load combination. Extreme load combination results when the effects of maximum credible earthquake are included in the usual load combination.

16.3. CAUSES OF FAILURE OF A GRAVITY DAM

A gravity dam may fail on account of overturning. For a gravity dam to be safe against overturning, the dimensions of the dam should be such that the resultant of all the forces intersects the base of the dam within its middle-third portion. Consider any horizontal section (including the base) of a gravity dam and the resultant of all the forces acting on the dam above the section. If the line of action of this resultant passes outside the downstream edge of the section, the dam would overturn. However, the section of a gravity dam is such that the line of action of the resultant force is within the upstream and downstream edges of the section and overturning would never results. But, if the line of action of the resultant passes sufficiently outside the middle-third of the horizontal section, it may cause crushing of the downstream edge of the section. This would reduce the effective width and, hence, the sliding resistance of the section and may cause the resultant to pass outside the dam section. Further, when the resultant passes downstream of the middle-third of the horizontal section, it induces tensile stresses at the upstream edge of the section. These tensile stresses may cause cracks in the dam section which would result in increased uplift pressure. The stabilising forces would, thus, be reduced. It follows, therefore, that before a gravity dam overturns bodily, other types of failures, such as crushing of toe material, sliding, cracking of the material due to tension, and increase in uplift may occur. A gravity dam is considered safe against overturning if the criteria of: (*i*) no tension on the upstream face, (*ii*) adequate resistance against sliding, and (*iii*) suitable quality and sufficient strength of concrete/masonry of dam and its foundation are satisfied.

Concrete and masonry are relatively weak in tension and as such the design of a gravity dam should ensure that there are no tensile stresses anywhere in the dam section. In very high gravity dams, however, if it becomes difficult to ensure such a condition, one may allow small tensile stresses not exceeding 50 N/cm^2 under the most adverse condition of loading.

The horizontal forces acting on a dam above any horizontal plane may cause failure of the dam due to sliding if these driving forces are more than the resistance to sliding on the plane. The resistance to sliding is due to the frictional resistance and shearing strength of the material along the plane under consideration. The shear-friction factor of safety, F_s, which is a measure of stability against sliding or shearing, can be expressed as follows:

$$F_s = \frac{CA + \Sigma W\mu}{\Sigma H} \tag{16.8}$$

where, C = unit cohesion,

A = area of the plane considered (A can be replaced by the width of the plane, if one considers unit length of the dam),

ΣW = sum of all vertical forces acting on the plane,

μ = coefficient of internal friction, and

ΣH = sum of driving shear forces *i.e.*, resultant horizontal forces.

The shear-friction factor of safety can be used to determine the stability against sliding or shearing at any horizontal section within a dam, its contact with the foundation or through the foundation along any plane of weakness. The minimum allowable values of F_s for gravity dam are 3.0, 2.0, and 1.0 for the usual, unusual, and extreme loading combinations, respectively (3). The value of F_s for any plane of weakness within the foundation should not be less than 4.0, 2.7, and 1.3 for the usual, unusual, and extreme loading combinations, respectively (3).

Generally, the acceptable factor of safety against overturning and shear for normal or usual loading condition is taken as 2.0. The corresponding value for extreme loading condition is 1.25. The acceptable value of the sliding factor (= the ratio of the sum of the horizontal forces and the sum of the vertical forces) is the ratio of the coefficient of static friction and the chosen factor of safety.

The maximum allowable compressive stress for concrete in a gravity dam should be less than the specified compressive strength of the concrete divided by 3.0, 2.0, and 1.0 for usual, unusual, and extreme load combinations, respectively. The compressive stress should not exceed 1035 N/cm^2 and 1550 N/cm^2 for usual and unusual load combinations, respectively (3).

The maximum allowable compressive stress in the foundation should be less than the compressive strength of the foundation divided by 4.0, 2.7, and 1.3 for usual, unusual, and extreme load combinations, respectively. These values of factor of safety are higher than those for concrete so as to provide for uncertainties in estimating the foundation properties.

16.4. STRESS ANALYSIS OF GRAVITY DAMS

Stress analyses of gravity dams can be carried out using one of the following methods depending upon the configuration of the dam, continuity between the blocks, and the degree of refinement required.

(*i*) The gravity method,

(*ii*) The trial-load method, and

(*iii*) The finite element method

The gravity method of analysis is used for designing straight gravity dams in which the transverse contraction joints of a gravity dam are neither keyed nor grouted. When the transverse contraction joints of a gravity dam are keyed, irrespective of their grouting, it becomes a three-dimensional problem and one should use the trial load method. This method assumes that the dam consists of three systems, *viz.,* the vertical cantilevers, the horizontal beams, and the twisted elements. Each of these systems is assumed to occupy the entire volume of the structure and is independent of the others. The loads on the dam are divided among these systems in such a manner as to cause equal deflections and rotations at conjugate points (1). This is achieved by trial process. The gravity method may, however, be used for a preliminary analysis of keyed and grouted dams. The finite element method, developed in recent years, can be used for both two-dimensional as well as three-dimensional problems. In this book, only the gravity method has been discussed.

16.4.1. Gravity Method

The gravity method of stress analysis is applicable to the general case of a gravity section when its blocks are not made monolithic by keying and grouting the joints between them. All these blocks of the gravity section act independently, and the load is transmitted to the foundation by cantilever action, and is resisted by the weight of the cantilever. The following assumptions are made in the gravity method of analysis (1):

(*i*) Concrete in the dam is a homogeneous, isotropic, and uniformly elastic material.

(*ii*) No differential movements occur at the site of the dam due to the water loads on walls and base of the reservoir.

(*iii*) All loads are transmitted to the foundation by the gravity action of vertical and parallel cantilevers which receive no support from the adjacent cantilever elements on either side.

(*iv*) Normal stresses on horizontal planes vary linearly from the upstream face to downstream face.

(*v*) Horizontal shear stresses have a parabolic variation across horizontal planes from the upstream face to downstream face of the dam.

The assumptions at serial numbers (*iv*) and (*v*) above are substantially correct, except for horizontal planes near the base of the dam where the effects of foundation yielding affect the stress distributions in the dam. Such effects are, however, usually small in dams of low or medium height. But, these effects may be significant in high dams in which cases stresses near the base should be checked by other suitable methods of stress analysis.

As shown in Fig. 16.4, ΣW and ΣH represent, respectively, the sum of all the resultant vertical and horizontal forces acting on a horizontal plane (represented by the section PQ) of a gravity dam. The resultant R of ΣW and ΣH intersects the section PQ at O' while O represents the centroid of the plane under consideration. The distance between O and O' is called the eccentricity of loading, e. When e is not equal to zero, the loading on the plane is eccentric and the normal stress σ_{yx} at any point (on the section PQ) x away from the centroid O is given as

$$\sigma_{yx} = \frac{\Sigma W}{A} \pm \frac{(\Sigma W)\, e}{I} x \tag{16.9}$$

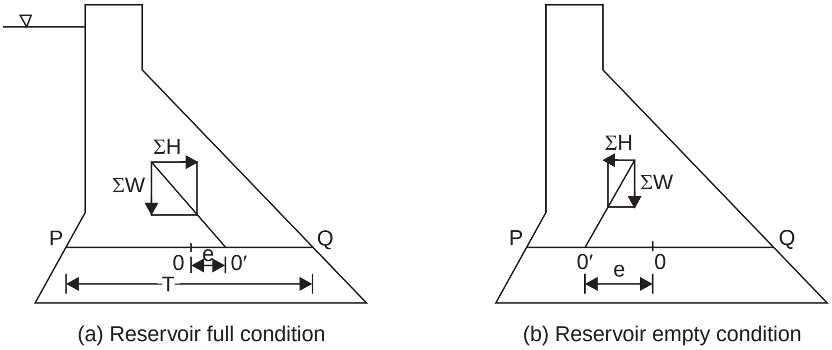


Fig. 16.4 Resultant force on a gravity dam

Here, A represents the area of the plane PQ, and I is the moment of inertia of the plane PQ about an axis passing through its centroid and parallel to the length of the dam. It should be noted that whereas the direct stress ($= \Sigma W/A$) at every point of the section PQ is always compressive, the nature of the bending stress ($= (\Sigma W)\, e\, x/I$) depends on the location of O' with respect to O. If O' lies between O and Q, there will be compressive bending stress for any point between O and Q, and tensile bending stress for any point between O and P. Accordingly, when the reservoir is full, one should use the positive sign in Eq. (16.9) for all points between O and Q, and the negative sign for all points between O and P. Similarly, when the reservoir is empty (in which case ΣH may be an earthquake force acting in the upstream direction), and O' lies between O and P, one should use the positive sign for all points between O and P, and the negative sign for all points between O and Q.

Considering unit length of the dam and the horizontal distance between the upstream edge P and the downstream edge Q of the plane PQ as T, one can write $A = T$ and $I = T^3/12$. Thus, Eq. (16.9) reduces to

$$\sigma_{yx} = \frac{\Sigma W}{T}\left(1 \pm \frac{12ex}{T^2}\right) \tag{16.10}$$

One can use this equation for determining the normal stress on the base of the dam BB' (Fig. 16.5) also. If the width of the base BB' is b, Eq. (16.10) for the base of the dam reduces to

$$\sigma_{yx} = \frac{\Sigma W}{b}\left(1 \pm \frac{12ex}{b^2}\right) \tag{16.11}$$

For the toe (B') and also the heel (B) of the dam, $x = b/2$. Hence, the normal stresses at the toe (σ_{yD}) as well as the heel (σ_{yU}) of the dam are as follows:

When the reservoir is full,

$$\sigma_{yD} = \frac{\Sigma W}{b}\left(1 + \frac{6e}{b}\right) \tag{16.12}$$

$$\sigma_{yU} = \frac{\Sigma W}{b}\left(1 - \frac{6e}{b}\right) \tag{16.13}$$

When the reservoir is empty,

$$\sigma_{yD} = \frac{\Sigma W}{b}\left(1 - \frac{6e}{b}\right) \tag{16.14}$$

$$\sigma_{yU} = \frac{\Sigma W}{b}\left(1 + \frac{6e}{b}\right) \tag{16.15}$$

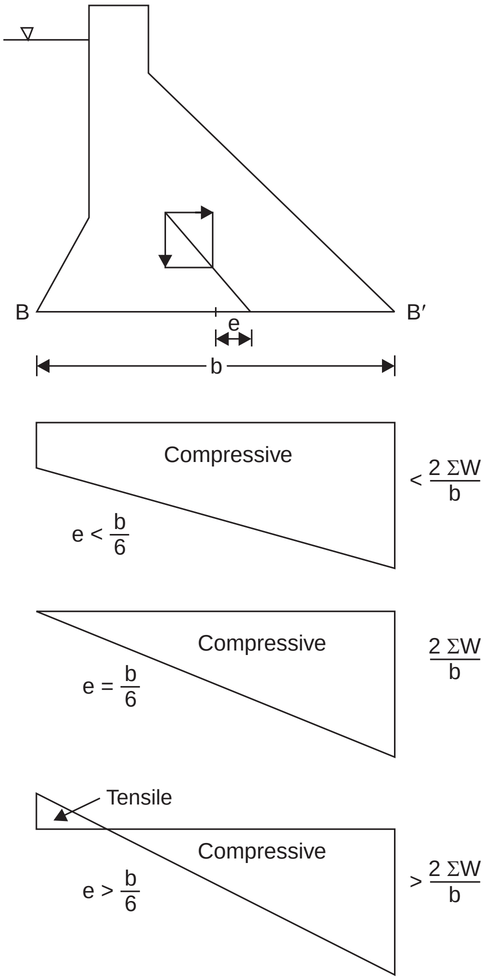


Fig. 16.5 Normal stresses on the base of a gravity dam

These equations indicate that if e is less than or equal to $b/6$, the stress is compressive all along the base and when e is greater than $b/6$ there can be tensile stresses on the base. The stress distributions for different values of e, when the reservoir is full, have been shown in Fig. 16.5. This means that if there has to be no tension at any point of the base of the dam, the resultant for all conditions of loading must meet the base within the middle-third of the base.

The principal planes and principal stresses enable one to know the range of the stresses acting at a point and thus design the structure on the basis of extreme values. A plane on which only normal stresses act is known as a principal plane. Shear stresses are not present on such a plane. Accordingly, the upstream and downstream faces of a gravity dam, having tail-water, are principal planes as the only force acting on these surfaces is on account of water pressure which acts normal to these surfaces. Further, at any point in a structure the principal planes are mutually perpendicular. Therefore, other principal planes would be at right angles to the upstream and downstream faces of a gravity dam. In an infinitesimal triangular element *PQR* at the toe of a gravity dam, (Fig. 16.6), the plane *QR* is at right angle to the downstream face, *PQ*. Hence, *PQ* and *QR* are the principal planes, and *PR* is part of the base of the dam. The stresses acting on the principal planes *PQ* and *QR* are, respectively, p' (tail-water pressure) and σ_{1D}, as shown in Fig. 16.6, and are the principal stresses. The normal and tangential stresses acting on *PR* are σ_{yD} and $(\tau_{yx})_D$, respectively. Since the element is very small, the stresses can be considered to be acting at a point. Considering the equilibrium of the element *PQR,* the algebraic sum of all the forces in the vertical direction should be zero. If one considers the unit length of the dam, then

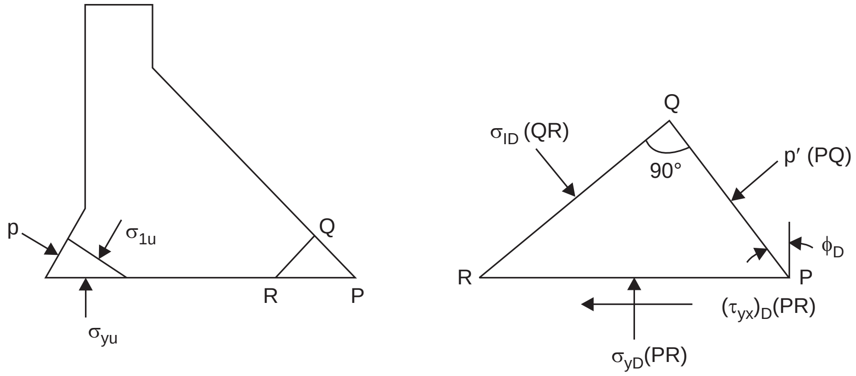


Fig. 16.6 Principal stresses in a gravity dam

$$\sigma_{1D}\,(QR)\cos\phi_D + p'\,(PQ)\sin\phi_D - \sigma_{yD}\,(PR) = 0$$

or
$$\sigma_{1D}\,(PR)\cos^2\phi_D + p'\,(PR)\sin^2\phi_D - \sigma_{yD}\,(PR) = 0$$

$\therefore$
$$\sigma_{1D} = \sigma_{yD}\sec^2\phi_D - p'\tan^2\phi_D \qquad (16.16)$$

Thus, knowing p' and σ_{yD} [from Eq. (16.11)] one can obtain, from Eq. (16.16), the principal stress σ_{1D} at the toe of the dam. Usually p' is either zero (no tail-water) or very small in comparison to σ_{1D}. Therefore, σ_{1D} is the major principal stress and p' is the minor principal stress. When p' is zero, Eq. (16.16) reduces to

$$\sigma_{1D} = \sigma_{yD}\sec^2\phi_D \qquad (16.17)$$

Considering the hydrodynamic pressure p_e' due to earthquake acceleration (towards the reservoir), the effective minor principal stress becomes $p' - p_e'$ and Eq. (16.16) becomes

$$\sigma_{1D} = \sigma_{yD}\sec^2\phi_D - (p' - p_e')\tan^2\phi_D \qquad (16.18)$$

When there is no tail-water, both p' and p_e' are zero, and Eq. (16.17) is used for the calculation of σ_{1D}.

Similarly, considering an infinitesimal element at the heel of the dam (Fig. 16.6), one can obtain expression for σ_{1U} as follows:

$$\sigma_{1U} = \sigma_{yU} \sec^2 \phi_U - (p + p_e) \tan^2 \phi_U \tag{16.19}$$

For the condition of empty reservoir, $p = p_e = 0$ and, hence,

$$\sigma_{1U} = \sigma_{yU} \sec^2 \phi_U \tag{16.20}$$

When the reservoir is full, the intensity of water pressure p is usually higher than the normal stress σ_{1U}. Therefore, at the heel, p is the major principal stress and σ_{1U} is the minor principal stress. For vertical upstream face, $\phi_U = 0$ and, therefore, σ_{1U} equals σ_{yU}.

Again, resolving the forces acting on the infinitesimal element PQR in the horizontal direction and equating their algebraic sum to zero for the equilibrium condition, one gets

$$(\tau_{yx})_D\,(PR) + p'\,(PQ) \cos \phi_D - \sigma_{1D}\,(QR) \sin \phi_D = 0$$

which yields
$$(\tau_{yx})_D = (\sigma_{1D} - p') \sin \phi_D \cos \phi_D$$
$$= (\sigma_{yD} \sec^2 \phi_D - p' \tan^2 \phi_D - p') \sin \phi_D \cos \phi_D$$

$\therefore$
$$(\tau_{yx})_D = (\sigma_{yD} - p') \tan \phi_D \tag{16.21}$$

Similarly, considering the equilibrium of the element at the heel of the dam,

$$(\tau_{yx})_U = -\,(\sigma_{yU} - p) \tan \phi_U \tag{16.22}$$

Including the effects of earthquake acceleration, Eqs. (16.21) and (16.22) reduce to

$$(\tau_{yx})_D = [\sigma_{yD} - (p' - p_e')] \tan \phi_D \tag{16.23}$$

and
$$(\tau_{yx})_U = -\,[\sigma_{yU} - (p + p_e)] \tan \phi_U \tag{16.24}$$

In the same way, one can calculate the principal and shear stresses at the upstream and downstream faces of the dam at any horizontal section by considering only the forces acting above the section.

16.5. ELEMENTARY PROFILE OF A GRAVITY DAM

The stability conditions required to be met for a gravity dam, subjected only to its self-weight W, force due to water pressure P, and uplift force U can be satisfied by a simple right-angled triangular section (Fig. 16.7) with its apex at the reservoir water level, and which is adequately wide at the base where the water pressure is maximum. Such a section is said to be an elementary profile of a gravity dam. For the empty-reservoir condition the only force acting on the dam is its self-weight whose line of action will meet the base at $b/3$ from the heel of the dam and thus satisfy the stability requirement of no tension. The base width of the elementary profile is determined for satisfying no tension and no sliding criteria as given below, and the higher of the two base widths is chosen for the elementary profile.

For the elementary profile shown in Fig. 16.7, if one considers that the resultant R of all the three forces W_c (= $0.5\, s\, \rho g b h$), W_1 (= $0.5\, \rho g h^2$), and U (= $0.5\, \rho g h b c'$) (here, s = specific gravity of concrete and c' is a correction factor for uplift force) passes through the downstream middle-third point, one gets

$$(0.5\, s\, \rho g b h)\,\frac{b}{3} - (0.5\, \rho g h^2)\,\frac{h}{3} - (0.5\, \rho g h b c')\,\frac{b}{3} = 0$$

or
$$b^2\,(s - c') = h^2$$

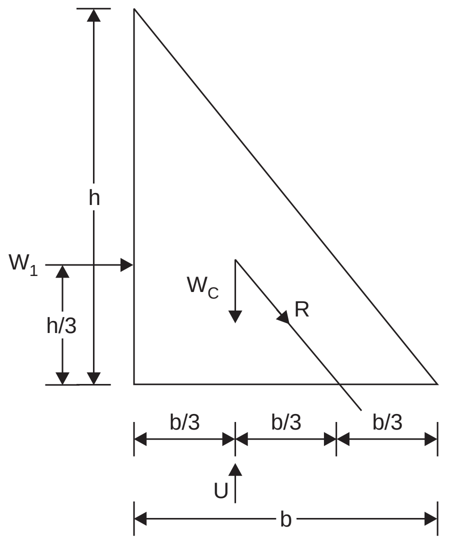


Fig. 16.7 Elementary profile of a gravity dam

or $$b = \frac{h}{\sqrt{s - c'}} \tag{16.25}$$

For $c' = 1$, $$b = \frac{h}{\sqrt{s - 1}} \tag{16.26}$$

and if uplift is ignored, $c' = 0$

$\therefore$ $$b = \frac{h}{\sqrt{s}} \tag{16.27}$$

For no-sliding requirement, one obtains $\mu\,(W_c - U) = P = W_1$ in which, μ is shear-friction coefficient.

or $$\mu\,(0.5\, s\rho g b h - 0.5\, \rho g h b c') = 0.5\, \rho g h^2$$

or $$b = \frac{h}{\mu\,(s - c')} \tag{16.28}$$

For $c' = 1$ $$b = \frac{h}{\mu\,(s - 1)} \tag{16.29}$$

and for no uplift, $$c' = 0, \text{ and } b = \frac{h}{\mu s} \tag{16.30}$$

It is obvious that for satisfying the requirement of stability, the elementary profile of a gravity dam should have minimum base width equal to the higher of the base widths obtained from no-sliding and no-tension criteria.

Again, for an elementary profile, $\Sigma W = (W_c - U)$

or $$\Sigma W = \frac{1}{2}\, b \rho g h\,(s - c')$$

$$\therefore \qquad \sigma_{yx} = \frac{\Sigma W}{b}\left(1 \pm \frac{12ex}{b^2}\right) \tag{16.31}$$

For no tension in the dam, $e = b/6$

Therefore, at the toe of the dam (*i.e.*, $x = b/2$)

$$\sigma_{yD} = \frac{2\,\Sigma W}{b}$$

$$\sigma_{yD} = \rho gh\,(s - c') \tag{16.32}$$

and at the heel of the dam (*i.e.*, $x = -\,b/2$)

$$\sigma_{yU} = 0$$

Accordingly, the principal stress $\sigma_{1D} = \sigma_{yD} \sec^2 \phi_D$

$$\therefore \qquad \sigma_{1D} = \rho gh\,(s - c')\,[1 + (b/h)^2]$$

$$= \rho gh\,(s - c')\left[1 + \frac{1}{(s - c')}\right]$$

$$= \rho\, gh\,(s - c' + 1) \tag{16.33}$$

Similarly,

$$(\tau_{yx})_D = \sigma_{yD} \tan \phi_D = \rho gh\,(s - c')\frac{b}{h}$$

$$= \rho gh\,(s - c')\,\frac{1}{\sqrt{s - c'}}$$

$$= \rho gh\,\sqrt{s - c'} \tag{16.34}$$

The principal and shear stresses at the heel are, obviously, zero.

Similarly, when the reservoir is empty, $\Sigma W = 0.5\ \rho gbhs$

$$\sigma_{yD} = 0$$

$$\sigma_{1U} = \sigma_{yU} = \frac{2\,\Sigma W}{b} = \rho\, ghs \tag{16.35}$$

Sometimes, depending upon whether or not the compressive stress at the toe σ_{1D} exceeds the maximum permissible stress σ_m for the material of the dam, a gravity dam is called a 'high' or 'low' dam. On this basis, the limiting height h_l is obtained by equating the expression for σ_{1D} with σ_m. Thus,

$$\sigma_m = \rho gh_l\,(s - c' + 1)$$

or

$$h_l = \frac{\sigma_m}{\rho g\,(s - c' + 1)} \tag{16.36}$$

If the height of a gravity dam is less than h_l, it is a low dam; otherwise, it is a high dam.

16.6. DESIGN OF A GRAVITY DAM

An elementary profile is only an ideal profile which needs to be modified for adoption in actual practice. Modifications would include provision of a finite crest width, suitable freeboard, batter in the lower part of the upstream face, and a flatter downstream face. The design of a gravity dam involves assuming its tentative profile and then dividing it into a number of zones by horizontal planes for stability analysis at the level of each dividing horizontal plane. The analysis can be either two-dimensional or three-dimensional. The following example illustrates the two-dimensional method of analysis of gravity dams.

Example 16.1 For the profile of a gravity dam shown in Fig. 16.8, compute principal stresses for usual loading and vertical stresses for extreme loading at the heel and toe of the base of the dam. Also determine factors of safety against overturning and sliding as well as shear-friction factors of safety for usual loading and extreme loading (with drains inoperative) conditions. Consider only downward earthquake acceleration for extreme loading condition.

Sediment is deposited to a height of 15 m in the reservoir. Other data are as follows:

Coefficient of shear friction, μ = 0.7 (usual loading)

= 0.85 (extreme loading)

Shear strength at concrete-rock contact, $C = 150 \times 10^4$ N/m^2

Weight density of concrete $= 2.4 \times 10^4$ N/m^3

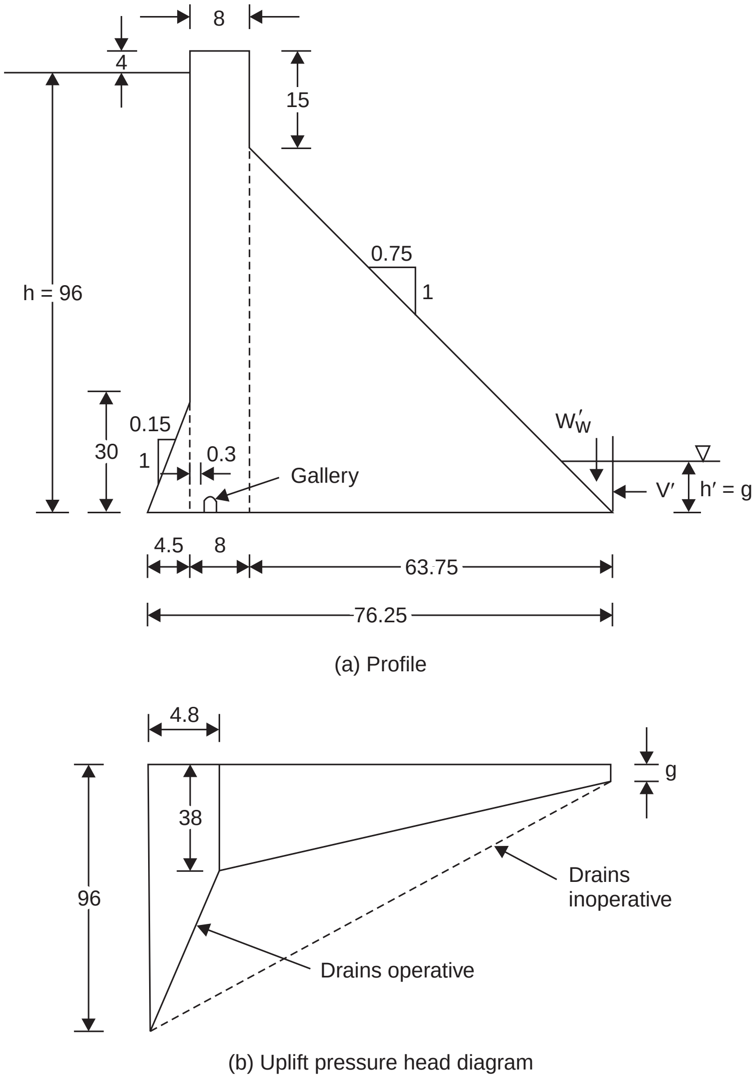


Fig. 16.8 Profile and uplift pressure diagram for the gravity dam of Example 16.1

Weight density of water $= 1 \times 10^4$ N/m^3

$\alpha_h = 0.1; \alpha_v = 0.05$

Solution:

Computation of Stresses:

(*i*) Usual loading combination (normal design reservoir elevation with appropriate dead loads, uplift (with drains operative), silt, ice, tail-water, and thermal loads corresponding to usual temperature):

Resultant vertical force = ΣW = sum of vertical forces at sl. nos. 1, 2 (*i*), 3 (*i*), and 4(*ii*) of Table 16.1.

$$= (8584.50 + 394.88 - 2000.68 + 32.48) \times 10^4$$
$$= 7011.18 \times 10^4 \text{ N}$$

Resultant horizontal force = ΣH = sum of horizontal forces at sl. nos. 2 (*ii*) and 4 (*i*) of Table 16.1.

$$= (-4567.50 - 153.00) \times 10^4$$
$$= -4720.50 \times 10^4 \text{ N}$$

Moment about toe of the dam at the base = ΣM = sum of moments at sl. nos. 1,2, 3 (*i*), and 4 of Table 16.1.

$$= [418302.75 + 27091.99 - 147334.50 - 96183.57 + 1662.88] \times 10^4$$
$$= 203539.55 \times 10^4 \text{ Nm}$$

Distance of the resultant from the toe, $\bar{y} = \dfrac{\Sigma M}{\Sigma W} = \dfrac{203539.55 \times 10^4}{7011.18 \times 10^4} = 29.03$ m

$\therefore$ Eccentricity, $e = 38.125 - 29.03 = 9.10$ m

(The resultant passes through the downstream of the centre of the base). Using Eqs. (16.12) and (16.13)

$$\sigma_{yD} = \frac{\Sigma W}{b}\left[1 + \frac{6e}{b}\right] = \frac{7011.18 \times 10^4}{76.25}\left[1 + \frac{6 \times 9.10}{76.25}\right]$$
$$= 157.79 \times 10^4 \text{ N/m}^2$$

$$\sigma_{yU} = \frac{\Sigma W}{b}\left[1 - \frac{6e}{b}\right] = \frac{7011.18 \times 10^4}{76.25}\left[1 - \frac{6 \times 9.10}{76.25}\right]$$
$$= 26.11 \times 10^4 \text{ N/m}^2$$

Using Eq. (16.16), the major principal stress at the toe,

$$\sigma_{1D} = \sigma_{yD} \sec^2 \phi_D - p' \tan^2 \phi_D$$

$$\therefore \quad \sigma_{1D} = 157.79 \times 10^4 \times 1.5625 - 9 \times 10^4 \times 0.5625$$
$$= 239.66 \times 10^4 \text{ N/m}^2$$

Using Eq. (16.21), shear stress at the toe, $(\tau_{yx})_D = (\sigma_{yD} - p') \tan \phi_D$

$$\therefore \quad (\tau_{yx})_D = (157.79 - 9) \times 10^4 \times 0.75$$
$$= 111.59 \times 10^4 \text{ N/m}^2$$

Table 16.1 Computation of forces and moments for unit length of dam section

Sl. No.	Type of load	Force computations (10^4 newtons)	Magnitude of forces: Vertical forces (downward + ve) (10^4 newtons)	Magnitude of forces: Horizontal forces (upstream + ve) (10^4 newtons)	Moment arm (metres)	Moment about the toe (anticlock-wise + ve) (10^4 Nm)
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1.	Dead load W_c	1 × 30 × 4.5 × 0.5 × 2.4	+ 162.00		73.25	+ 11866.50
		1 × 100 × 8 × 2.4	+ 1920.00		67.75	+ 130080.00
		1 × 85 × 63.75 × 0.5 × 2.4	+ 6502.50		42.50	+ 276356.25
			+ 8584.50			+ 418302.75
2.	Water load					
(i)	Vertical					
(a)	Head-water W_w	1 × 66 × 4.5 × 1.0	+ 297.00		76.25 – 2.25 = 74.00	+21978.00
		1 × 0.5 × 30 × 4.5 × 1.0	+ 67.50		76.25 – 1.50 = 74.75	+5045.63
(b)	Tail-water W_w'	1 × 0.5 × 9.0 × 6.75 × 1.0	+ 30.38		(9 × 0.75)/3 = 2.25	+ 68.36
			+ 394.88			+ 27091.99
(ii)	Horizontal					
(a)	Head-water W_l	1 × 0.5 × 96 × 96 × 1.0		– 4908.00	32.00	–147456.00
(b)	Tail-water W_l'	1 × 0.5 × 9 × 9 × 1.0		+ 40.50	3.00	+ 121.50
				– 4567.50		– 147334.50
3.	Uplift force, U					
(i)	Drains operative	1 × 38 × 4.8 × 1	– 182.40		73.85	– 13470.24
		1 × 58 × 4.8 × 0.5 × 1	– 139.20		74.65	– 10391.28
		1 × 29 × 71.45 × 0.5 × 1	– 1036.03		47.63	– 49345.87
		1 × 9 × 71.45 × 1	– 643.05		35.73	– 22976.18
			– 2000.68			– 96183.57
(ii)	Drains inoperative	1 × 0.5 × 87 × 76.25 × 1	– 3316.88		50.83	– 168597.01
		1 × 9 × 76.25 × 1	– 686.25		38.13	– 26166.71
			– 4003.13			– 194763.72

(Contd.)...

(1)	(2)	(3)	(4)	(5)	(6)	(7)
4.	Load due to sediment deposit W_s					
(i)	Excess horizontal pressure	1 × 0.5 × 15 × 15 × 1.36		– 153	15/3 = 5.00	– 765.00
(ii)	Excess vertical load	1 × 0.5 × 15 × 2.25 × 1.925	32.48		76.25 – 4.5/3 = 74.75	+ 2427.88
						+ 1662.88
5.	Earthquake forces					
(i)	Inertial horizontal force due to weight of the dam	162.00 × 0.1		– 16.20	10.00	– 162.00
		1920.00 × 0.1		– 192.00	50.00	– 9600.00
		6502.50 × 0.1		– 650.25	28.33	– 18421.58
				– 858.45		– 28183.58
(ii)	Hydrodynamic force	At the base $c = c_m = 0.73$ (for $\phi_U = 0$)				
(a)	Head-water ($c_1 = c_m$ for $y = h$)	$V_{pe} = 0.726\,(0.73 \times 0.1 \times 1 \times 96) \times 96$		– 488.43		
		$M_{pe} = 0.299\,(0.73 \times 0.1 \times 1 \times 96) \times (96)^2$				– 19311.13
(b)	Tail-water[1]	At the base $c = c_m = 0.47$ (for $\phi_D = \tan^{-1}(0.75)$)				
		$V_{pe} = 0.726\,(0.47 \times 0.1 \times 1 \times 9) \times 9$		– 2.76		
		$M_{pe} = 0.299\,(0.47 \times 0.1 \times 1 \times 9) \times (9)^2$				– 10.25
				– 491.19		19321.38

[1]**Hydrodynamic force due to tail -water has been considered negative for the most critical condition.**

Using Eq. (16.19), with $p_e = 0$, the minor principal stress at the heel

$$\sigma_{1U} = \sigma_{yU} \sec^2 \phi_U - p' \tan^2 \phi_U$$
$$= 26.11 \times 10^4 \times 1.0225 - 96 \times 10^4 \times 0.0225$$
$$= 24.54 \times 10^4 \text{ N/m}^2$$

Using Eq (16.22), shear stress at the heel, $(\tau_{yx})_U = (\sigma_{yU} - p') \tan \phi_U$

$\therefore$
$$(\tau_{yx})_U = -(26.11 - 96.00) \times 10^4 \times 0.15$$
$$= 10.48 \times 10^4 \text{ N/m}^2$$

Further, major principal stress at the heel = $p = 96 \times 10^4$ N/m^2

and minor principal stress at the toe = $p' = 9.0 \times 10^4$ N/m^2

$$\text{Factor of safety for overturning} = \frac{\text{stabilising moment}}{\text{overturning moment}}$$

$$= \frac{(418302.75 + 27091.99 + 1662.88) \times 10^4}{(147334.50 + 96183.57) \times 10^4} = 1.84$$

$$\text{Sliding factor} = \frac{\Sigma H}{\Sigma W}$$

$$= \frac{4720.55 \times 10^4}{7011.18 \times 10^4} = 0.67$$

Shear-friction factor of safety (with drains operative),

$$F_s = \frac{Cb \times 1 + \mu \Sigma W}{\Sigma H}$$

$$= \frac{150 \times 10^4 \times 76.25 \times 1 + 0.7(7011.18 \times 10^4)}{4720.50 \times 10^4} = 3.46$$

(*ii*) Extreme loading combination (usual loading combination with drains inoperative and the loading due to earthquake):

The inertial and hydrodynamic forces and corresponding moments due to horizontal earthquake have been computed as shown in Table 16.1. The effect of vertical earthquake can be included in stability computations by multiplying the forces by $(1 + \alpha_v)$ and $(1 - \alpha_v)$ for upward and downward accelerations, respectively. Since the computation of hydrodynamic force involves the use of unit weight of water, the hydrodynamic force will also be modified by vertical acceleration due to earthquake. Further, the effect of earthquake on uplift forces is considered negligible. For 'reservoir full' condition, the downward earthquake acceleration results in more critical condition. Therefore, the following computations have been worked out for the downward earthquake acceleration only.

Resultant vertical force with downward earthquake acceleration

$$= (8584.50 + 394.88 + 32.48) \times 10^4 \times 0.95 - 2000.68 \times 10^4$$
$$= 6560.59 \times 10^4 \text{ N}$$

Resultant horizontal force with downward earthquake acceleration

$$= (4567.50 + 153 + 858.45 + 491.19) \times 10^4$$
$$= 6070.14 \times 10^4 \text{ N}$$

Resultant moment about the toe with downward acceleration

$$= (418302.75 + 27091.99 - 147334.50 + 1662.88 - 28183.58 - 19321.38) \times 10^4 \times 0.95 - 194763.72 \times 10^4$$

$$= 44843.53 \times 10^4 \text{ Nm}$$

Now, $$\bar{y} = \frac{\Sigma M}{\Sigma W} = \frac{44843.53 \times 10^4}{6560.59 \times 10^4} = 6.835 \text{ m}$$

∴ Eccentricity, $e = 38.125 - 6.835 = 31.29$ m

The resultant passes through the downstream side of the centre of the base. The value of e is more than $b/6$ *i.e.*, 12.71 m. Therefore, there would be tensile stresses around the heel of the dam. The vertical stresses at the toe and heel with downward earthquake acceleration are,

$$\sigma_{yD} = \frac{\Sigma W}{b}\left(1 + \frac{6e}{b}\right) = \frac{6560.59 \times 10^4}{76.25}\left(1 + \frac{6 \times 31.29}{76.25}\right)$$

$$= 297.89 \times 10^4 \text{ N/m}^2$$

and $$\sigma_{yU} = \frac{\Sigma W}{b}\left(1 - \frac{6e}{b}\right) = \frac{6560.59 \times 10^4}{76.25}\left(1 - \frac{6 \times 31.29}{76.25}\right)$$

$$= -125.81 \times 10^4 \text{ N/m}^2$$

Factor of safety against overturning

$$= \frac{(418302.75 + 27091.99 + 1662.88) \times 10^4}{(147334.50 + 194763.72 + 28183.58 + 19321.38) \times 10^4} = 1.15$$

Sliding factor $$= \frac{\Sigma H}{\Sigma W}$$

$$= \frac{(4567.50 + 153 + 858.45 + 491.19) \times 10^4}{6560.59 \times 10^4}$$

$$= \frac{6070.14}{6560.59} = 0.925$$

Shear-friction factor of safety, $$F_s = \frac{Cb \times 1 + \mu \Sigma W}{\Sigma H}$$

$$= \frac{150 \times 10^4 \times 76.25 \times 1 + 0.7 \times 6560.59 \times 10^4}{6070.14 \times 10^4}$$

$$= \frac{16029.91}{6070.14} = 2.64$$

16.7. FOUNDATION TREATMENT

The foundation of a gravity dam should be firm and free of major faults which, if present, may require costly foundation treatment. The entire loose overburden over the area of the foundation to be occupied by the base of the dam should be removed. The dam itself must be based on the firm material which can withstand the loads imposed by the dam, reservoir, and other appurtenant structures. To consolidate the rock foundation and to make it an effective barrier to seepage under the dam, the foundation is often grouted. Grouting consists of filling the cracks and voids in the foundation with grout mixtures (cement-water mixtures) under pressure. The spacing, length, pattern of grout holes, and grouting procedure depend on the height of the structure and the geologic characteristics of the foundation. Grouting operations are carried out from the surface of the excavated foundation or from galleries within the dam or from tunnels driven into the abutments or from other suitable locations, such as the upstream fillet of the dam.

For the purpose of seepage control, a deep grout curtain is constructed near the heel of the dam by drilling deep holes and grouting them under high pressures. These holes, if drilled from a gallery, are identified as 'A' holes. Curtain grouting is carried out only after consolidation grouting so that the higher grouting pressure does not cause displacement in the rock or loss of grout through surface cracks. In low dams, galleries are not provided and high-pressure grouting is carried out through curtain holes located in the upstream fillet of the dam before reservoir filling begins. Such grouting holes are identified as 'C' holes.

Consolidation grouting to fill voids, fracture zones, and cracks at and below the surface of the excavated foundation is accomplished by drilling and grouting relatively shallow holes. These holes are identified as 'B' holes and the grouting is carried out at low pressures.

Water-cement ratios for grout mixes depend on the permeability of the rock foundation and may be around 6 : 1 for consolidation grouting. Pressures for consolidation grouting depend on the strength characteristics of the foundation and may vary over wide range of 70 to 700 kPa. Holes of diameters of approximately 5 cm, spaced at about 5 m intervals, are usually drilled to a depth of about 15 m depending upon the local conditions.

Even after providing a grout curtain some water will percolate through and around the grout curtain. This water, if not removed, may build up very high hydrostatic pressures at the base of the structure. Hence, this water must be suitably drained. This is achieved by drilling one or more lines of holes downstream of the grout curtain. The spacing, depth, diameter, and pattern of these holes would depend on the foundation conditions. Drain holes are drilled only after completion of all foundation grouting. These can be drilled from foundation and drainage galleries within the dam, or from the downstream face of the dam. A suitable system for the collection and safe disposal of the drainage water must also be provided.

16.8. MASS CONCRETE FOR DAMS

Mass concrete can be defined (6) as any large volume of cast-in-place concrete with dimensions large enough to require measures to cope with the generation of heat and attendant volume change to minimize cracking. Like regular concrete, mass concrete too is primarily composed of cement, aggregate, and water. Additionally, it has pozzolans and other admixtures to improve its characteristics.

Proper proportioning of mass concrete mixture is aimed at: (*i*) achieving economy, (*ii*) low temperature-rise potential with adequate workability for placing, and (*iii*) adequate strength, durability, and impermeability to serve efficiently the structure in which it is used. For this purpose, "low heat" portland cement would always be preferred for massive structures such as dams. Obviously, both economy and low rise in temperature would be achieved by limiting the cement content of mass concrete to as low a value as possible.

Aggregate grading has considerable effect on the workability of concrete. Fine aggregate is defined (6) as aggregate passing No. 4 (4.76 mm) sieve. It may be composed of natural grains, manufactured grains obtained by crushing larger size rock particles, or a mixture of the two. Fine aggregate should consist of hard, dense, durable, and uncoated rock fragments, and should not contain harmful grains of clay, silt, dust, mica, organic matter or other impurities to such an extent that they affect adversely the desired properties of concrete.

Coarse aggregate is defined (6) as gravel, crushed gravel, or a crushed rock, or a mixture of these, generally within the range of 4.76 mm to 150 mm in size. Coarse aggregate should also consist of hard, dense, durable, and uncoated rock fragments. Rock which is very fragile or which tends to degrade during processing, transporting, or in storage should be avoided. Further, rocks having an absorption greater than 3 per cent or a specific gravity less than 2.5 are not considered suitable for mass concrete.

The shape of the aggregate particles affects workability and, hence, water requirement. Round particles provide best workability. More than 25 per cent of flat (width-thickness ratio greater than 3) and elongated (length-width ratio greater than 3) particles should not be permitted in each size group.

Water used for preparing mass concrete mix should neither significantly affect the hydration reaction of portland cement nor interfere with the phenomena that are intended to occur during the mixing, placing, and curing of concrete. Water which is suitable for human consumption is acceptable for use in mass concrete.

Pozzolans are used to improve the workability and quality of concrete, to effect economy, and to protect against disruptive expansion caused by the reaction between different constituents of mass concrete. A pozzolan is defined (6) as a siliceous or siliceous and aluminous material which, in itself, possesses little or no cementitious value but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties. Natural pozzolanic materials occur in the form of obsidian, pumicite, volcanic ashes, tuffs, clays, shales, and diatomaceous earth. Most of these pozzolans require grinding. Fly ash (fuel dust from power plants burning coal) too can be an excellent pozzolan as it has a low carbon content, a fineness about the same as that of portland cement, and occurs in the form of very fine glassy spheres.

Admixtures are generally used to alter the properties of concrete (such as increased workability or reduced water content, acceleration or retardation of setting time, acceleration of strength development, and improved resistance to weather and chemical attacks) to make it more suitable for a particular purpose. For example, calcium chloride can be used to accelerate strength development in mass concrete during winter. Air-entraining admixtures (inexpensive soaps, detergents, etc.) entrain air which greatly improves the workability of concrete and thus permits the use of harsher and more poorly graded aggregates and also those of undesirable shapes.

Mass concrete usually contains low portions of cementing materials, sand and water. Uniformity of batching is, therefore, very essential for achieving the desired level of workability. Since mass concrete is produced on a large scale, the uniformity of batching is easily attained by using the most effective methods and equipment such as (6): (*i*) finish screening of coarse aggregate at the batching plant, preferably on horizontal screens, (*ii*) a device for instant reading of approximate moisture content of sand, (*iii*) refinements in batching equipment such as full-scale springless dials which register all stages of the weighing operation, (*iv*) automatic weighing and cutoff features, (*v*) interlocks to prevent recharging when some material remains in a scale hopper, and (*vi*) graphic recording of the various weighing and mixing operations.

Mass concrete is best placed in successive layers which should not exceed 45 to 50 cm in thickness with 10 to 15 cm maximum-size aggregate and less than 4 cm slump placed with 3 to

6 m^3 buckets and powerful 15 cm diameter vibrators. Vibration is vital for successful use of efficient, lean, low-slump mass concrete. Mass concrete is best cured with water (for the additional cooling in warm weather) for at least 14 days or up to 28 days if pozzolan is used as one of the cementing materials.

A major problem associated with mass concrete is the probability of high tensile stresses due to generation of heat by the hydration of cement along with subsequent differential cooling. A decrease in temperature of concrete causes volumetric changes resulting in the development of tensile stresses and consequent cracking in the concrete mass. Such cracking in concrete dams is undesirable as it adversely affects their water-tightness, internal stresses, durability, and appearance. Temperature drop is, therefore, controlled by controlling placing temperature, limiting the temperature-rise potential of concrete, controlling lift thickness and placing schedule, and removal of heat through embedded cooling coils. From the considerations of temperature control, the longer interval between successive lifts is preferred provided the ambient temperatures are lower than those of the concrete surfaces while internal temperature is rising. Presently, it is common practice to pre-cool mass concrete before its placement. Best uniformity of mix is attained when maximum use of ice is made for precooling mass concrete.

16.9. STRUCTURAL JOINTS

Joints in concrete dams are essentially designed cracks which are suitably located and treated to minimise undesirable effects. These joints are of three types.

Contraction joints prevent tensile cracks on account of volumetric shrinkage due to drop in temperature. A concrete dam is usually constructed in blocks separated by the transverse contraction joints which are normal to the axis of the dam and are continuous from the upstream face to the downstream face. Longitudinal contraction joints in the blocks formed by the transverse contraction joints are also sometimes considered necessary. The contraction joints are vertical and normally extend from the foundation to the top of the dam. Reinforcement should not extend across a contraction joint.

Expansion joints accommodate volumetric increase due to rise in temperature besides preventing transfer of stress between different units of the structure. Contraction and expansion joints are constructed in such a manner that there is no bond between the adjacent units of the structure.

Construction joint is the surface of the previously placed concrete upon or against which new concrete is to be placed. Besides permitting subsequent placing of concrete, these joints facilitate construction, reduce shrinkage stresses, and permit installation and embedded metal work. Suitable measures are adopted to ensure proper bond between the previously placed concrete and new concrete. These measures include cleaning by high-velocity water jets and roughening the surface of the previously placed concrete. A thin mortar layer is sometimes placed on the surface before placing new concrete. Construction joints do not require water seals and keys.

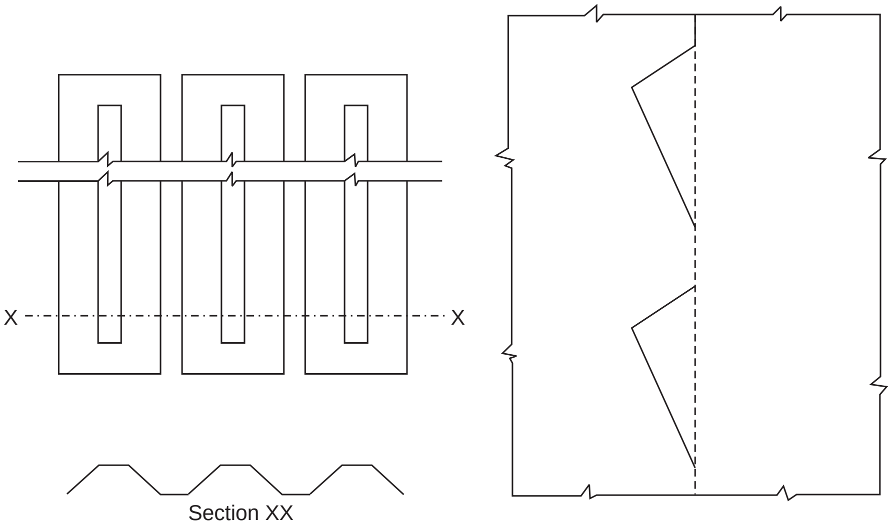
16.10. KEYS AND WATER SEALS

Vertical keys in transverse joints [Fig. 16.9 (*a*)] and horizontal keys in longitudinal joints [Fig. 16.9 (*b*)] are provided in a dam to increase the shearing resistance between its adjacent concrete blocks. The resulting structure has better stability due to the transfer of load from one block to another through the keys. The keys also increase the percolation distance through the joints and thus reduce water leakage. They also hasten the sealing of the joints with sediment deposits.

Shear keys provided in longitudinal contraction joints improve the stability of the dam by increasing the resistance to vertical shear.

Contraction joints are sometimes grouted to bind the blocks together so that the structure behaves like a monolithic mass. Even if the stability of the dam does not require the entire mass to act as a monolith, the longitudinal contraction joints must always be grouted so that blocks in a transverse row act monolithically.

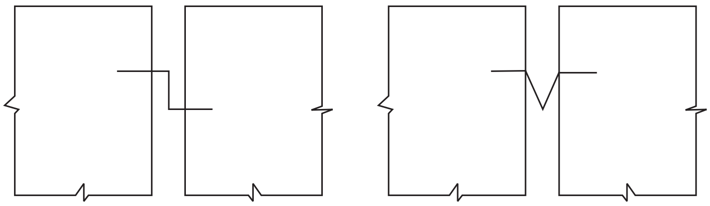
The opening of the transverse joints between adjacent blocks creates a passage for leakage of water from the reservoir to the downstream face of a dam. To prevent this leakage, seals (Fig. 16.10) are installed in the joints adjacent to the upstream face. Seals in longitudinal and transverse joints are also useful during grouting operations for confining the grout mixture to the joint. The most common type of seal used in concrete dams has been a metal seal which is embedded in concrete across the joint. In addition, polyvinyl chloride (PVC) seals and rubber seals have also been used.



(a) Vertical keys for transverse joints

(b) Horizontal keys for longitudinal joints

Fig. 16.9 Typical keys for joints in gravity dam



(a) Z-type

(b) M-type

Fig. 16.10 Metallic water stops in transverse joints

16.11. GALLERIES

A gallery is an opening within a dam that provides access into or through the dam. These may run either longitudinally or transversely and may be either horizontal or inclined. The following are the common types and uses of galleries (1):

(*i*) Drainage galleries provide a drainage way for water percolating through the upstream face or seeping through the foundation.

(*ii*) Grouting galleries provide space for drilling and grouting the foundation.

(*iii*) Inspection galleries provide access to the interior of the structure for observing its behaviour after completion.

(*iv*) Gate galleries (or chambers or vaults) provide access to, and room for, such mechanical and electrical equipment as are used for the operation of gates in spillways and outlet works.

(*v*) Cable galleries provide access through the dam for control cables and/or power cables and related equipment.

(*vi*) Visitors' galleries provide access routes for visitors.

Other galleries may be needed in a particular dam to meet special requirements, such as the artificial cooling of concrete blocks, the grouting of contraction joints, and so on.

16.12. INSTRUMENTATION

The behaviour of concrete gravity dam as well as its foundation is observed through suitable instruments during the periods of construction, reservoir filling, and operation of the reservoir. Instruments are employed for measurements of strain, temperature, stress, deflection, and deformation of the foundation. The data obtained from various instruments are helpful not only for assessing the safety of the structure during construction as well as operation periods, but, also for future design and operational studies.

The instruments (or the methods) employed for obtaining information on the behaviour of a concrete dam and its foundation can be broadly classified into two categories. The first category includes such instruments as are either embedded in the concrete mass of the structure or placed on surface of the dam and its appurtenant structures. Such instruments measure strain, stress, contraction joint opening, temperature, concrete pore pressures, and foundation or deformation. The second category of instruments include precise surveying instruments which make measurements of horizontal and/or vertical deformation using targets located on the downstream face or top of a dam, inside the galleries and vertical wells in a dam, in tunnels, and on the abutments.

16.13. OUTLETS

An outlet in a dam is a combination of structures and equipment required for the safe operation and control of water released from a reservoir to serve different objectives. Outlets are classified according to the purpose they serve. River outlets regulate flows to the river and control the water level in the reservoir. Besides, the river outlets may be useful for increasing the flow downstream of the dam alongwith the normal spillway discharge. In addition, river outlets may also act as a flood control regulator to release waters temporarily stored in flood control storage space or to evacuate storage in anticipation of flood inflows. Further, river outlets may also serve to empty the reservoir for inspection, repair, and maintenance of the upstream face of the dam and other structures which are normally inundated.

Irrigation, municipal water supply, and industrial outlets control the flow of water into a canal, pipeline, or river to satisfy specified needs. The design of these outlets will depend primarily on irrigation, commercial, industrial, and residential water needs and also on the capacity requirements with the reservoir at a predetermined elevation as well as the amount of control required as the elevation of the reservoir fluctuates.

Power outlets provide passage of water to the turbines for generation of hydropower. The power outlets should be so designed as to minimise hydraulic losses and to obtain the maximum economy in construction as well as operation.

EXERCISES

16.1 Describe the common forces acting on a gravity dam.

16.2 What are the main causes of failure of a gravity dam ?

16.3 Write notes on

(*i*) Galleries in gravity dams,

(*ii*) Foundation treatment for gravity dams,

(*iii*) Structural joints in gravity dams, and

(*iv*) Keys and water seals in gravity dams.

16.4 Check the stability of the gravity dam shown in Fig. 16.11 and calculate the stresses at the toe and heel for both empty and full-reservoir conditions. For full-reservoir conditions, assume an earthquake acceleration equal to 0.1 g. Assume coefficient of shear friction = 0.70, specific gravity of concrete = 2.40, and shear strength at concrete-rock contact surface = 140 × 10^4 N/m^2. Other data, if required, may be suitably assumed.

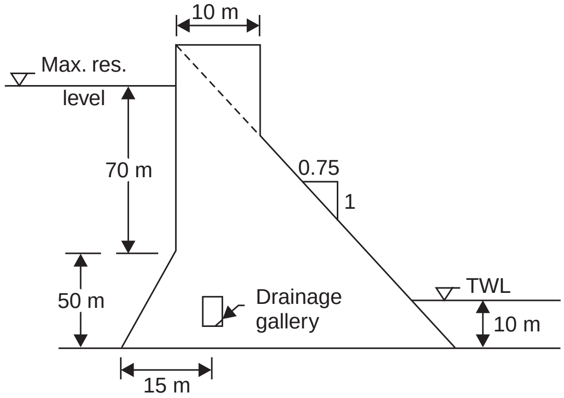


Fig. 16.11 Sketch for Exercise 16.4

16.5 Figure 16.12 shows cross-section of a gravity dam which has the following data:

Angle of internal friction of silt	= 30°
Submerged unit weight of silt	= 15 kN/m^3
Horizontal earthquake acceleration	= 0.15 g
Vertical earthquake acceleration	= 0.075 g
Shear strength between dam and foundation	= 1500 kN/m^2
Coefficient of friction	= 0.75

Determine, for extreme load combination (without ice pressure), the following:

(*a*) various relevant stresses at the toe and heel, and

(*b*) factors of safety against overturning and sliding, and the shear friction factor.

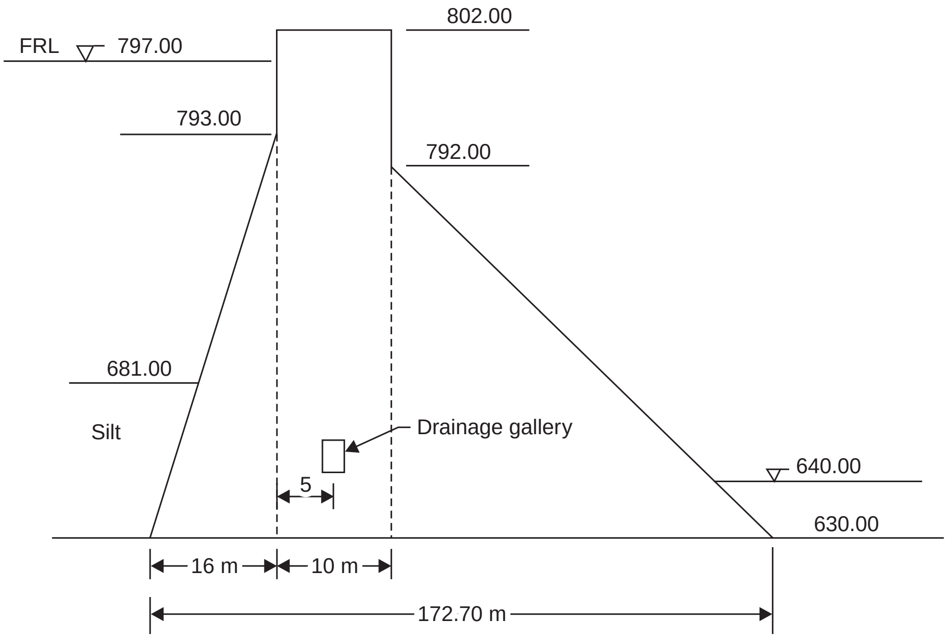


Fig. 16.12 Sketch for Exercise 16.5

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17
SPILLWAYS

17.1. GENERAL

The occurrence of a flood in an unobstructed natural stream is considered to be a natural event for which no individual or group is held responsible. However, if a flood occurs on account of the failure of an artificial obstruction (such as a dam) constructed across a natural stream, the agency responsible for the construction of the obstruction is held responsible. Embankment dams constructed of earth or rockfill material are very likely to be destroyed, if overtopped. Concrete dams may, however, tolerate moderate overtopping. The damage to life and property on account of the failure of a dam would be catastrophic. As such, there must always be a provision to release excess water safely when the reservoir has been filled to its capacity so that the dam itself is not overtopped. This is achieved by constructing a spillway. Spillways release safely the surplus water which cannot be contained in the reservoir created by the dam. The surplus water is usually drawn from the top of the reservoir and conveyed through an artificial waterway back to the river downstream of the dam or to some other natural drainage channel. Spillway can be constructed either as part of the main dam, such as in overflow section of a concrete dam or as a separate structure altogether. Besides being capable of releasing surplus water, a spillway must be able to meet hydraulic and structural requirements and must be located such that spillway discharges do not damage the toe of the dam. Insufficient spillway capacity and/or the failure of a spillway will cause widerspread damage and loss of life. As such, the design criteria for a spillway are usually conservative. The inflow design flood, used to determine the spillway capacity, is also estimated conservatively.

The frequency of flow over a spillway would mainly depend on the runoff characteristics of the drainage area, reservoir storage, and the available outlet and/or diversion capacity. For example, at a dam with storage and outlet capacities relatively small (compared to normal river flows), spillway will be used almost continuously. Under favourable site conditions, one should examine the possibility of providing an auxiliary spillway in conjunction with a smaller service spillway. In such a situation, the service spillway is designed to pass frequent floods of smaller magnitude. The auxiliary spillway operates only when flood of magnitude larger than the discharging capacity of the service spillway is passing. Sometimes, the capacity of outlet structures may be increased so that these may also serve as service spillways. The auxiliary spillway is used infrequently and, hence, is not designed for the same degree of safety as required for other spillways.

At some projects, emergency spillways are also provided for additional safety to meet emergencies not anticipated in the normal design. Such emergencies could be on account of malfunctioning of regular spillway gates, damage to the regular spillway, shutdown of outlet works, or the occurrence of two floods in quick succession.

A dam always has some storage capacity above its normal storage level. This storage capacity is termed surcharge storage. If a dam could be made so high as to provide ample surcharge storage to contain the entire volume of the incoming flood, theoretically no spillway, except an emergency type, would be needed provided that the outlet capacity of the dam can evacuate the surcharge storage well before the arrival of the next flood. Such an ideal situation permitting retention of entire incoming flood by surcharge storage, however, would never exist. In the absence of surcharge storage, spillway must be sufficiently large to pass the peak flood discharge. In such a situation, the peak rate of inflow is more important than the total volume of the incoming flood. On the other hand, if relatively large surcharge storage can be made economically at a dam, a portion of the incoming flood is retained temporarily in the surcharge storage of the reservoir and the discharging capacity of spillway can be reduced significantly. Economic considerations will usually require that a reservoir be designed to have reasonable surcharge storage as well.

Using the overflow characteristics of an assumed spillway type and known inflow design flood, the maximum spillway discharge and the maximum reservoir water level can be determined by flood routing. For known maximum spillway discharge, the components of the trial spillway can be designed and a complete layout of the spillway prepared. Cost estimates of the trial spillway and dam can now be made.

All relevant factors of topography, hydrology, geology, hydraulics, design requirements, costs, and benefits must be considered for determining the best combination of storage and spillway capacity for the chosen inflow design flood. Some of these important factors are as follows (1):

(*i*) The characteristics of the inflow flood hydrograph,

(*ii*) The damages which would result if the inflow flood occurred (*a*) without the dam, (*b*) with the dam in place, and (*c*) after the failure of the dam or spillway.

(*iii*) The effects of various dam and spillway combinations on the upstream and downstream of the dam on account of the resulting backwater and tail-water effects.

(*iv*) Relative costs of various combinations of storage and spillway capacity including the outlet facilities which can be utilised for the duration of the flood.

The cost estimates of various combinations of spillway capacity and dam height for the trial spillways will form the basis for the selection of an economical spillway and the optimum combination of surcharge storage (or the height of the dam) and the spillway capacity.

17.2. FLOOD ROUTING

The change in storage Δs of a reservoir depends on the difference between the amount of inflow and outflow and can be expressed by the equation,

$$\Delta s = Q_i \Delta t - Q_o \Delta t \tag{17.1}$$

where, Δs is the change in storage during time interval Δt, and Q_i and Q_o are, respectively, the average rates of inflow and outflow during the time interval Δt. The rate of inflow at any time is obtained from the inflow flood hydrograph selected for the purpose (Fig. 17.1). The rate of outflow (which should, strictly speaking, include outflow from the river outlets, irrigation outlets, and power turbines) is obtained from the outflow discharge versus reservoir water surface elevation curve (Fig. 17.2). Similarly, the storage is obtained by the reservoir storage versus reservoir water surface elevation curve (Fig. 17.3). While the inflow flood hydrogrpah and the storage curve would remain fixed for a given project site, the spillway discharge curve would depend not only on the size and type of the spillway but also on the manner in which the spillway and outlets, in some cases, are operated to regulate the outflow. If one could establish simple mathematical expression for these three curves, a solution of flood routing

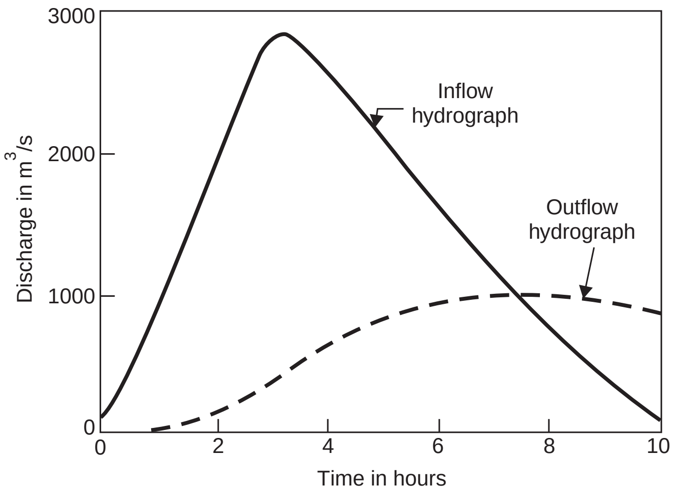


Fig. 17.1 Typical hydrographs of inflow and outflow floods

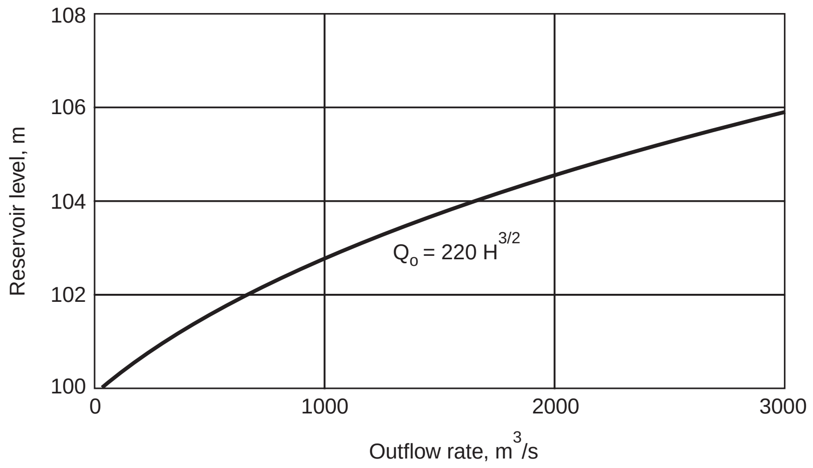


Fig. 17.2 Spillway discharge versus reservoir level

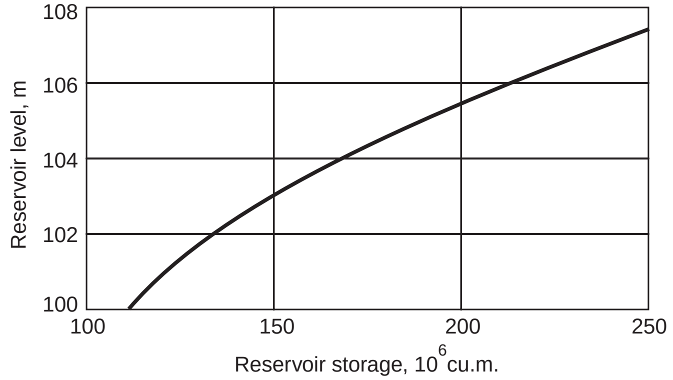


Fig. 17.3 Reservoir level versus reservoir capacity

could be obtained simply by mathematical integration. This, however, is not possible and one has to use one of the several techniques of flood routing ranging from purely arithmetical method to an entirely graphical solution. One such simple arithmetical trial and error method makes the flood routing computations (Table 17.1) in the following manner:

(*i*) Select a suitable time interval Δt (Col. 2),

(*ii*) Obtain inflow rates for different times from the inflow hydrograph (Fig. 17.1) and enter these values in Col. 3.

(*iii*) Average inflow rates for time interval Δt are entered in Col. 4.

(*iv*) Determine the amount of inflow volume in time and enter the value in Col. 5.

(*v*) Assume a trial reservoir water surface elevation (Col. 6).

(*vi*) Obtain the rate of outflow from Fig. 17.2 for the assumed trial reservoir water surface elevation and enter its value in Col. 7.

(*vii*) Obtain the average rate of outflow for the time interval under consideration and enter this value in Col. 8.

(*viii*) Obtain the amount of outflow volume in time Δt and enter the value in Col. 9.

(*ix*) Obtain the change in storage Δs by subtracting outflow volume from inflow volume (Col. 10).

(*x*) Add Δs of Col. 10 to the total reservoir storage at the beginning of the time interval (Col. 11).

(*xi*) Determine the reservoir water surface elevation using Fig. 17.3 and enter the same in Col. 12.

(*xii*) Compare the reservoir water surface elevation (Col. 12) with the trial reservoir water surface elevation (Col. 6). If they do not match within specified accuracy, say 3 cm, make another trial and repeat until the agreement is reached.

Based on the above procedure, flood routing computations have been carried out for the curves of Figs. 17.1-17.3 as shown in Table 17.1. Using the values of the outflow rate (Col. 7), the outflow hydrograph can be prepared as shown in Fig. 17.1. Obviously, the volume indicated by the area between the inflow and outflow hydrographs would be the surcharge storage.

Another method of routing a flood is a graphical method which is also known as inflow-storage-discharge curves method or, simply, ISD method. Equation (17.1) can, alternatively, be written as

$$s_{n+1} - s_n = \frac{Q_{i,n} + Q_{i,n+1}}{2}\Delta t - \frac{Q_{o,n} + Q_{o,n+1}}{2}\Delta t \tag{17.2}$$

in which, $Q_{i,n}$ and $Q_{o,n}$ represent, respectively, the inflow and outflow discharge rates at the beginning of the n^{th} time step (or, at the end of $(n-1)^{\text{th}}$ time step) of duration Δt during which the reservoir storage volume changed by $(s_{n+1} - s_n)$. Equation (17.2) is rewritten as

$$\left(\frac{2s_{n+1}}{\Delta t} + Q_{o,\,n+1}\right) = \left(Q_{i,\,n} + Q_{i,\,n+1}\right) + \left(\frac{2s_n}{\Delta t} - Q_{o,\,n}\right) \tag{17.3}$$

For this method, one would require a graph $\left(\dfrac{2s_n}{\Delta t} + Q_o\right)$ versus outflow discharge rate Q_o, Fig. 17.4. This graph can be prepared by knowing the values of s and Q_o for different values of reservoir water surface elevation (Figs. 17.2 and 17.3) and suitably chosen time interval, Δt. Procedure for computation of the outflow hydrograph is as follows :

Table 17.1 Flood routing computations

Time t (hrs)	Time interval Δt (hrs)	Inflow rate at time t (m^3/s)	Average inflow rate Q_i for Δt (m^3/s)	Inflow volume during Δt ($10^4\ m^3$)	Trial reservoir level at time t (m)	Outflow rate Q_0 at time t (m^3/s)	Average outflow rate Q_0 for Δt (m^3/s)	Outflow volume during Δt ($10^4\ m^3$)	Change in storage Δs during Δt ($10^4\ m^3$)	Total storage at the end of t ($10^4\ m^3$)	Reservoir level at the end of t (m)	Remarks
1	*2*	*3*	*4*	*5*	*6*	*7*	*8*	*9*	*10*	*11*	*12*	*13*
0		100				0				11000.00		
	1		475	171.0	100.2	19.68	9.84	3.54	167.46	11167.46	100.20	Okay
1		850										
	1		1340	482.4	100.4	55.66	37.67	13.56	468.84	11636.30	100.75	High
2		1830			100.7	128.85	74.27	26.74	455.66	11623.12	100.70	Okay
	1		2310	831.6	101.0	220.00	174.43	62.80	768.80	12391.92	101.42	High
3		2790			101.4	364.43	246.64	88.79	742.81	12365.93	101.40	Okay
	1		2695	970.2	101.80	531.29	447.86	161.23	808.97	13174.90	102.00	High
4		2600			102.00	622.25	493.34	177.60	792.60	13158.53	102.00	Okay
	1		2360	849.6	102.50	869.63	745.94	268.54	581.06	13739.59	102.40	Low
5		2120			102.35	792.55	707.40	254.66	594.94	13753.47	102.35	Okay
	1		1880	676.8	103.00	1143.15	967.85	348.43	328.37	14081.84	102.55	Low
6		1640			102.60	922.32	857.44	308.68	368.12	14121.59	102.60	Okay
	1		1405	505.8	102.70	976.04	949.18	341.71	164.09	14285.68	102.70	Okay
7		1170										
	1		950	342.0	102.70	976.04	976.04	351.37	–9.37	14276.31	102.70	Okay
8		730										
	1		540	194.4	102.60	922.32	949.18	341.71	–147.31	14129.00	102.60	Okay
9		350										
	1		215	77.4	102.50	869.63	895.98	322.55	–245.15	13883.85	102.45	Low
10		80			102.45	843.67	883.00	317.88	–240.48	13888.52	102.45	Okay

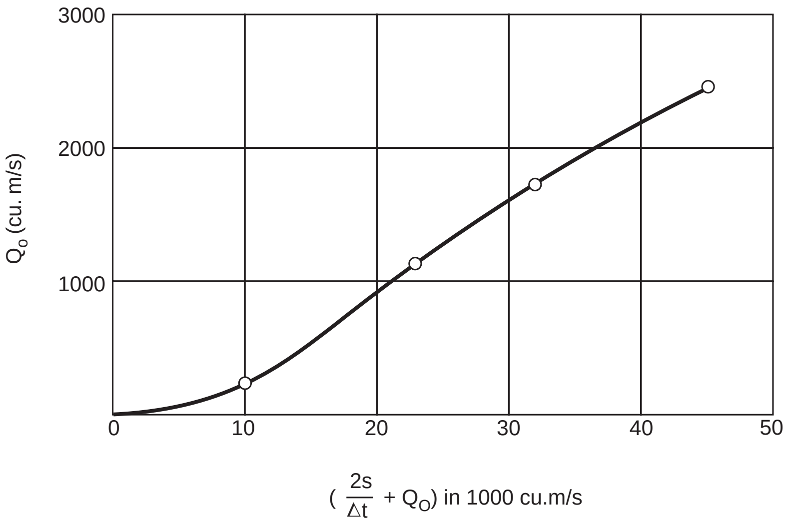


Fig. 17.4

(*i*) At the beginning of the first time interval (*i.e.*, the routing period), both s_1 and $Q_{o,1}$ are equal to zero if the storage dealt with in this interval is treated as the storage above the spillway crest level. Therefore, Eq. (17.3) yields

$$\frac{2s_2}{\Delta t} + Q_{o,2} = (Q_{i,1} + Q_{i,2})$$

Since both $Q_{i,1}$ and $Q_{i,2}$ are known (Fig. 17.1), $\frac{2s_2}{\Delta t} + Q_{o,2}$ is determined.

(*ii*) Read the value of $Q_{o,2}$ (*i.e.*, the outflow rate at the end of the first time interval, *i.e.*, $t = \Delta t$) for known value of $\frac{2s_2}{\Delta t} + Q_{o,2}$ from Fig. 17.4.

(*iii*) obtain $(\frac{2s_2}{\Delta t} - Q_{o,2})$ (*i.e.*, the value of $(\frac{2s_2}{\Delta t} - Q_o)$ at the beginning of the second time interval) which is equal to $[(\frac{2s_2}{\Delta t} + Q_{o,2}) - 2Q_{o,2}]$.

(*iv*) For the second time interval $\left(\frac{2s_3}{\Delta t} + Q_{o,3}\right) = \left(Q_{i,2} + Q_{i,3}\right) + \left(\frac{2s_2}{\Delta t} - Q_{o,2}\right)$

Since, R.H.S. is known, one can determine L.H.S., the corresponding $Q_{o,3}$ (*i.e.*, the outflow rate at the end of the second time interval *i.e.*, $t = 2\Delta t$), and the value of $\left(\frac{2s_3}{\Delta t} - Q_{o,3}\right)$ *i.e.*, the value of $\frac{2s}{\Delta t} - Q_o$ at the beginning of the third time interval ($t = 2\Delta t$).

(v) Repeat step (iii) for subsequent time steps till the end of the last time interval.

Flood routing problem of Table 17.1 has been solved using ISD method in Tables 17.2 and 17.3 and Figs. 17.4 and 17.5.

Table 17.2 : Computation of $\left(\frac{2s}{\Delta t} + Q_o\right)$ for Q_o with Δt = 3600 sec.

Elevation	*Outflow rate* (Q_o) m^3/s	*Storage* m^3	*Storage above crest level (s)* m^3	$\frac{2s}{\Delta t} + Q_o$
100	0.00	110×10^6	0	0
101	220.00	128×10^6	18×10^6	10220
102	622.25	134×10^6	24×10^6	13956
103	1143.15	149×10^6	39×10^6	22810
104	1760.00	165×10^6	55×10^6	32316
105	2459.68	186×10^6	76×10^6	44682
106	3233.33	215×10^6	215×10^6	61567

Table 17.3 : Computation of Q_o

Time (hr)	*Inflow rate*	$Q_{i,n} + Q_{i,n+1}$	$\left(\frac{2s}{\Delta t} - Q_{o,n}\right)$*	$\frac{2s}{\Delta t} + Q_{o,n+1}$	$Q_{o,n+1}$	*Reservoir level*
0	100		0	–	–	
1	850	950	930	950	10	100.127
2	1830	2680	3510	3610	50	100.372
3	2790	4620	7830	8130	150	100.775
4	2600	5390	12370	13220	425	101.551
5	2120	4720	15690	17090	700	102.163
6	1640	3760	17700	19450	875	102.510
7	1170	2810	18610	20510	950	102.652
8	730	1900	18610	20510	950	102.652
9	350	1080	17890	19690	900	102.558
10	80	430	16770	18320	775	102.315

* $\frac{2s}{\Delta t} + Q_{o,\,n+1} - 2Q_{o,\,n+1}$

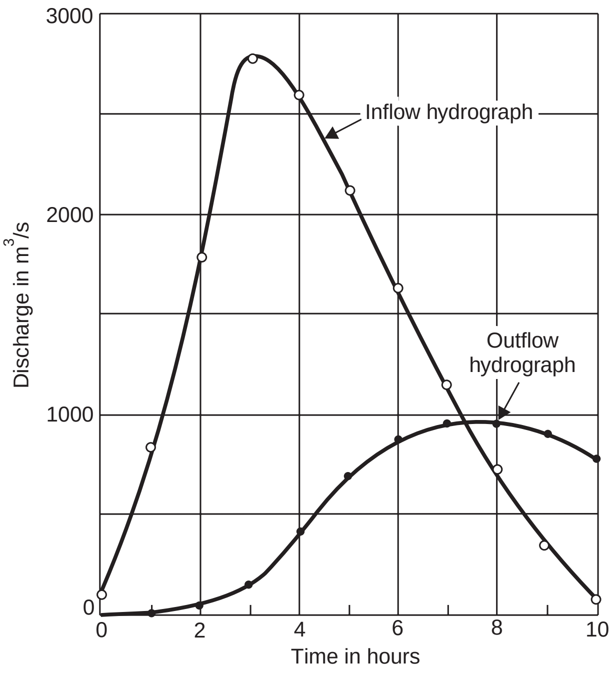


Fig. 17.5

Example 17.1 A flood enters a reservoir at 0 hours. The ordinates of the inflow hydrograph are as follows:

Hour	0	1	2	3	4	5	6	7
Discharge, Q(m^3/s)	0	100	200	300	400	500	400	300

The area of the waterspread increases linearly from 0.7×10^6 m^2 at the spillway crest level to 1.1×10^6 m^2 at 4 m above the crest level. The effective length of the spillway is 150 m and the coefficient C in the discharge equation $Q = C\,L\,H^{3/2}$ is 2.0 in S.I. units. Using a single time step from 0 to 4 hours and assuming that water is at the crest level at the beginning of the flood, find the reservoir level at 4 hours.

Solution: Let reservoir level be H (m) above the spillway crest at 4 hours from the beginning of the flood. Therefore, Eq. (17.2) reduces to

$$\frac{0+400}{2} \times (4 \times 3600) - \frac{0+300\,H^{3/2}}{2} \times (4 \times 3600)$$

$$= \left[0.7 \times 10^6 + (0.7 \times 10^6 + \left(\frac{1.1-0.7}{4}\right) \times 10^6 H\right] \frac{H}{2}$$

$$5\,H^2 + 216\,H^{3/2} + 70\,H = 288$$

$$\therefore \qquad H = 0.9925 \text{ m}$$

Example 17.2 The pond upstream of a power house may be approximated as a rectangular channel of width 80 m and length 2.0 km. The inflow as well as outflow into the pond at the beginning was 100 m^3/s. The inflow increases gradually to 200 m^3/s in two hours. Assuming that the outflow is through a sluice gate and discharge through which is expressed as $Q = 80\sqrt{H}$ (in which, H is the head of water above the sill of the sluice gate in the pond), determine the head, H_1 of water above the sill of the sluice gate at the beginning of the increased inflow and at two hours since the beginning.

Solution: Let H be the head of water at the beginning of the increased inflow. Using $Q = 80\sqrt{H}$,

$$100 = 80\sqrt{H_1}$$

$$H_1 = 1.5625 \text{ m}$$

Using Eq. (17.2),

$$\frac{100 + 200}{2} \times (2 \times 3600) - \frac{100 + 80\sqrt{H_2}}{2} \times (2 \times 3600)$$

$$= 80 \times 2000\,(H_2 - 1.5625)$$

in which, H_2 is the head of water at 2 hours since the beginning of the increased flow. Thus,

$$80\,H_2 + 144\,\sqrt{H_2} = 485$$

$$\therefore \qquad H_2 = 2.96 \text{ m}$$

17.3. COMPONENTS OF SPILLWAY

The main components of a spillway are: (*i*) control structure, (*ii*) conveyance structure, (*iii*) terminal structure, and (*iv*) entrance and exit channels.

17.3.1. Control Structure

The control structure of a spillway regulates and controls the outflow from the reservoir. It is usually located at the upstream end of the spillway and consists of some form of orifice or overflow crest. In some cases, however, the control may be at the downstream end. For example, in a 'morning glory' spillway, the downstream tunnel rather than the crest of orifice controls the flow at higher heads. In plan, the outflow crest can be straight, curved, U-shaped, semicircular, or circular. The crest can be sharp, broad, ogee-shaped, or of some other cross-section. Similarly, orifice can have different shapes and may be placed in a horizontal, vertical, or inclined position. Orifices too can be sharp-edged, round-edged or bellmouth-shaped.

17.3.2. Conveyance Structure

The outflow released through the control structure is usually conveyed to the downstream river channel through a discharge channel or waterway. Free fall spillways, however, do not require any such conveyance structure. The conveyance structure can be the downstream face of the dam (if the spillway has been constructed in the main body of the dam), or an open channel excavated along the ground surface of one of the abutments, or an underground tunnel excavated through one of the abutments. The conveyance structure too can have a variety of cross-section depending upon the geologic and topographic characteristics of the site and hydraulic requirements.

17.3.3. Terminal Structure

When water flows from the reservoir level to the downstream river level, the static energy is converted into kinetic energy which, if not properly dissipated, may cause enough scour, near the toe of the dam, that can damage the dam, spillway, and other structures. Therefore, suitable stilling basins at the downstream end of the spillway are usually provided so that the excess kinetic energy is dissipated, and the discharge into the river does not result in objectionable scour. The excess kinetic energy can be dissipated by a hydraulic jump basin or a roller bucket or some other suitable energy dissipator or absorber. In some cases, however, the overflowing water may be delivered directly to the stream if the stream bed consists of erosion-resistant bed rock. The incoming jet should always be projected some distance downstream from the end of the structure by means of such structures as flip buckets or cantilevered extensions.

17.3.4. Entrance and Exit Channel

Entrance channel conveys water from the reservoir to the control structure while the exit channel conveys flow from the terminal structure to the stream channel downstream of the dam. Entrance and exit channels are, however, not required because such spillways draw water directly from the reservoir and discharge it directly into the stream channel as, for example, in case of the overflow spillway in a concrete dam. In spillways which are placed along the abutments or located near saddles or ridges, entrance and exit channels would be needed.

17.4. TYPES OF SPILLWAY

Spillway is usually referred to as controlled or uncontrolled depending on whether spillway gates for controlling the flow have been provided or not. A free or uncontrolled spillway automatically releases water whenever the reservoir level rises above the overflow crest level. The main advantage of an uncontrolled spillway is that it does not require constant attendance and operation of the regulating devices by an operator. Besides, there are no problems related to the maintenance and repair of the devices. If it is not possible to provide a sufficiently long uncontrolled spillway crest or obtain a large enough surcharge head to meet the requirements of spillway capacity, one has to provide regulating gates. Such gates enable release of water, if required, even when the reservoir level is below the normal reservoir water surface level. Most common types of spillway are as follows:

(*i*) Free overfall (straight drop) spillway,

(*ii*) Ogee (overflow) spillway,

(*iii*) Side-channel spillway,

(*iv*) Chute (or open channel or trough) spillway,

(*v*) Shaft (or morning glory) spillway,

(*vi*) Siphon spillway,

(*vii*) Cascade spillway, and

(*viii*) Tunnel (conduit) spillway.

17.4.1. Free Overfall Spillway

As the name indicates, the flowing water drops freely from the crest of a free overfall spillway. At times, the crest is extended in the form of an overhanging lip to direct small discharges away from the downstream face of the overflow section. The underside of the falling water jet is properly ventilated so that the jet does not pulsate. Such a spillway is better suited for a thin arch dam whose downstream face is nearly vertical. Since the flowing water usually drops into the stream bed, objectionable scour may occur in some cases and a deep plunge pool may be formed. If erosion cannot be tolerated, plunge pool is created by constructing an auxiliary dam downstream of the main dam. Alternatively, a basin is excavated and is provided with a concrete apron. When tail-water depth is sufficient, a hydraulic jump forms when the water jet falls upon a flat apron. Free overfall spillways are restricted only to situations where the hydraulic drop from the reservoir level to tail-water level is less than about 6 m.

17.4.2. Ogee (Overflow) Spillway

An ogee spillway has a control weir whose profile is as shown in Fig. 17.6. The upper part of the spillway surface matches closely with the profile of the lower nappe of a ventilated sheet of water falling freely from a sharp-crested weir. The lower part of the spillway surface is tangential to the upper curve and supports the falling sheet of water. The downstream end of the spillway is in the form of a reverse curve which turns the flow into the apron of a stilling basin or into the spillway discharge channel. An ogee spillway is generally used for concrete and masonry dams. It is ideally suited to wider valleys where sufficient crest length may be provided.

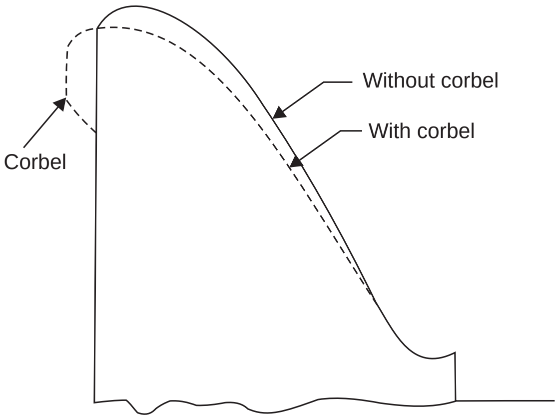


Fig. 17.6 Ogee spillway with an overhang

The profile of an ogee spillway is designed for a given design discharge (or corresponding design surcharge head or, simply, design head). When the flowing discharge equals the design discharge, the flow adheres to the spillway surface with minimum interference from the boundary surface and air has no access to the underside of the water sheet. The discharge efficiency is maximum under such condition and the pressure along the spillway surface is atmospheric. If the flowing discharge exceeds the design discharge, the water sheet tends to pull away from the spillway surface and thus produces sub-atmospheric pressure along the surface of the spillway. While negative pressure may cause cavitation and other problems, it increases the effective head and increases the discharge. On the other hand, positive hydrostatic pressure will occur on the spillway surface, if the flowing discharge is less than the design discharge.

Model tests have indicated that the design head may be safely exceeded by about 50% beyond which cavitation may develop. Therefore, spillway profle may be designed for 75% of the peak head for the maximum design flood.

An upstream overhang, known as corbel, is added to the upstream face of the spillway as shown in Fig. 17.6. The effect of the corbel is to shift the nappe (and, hence, the spillway profile) backward which results in saving of concrete. If the height of the vertical face of the corbel is kept more than 0.3 times the head over the crest, the discharge coefficient of the spillway will be practically the same as it would be if the vertical face of the corbel were to extend to the full height of the spillway.

The shape of an ogee crest, approximating the profile of the underside of a water jet flowing over a sharp-crested weir, depends on: (*i*) the head, (*ii*) the inclination of the upstream face of the overflow section, and (*iii*) the height of the overflow section above the floor of the entrance channel which affects the velocity of approach to the crest. A simple shape of an ogee crest suitable for dams with vertical upstream face is shown in Fig. 17.7. It consists of an upstream surface shaped as an arc of a circle upto the apex of the crest followed by a parabolic downstream surface. This type of ogee crest is suitable for preliminary estimate and for final designs when a more refined shape is not required (2).

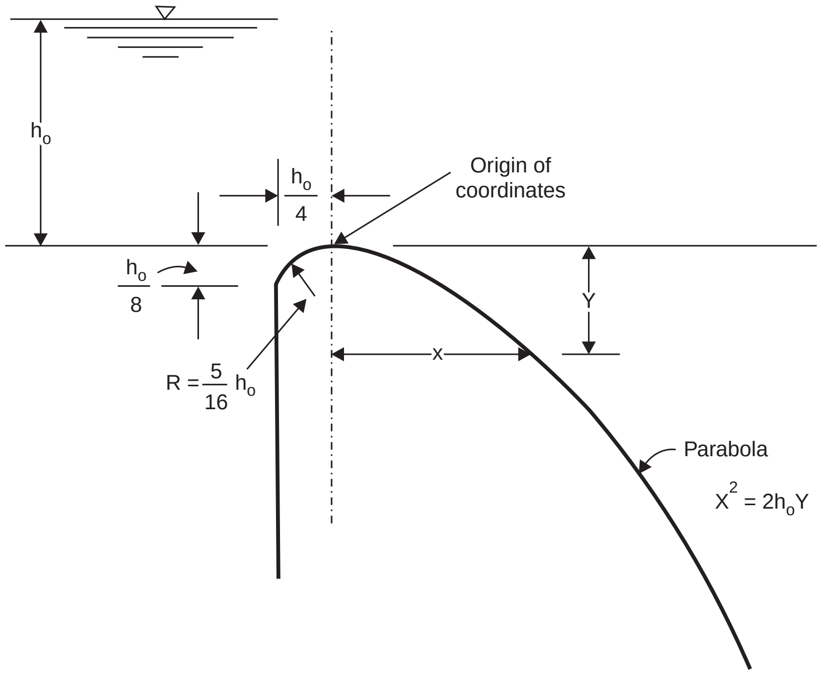


Fig. 17.7 Ogee crest for spillway with vertical upstream face

An extensive study of crest shapes has been made by USBR (3). These crest shapes can be represented by the profile shown in Fig. 17.8 (*a*) and defined with respect to the coordinate axes at the apex of the crest. The part of the profile upstream of the apex of the crest consists of either a single curve and a tangent or a compound circular curve. The profile downstream of the apex of the crest is defined by the equation

$$\frac{y}{H_0} = -k\left(\frac{x}{H_0}\right)^n \tag{17.4}$$

in which, H_0 is the total head on the crest including the velocity of approach and the constant k and n depend on: (*i*) the inclination of the upstream face, and (*ii*) the velocity of approach as has been shown in Fig. 17.8 (*b*) and 17.8 (*c*). Similarly, the value of X_c, Y_c, R_1, and R_2, shown as elements of the crest profile in Fig. 17.8 (*a*), can be obtained from Fig. 17.8(*d*), 17.8 (*e*), and 17.8 (*f*).

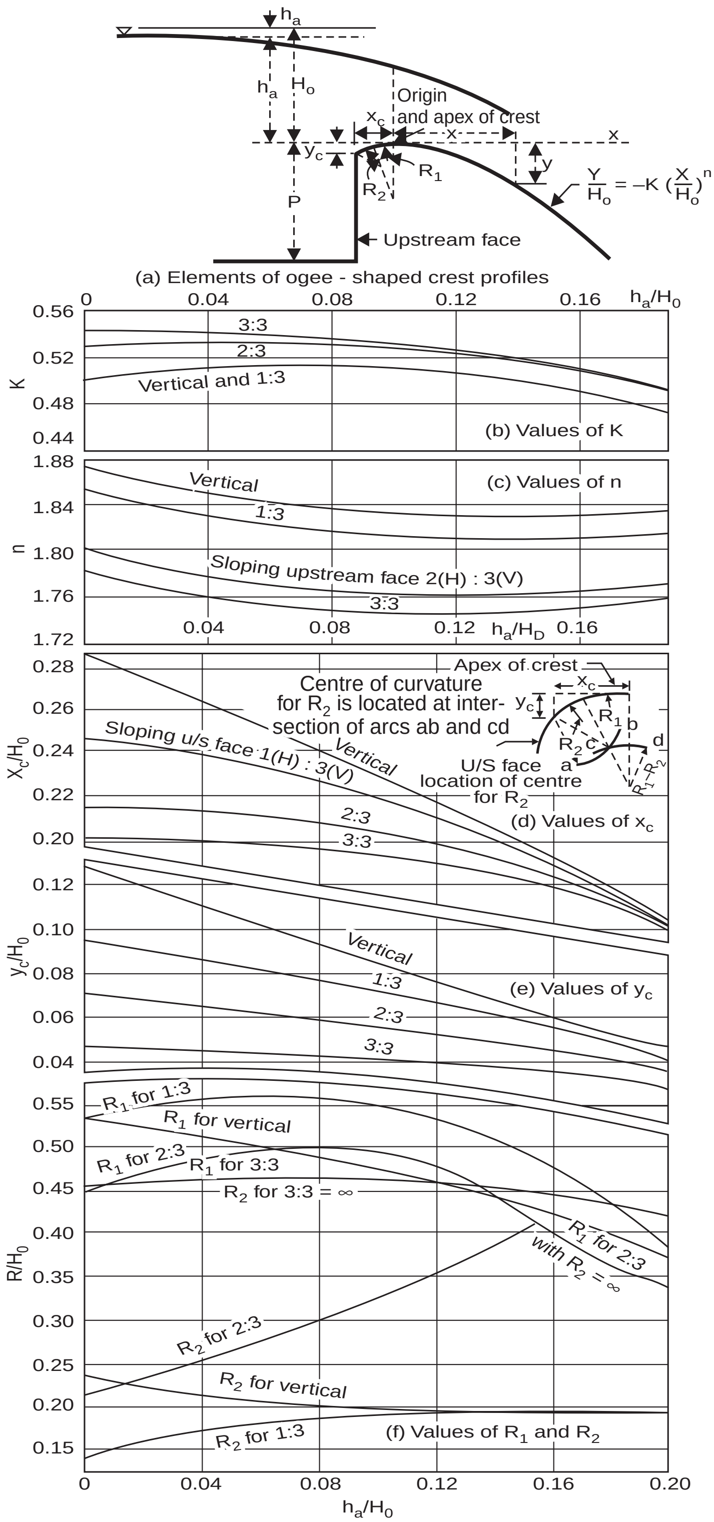


Fig. 17.8 Parameters for ogee-shaped crest profile

The discharge Q flowing over an ogee crest of effective length L can be obtained from the formula

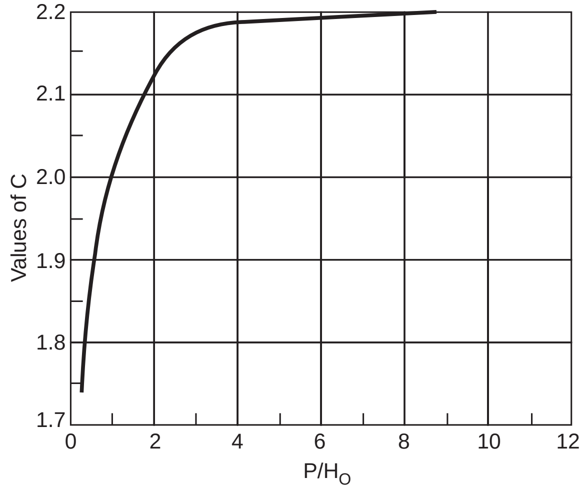
$$Q = CLH_o^{3/2} \tag{17.5}$$

in which, C is a variable coefficient of discharge which depends on: (*i*) depth of approach [Fig. 17.9 (*a*)], (*ii*) slope of the upstream face [Fig. 17.9 (*b*)], (*iii*) position of the downstream apron [Fig. 17.9 (*c*)], (*iv*) the downstream tail-water level, [Fig. 17.9 (*d*)], and (*v*) the relation between the actual crest shape (corresponding to the design head) and the ideal nappe shape (corresponding to the actual head of flow) [Fig. 17.9 (*e*)]. The effect of the downstream apron and tail-water level would generally be felt when ogee crest is being used as control structure for a side-channel spillway and the tail-water level is high enough to affect the discharge, *i.e.*, the crest is submerged. For usual ogee spillways, this situation would not arise.

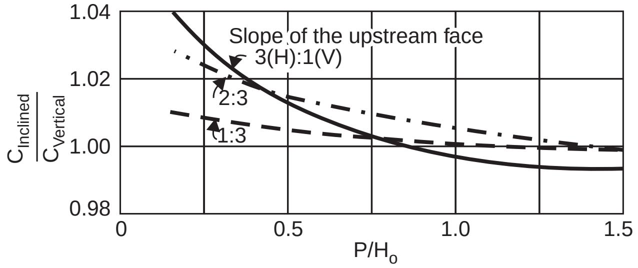
The effective length of the crest L will be less than the total length of the crest on account of side contractions whenever crest piers are provided. The effective length L and total length L' of a crest are related as follows (1):

$$L = L' - 2\,(N'K_p + K_a)\,H_o \tag{17.6}$$

Here, N' is the number of piers, and K_p and K_a are, respectively, pier and abutment coefficients. Values of K_p vary between zero (for pointed nose piers) and 0.02 (for square-nosed pier with suitable rounded corners). Values of K_a vary between zero and 0.2.

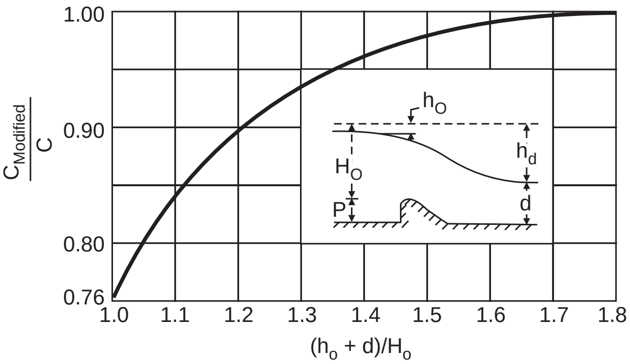


(a) Variation of coefficient of discharge for ogee-shaped crest with vertical uptream face

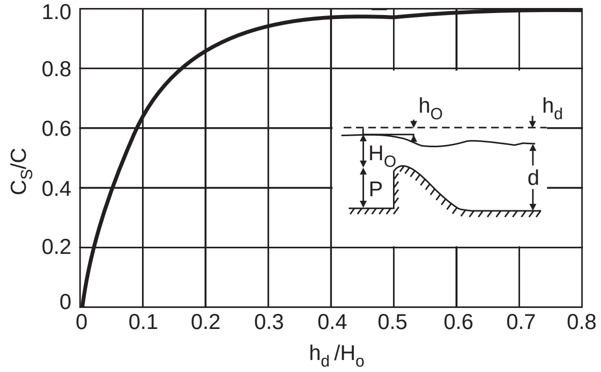


(b)- Coefficient of discharge for ogee-shaped crest with sloping uptream face

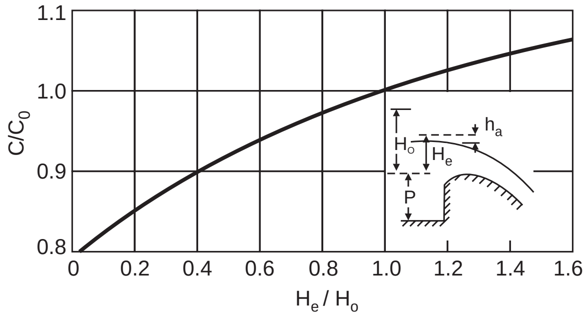
Fig. 17.9 (a and b). Variations of coefficient of dischrge for ogee spillway (*Contd...*)



(c) Ratio of discharge coefficient due to apron effect



(d) Ratio of discharge coefficient due to tailwater effect



(e) Coefficient of discharge for H_o other than the design head

Fig. 17.9 (c, d and e) Variations of coefficient of discharge for ogee spillway (4)

Example 17.3 Obtain USBR profile for an ogee spillway with the following data:

Design discharge = 13875 m^3/s

Crest length of spillway = 183 m

Crest level of spillway = 203.34 m

River bed level = 166.50 m

Solution: Assuming C = 2.20 in Eq. (17.5), one obtains

$$H_o = \left[\frac{13875}{183 \times 2.20}\right]^{2/3} = 10.59 \text{ m}$$

and $$P = 203.34 - 166.50 = 36.84 \text{ m}$$

∴ $$\frac{P}{H_o} = 3.48$$

From Fig. 17.9 (a), C = 2.18

With C = 2.18, Eq. (17.5) yields H_0 = 10.655 m

Design head for spillway profile = 0.75 × 10.655 = 7.99 m

$$\simeq 8.00 \text{ m (say)}$$

∴ $$\frac{P}{H_o} = \frac{36.84}{8.00} = 4.605$$

$$\text{Approach velocity, } v_a = \frac{13875}{183\,(36.84 + 8.00)} = 1.69 \text{ m/s}$$

∴ $$h_a = 0.15 \text{ m} \quad \text{and} \quad h_a/H_o = 0.019$$

From Figs. 17.8 (b) and (c), k = 0.504 and n = 1.861

Hence, the profile of the ogee spillway, [Fig. 17.8 (a)] is given by

$$\frac{y}{8.00} = -\,0.504\left[\frac{X}{8.00}\right]^{1.861}$$

17.4.3. SIDE-CHANNEL SPILLWAY

The control weir of a side-channel spillway (Fig. 17.10) is located alongside and approximately parallel to the upstream portion of the spillway discharge channel which itself may be either an open channel, a closed conduit, or an inclined tunnel. The spillway discharge flows over the weir crest and falls into a narrow trough (*i.e.*, upstream of the discharge channel) and takes an approximately 90°-turn before continuing into the spillway discharge channel. The control structure, in plan, may be straight, curved, semi-circular or U-shaped. The overflow section may be broad-crested instead of ogee-shaped.

One should note that when the flow in the side channel trough is subcritical, the incoming flow from the control structure (*i.e.*, the overflow crest) will not cause high transverse velocities because of the low drop due to relatively higher depth of flow in the trough. This would effect good diffusion and intermingling of the incoming flow with the trough water due to relatively low velocities of both the incoming flow and the flow in the trough. Therefore, there would be comparatively smooth flow in the side channel trough. However, when the flow in the side channel trough is supercritical, the flow velocities in the trough would be high and the depth of flow small, causing the incoming flow to have a relatively higher drop. Therefore, the intermixing of the high-energy transverse flow with the trough stream will be rough and turbulent producing violent wave action causing vibrations. Therefore, the flow in the side channel trough should be maintained at subcritical condition for good hydraulic performance.

Moreover, the amount of excavation would also increase for larger bed widths. While the flow in the side channel trough should preferably be subcritical, the flow in the discharge channel is supercritical and a control section downstream of the trough is provided by either constricting the channel width or raising the channel bottom.

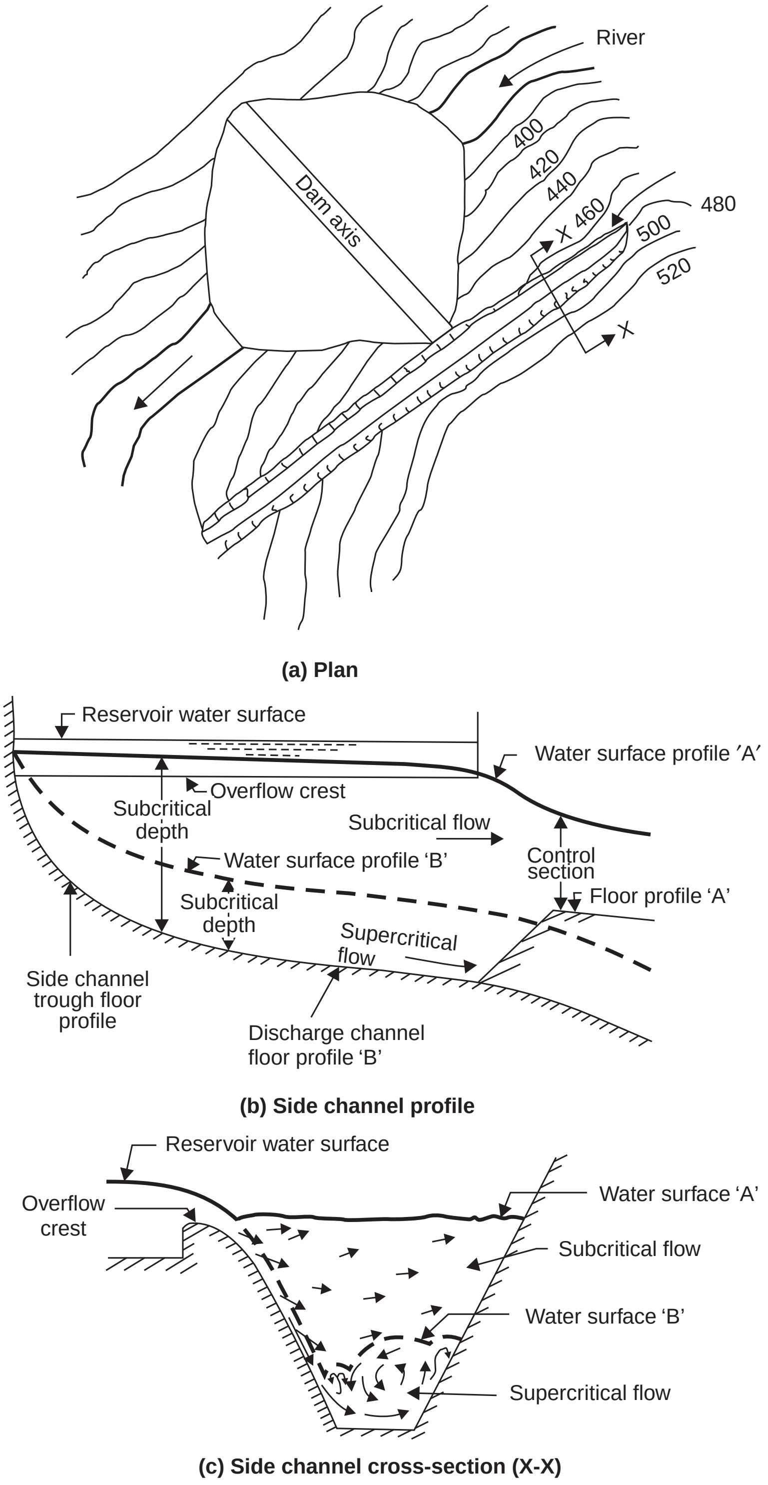


Fig. 17.10 Side-channel spillway

Because of spatial flow conditions, the depth of flow in the side-channel trough would never be the same at different sections. For any short reach of channel Δx, the change in water surface Δy can be determined by either of the following equations (4):

$$\Delta y = \frac{Q_1(v_1 + v_2)}{g(Q_1 + Q_2)}\left[(v_2 - v_1) + \frac{v_2(Q_2 - Q_1)}{Q_1}\right] \tag{17.7}$$

$$\Delta y = \frac{Q_2(v_1 + v_2)}{g(Q_1 + Q_2)}\left[(v_2 - v_1) + \frac{v_1(Q_2 - Q_1)}{Q_2}\right] \tag{17.8}$$

Here, Q_1 and v_1 are the values of discharge and velocity, respectively, at the beginning of the reach, and Q_2 and v_2 are those values at the end of the reach. For free flow conditions, the behaviour of a side-channel spillway is similar to that of an overflow spillway and is dependent on the profile of the weir crest. For larger discharges, however, the flow over the crest may be submerged and the flow conditions will then be governed by the conditions in the side-channel trough. A side-channel spillway is an ideal choice: (*i*) for earth or rockfill dams in narrow canyons and for situations where direct overflow is not permissible, (*ii*) where the space required for a chute spillway of adequate crest length is not available, or (*iii*) when a long overflow crest is required in order to limit the surcharge head for the design inflow flood. Because of the turbulences and vibrations inherent in side channel flow, a side channel spillway is generally not considered except when a strong foundation (such as rock foundation) exists.

Design of side channel trough involves computation of water surface profile starting from the control section (rectangular in shape and at which the critical depth, velocity and the velocity head are known for given discharge) to the upstream end of the side channel trough. The trough is usually trapezoidal with side slopes of $0.5H{:}1V$ and relatively flat bed slope that would provide larger depth and smaller velocities to insure better intermixing of flows in the initial reach of the trough. A cross-section with minimum width – depth ratio will result in the best hydraulic performance (5). However, minimum bed width (say, about 3 m) is required to avoid construction difficulties due to confined working space. There would be a transition between the downstream end of the trough and the control section. The head loss in transition (to include losses due to contraction and friction, and also losses due to diffusion of flows in the trough) is assumed to be equal to 0.2 times the difference in velocity heads between the ends of the transition. The flow characteristics (depth and velocity or velocity head) at the downstream end of the trough (*i.e.*, upstream end of the transition) are obtained by solving Bernoulli's equation. The equation will have to be solved by trial and error. For this, assume a suitable value of the depth of flow at the upstream end of the transition and the corresponding velocity head. If these values satisfy the Bernoulli's equation (applied for the two end sections of the transition), one has obtained the flow characteristics at the upstream end of the transition. Otherwise, one has to assume another trial value and repeat the computations till the Bernoulli's equation is satisfied. Thereafter, the water surface profile along the side channel trough can be determined using either Eq. (17.7) or Eq. (17.8). The channel profile and the water surface profiles are, then, plotted relative to the crest (of the control structure) and the reservoir water level. The maximum submergence at the upstream end of the trough that can be tolerated is about two-third the head over the control structure. If the maximum water surface level in the side channel trough results in submergence more than the permissible value, the end of the side channel trough will have to be lowered.

The control structure of a side channel spillway generally consists of an ogee crest which is designed by the method described in section 17.4.2. Flow in the discharge channel downstream from the control will be the same as that in an ordinary channel or chute spillway.

17.4.4. Chute Spillway

In a chute (or trough) spillway, the spillway discharge flows in an open channel (named as 'chute' or 'trough') right from the reservoir to the downstream river. The open channel can be located either along the abutment of the dam or through a saddle, (Fig. 17.11). The channel bed should always be kept in excavation and its side slope must be designed to be stable with sufficient margin of safety. As far as possible, bends in the channel should be avoided. If it becomes necessary to provide a bend, it should be gentle. The spillway control structure can be an overflow crest, or a gated orifice or some other suitable control device. The control device is usually placed normal or nearly normal to the axis of the chute. The simplest form of a chute spillway is an open channel with a straight centre line and constant width. However, often the axis of either the entrance channel or the discharge channel is curved to suit the topography of the site. The flow condition varies from subcritical upstream of the controlling crest to critical at the crest and supercritical in the discharge channel. The chute spillway is ideally suited with earthfill dams because of: (*i*) simplicity of their design and construction, (*ii*) their adaptability to all types of foundation ranging from solid rock to soft clay, and (*iii*) overall economy usually obtained by the use of large amounts of spillway excavation for the construction of embankment. The chute spillway is also suitable for concrete dams constructed in narrow valleys across a river whose bed is erodible for which the ogee spillway becomes unsuitable.

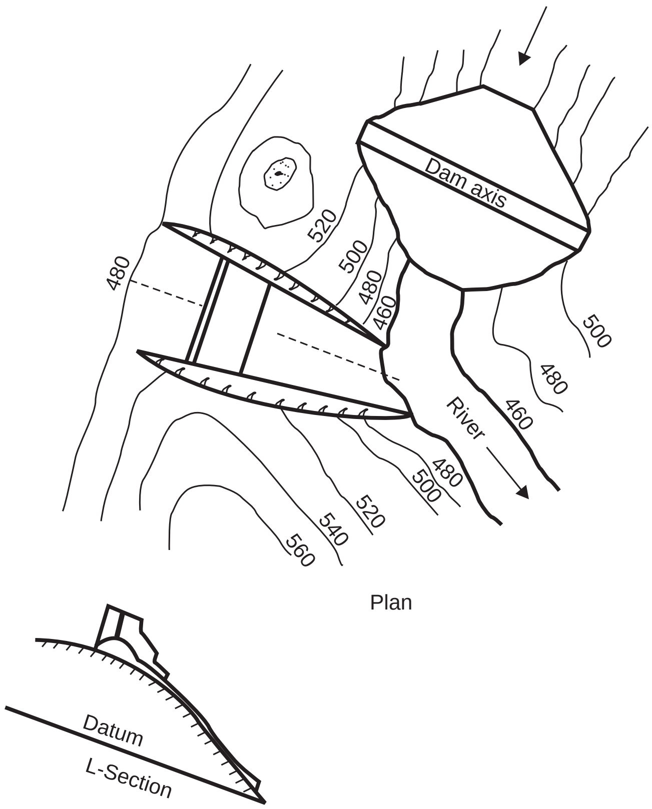


Fig. 17.11 Chute spillway

17.4.5. Shaft Spillway

In a shaft spillway (Fig. 17.12) water enters a horizontal crest, drops through a vertical or sloping shaft, and then flows to the downstream river channel through a horizontal or nearly horizontal conduit or tunnel. A rock outcrop projecting into the reservoir slightly upstream of the dam would be an ideal site for shaft spillway. Depending upon the level of the rock outcrop and the required crest level, a spillway may have to be either constructed or excavated. The diversion tunnels, if used for river diversion purposes during construction, can be utilised for discharge tunnels of the spillway. Radial piers provided on the spillway crest ensure radial flow towards the spillway and also provide support to the bridge which would connect the spillway with the dam or the surrounding hill.

A shaft spillway with a funnel-shaped inlet is called a "morning glory" or "glory hole" spillway. One of its distinguishing characteristics is that near maximum capacity of the spillway is attained at relatively low heads. Therefore, a shaft spillway is ideal when maximum spillway discharge is not likely to be exceeded. Because of this feature, however, the spillway becomes unsuitable when a flow larger than the selected design flow occurs. This disadvantage can be got rid of by providing an auxiliary or emergency spillway and using the shaft spillway as service spillway.

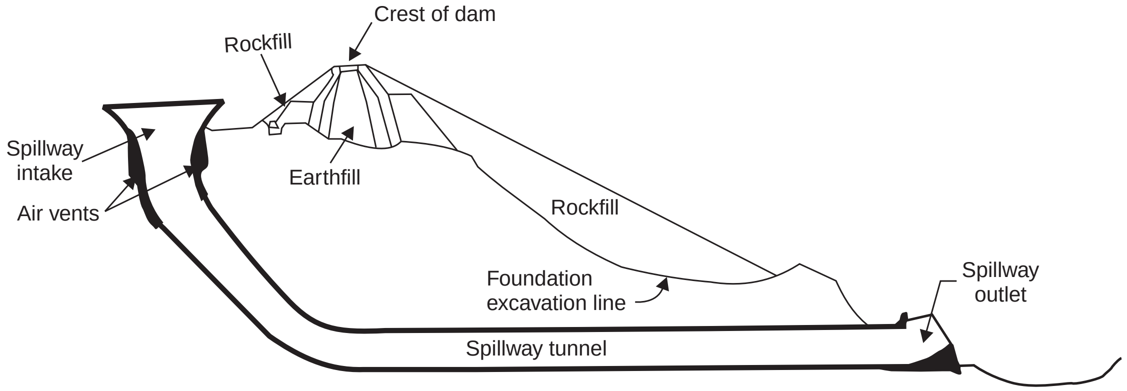


Fig. 17.12 Shaft spillway

Depending upon the type of crest, the shaft spillway can be either standard-crested or flat-crested (Fig. 17.13). In a standard-crested spillway, the water begins its free fall immediately upon leaving the crest whereas in the flat-crested spillway water approaches the crest on a flat slope before beginning its free fall. The standard-crested spillway would have a smaller diameter crest since its coefficient of discharge is greater than that of a flat crest. Therefore, if the shaft spillway is to be constructed in the form of a tower, it would be economical to have a standard-crested spillway. However, a flat-crest shaft spillway has a smaller funnel diameter and is, therefore, more advantageous when the spillway is to be excavated in rock. The design of a standard-crested shaft spillway has been discussed here.

The design of a standard-crested shaft spillway involves the determination of the funnel radius, R, and the head over the theoretical sharp crest, H, for known discharge, Q and the allowable maximum head, h on the spillway crest (Fig. 17.14). The method given by Creager, *et al.* (6) is valid for negligible velocity of approach and involves trial at two stages and is based on the following equations:

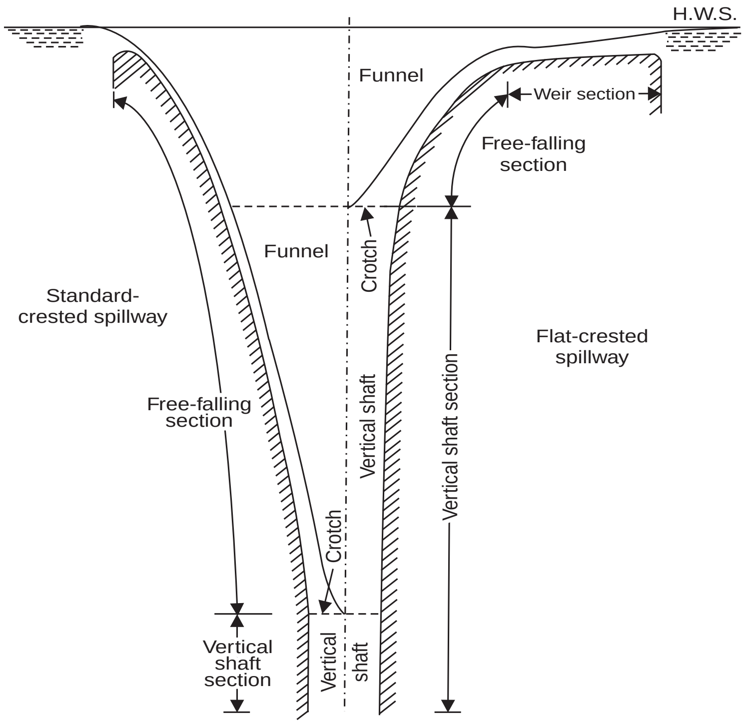


Fig. 17.13 Standard-crested and flat-crested profiles for shaft spillway

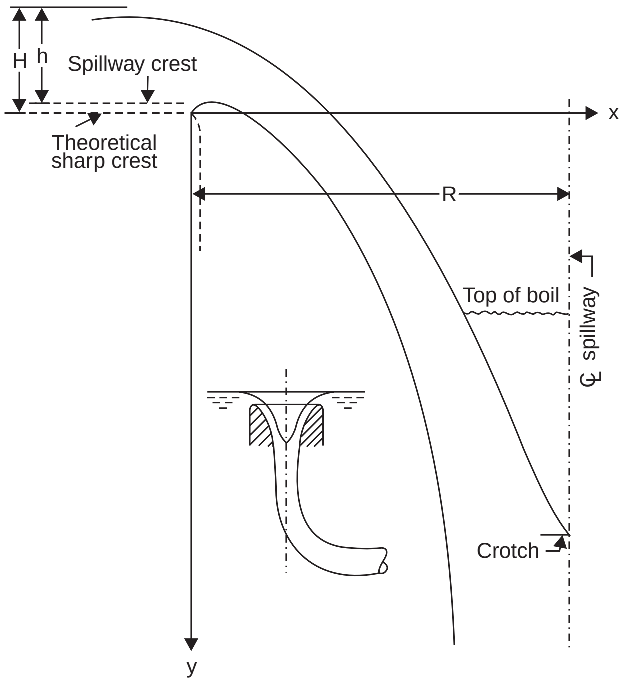


Fig. 17.14 Jet profile over standard-crested shaft spillway

$$Q = 2\pi RC_1\sqrt{g}H^{3/2} \tag{17.9}$$

$$\frac{r}{H} = 0.11 - 0.10\frac{H}{R} \tag{17.10}$$

$$H = h + r \tag{17.11}$$

In these equations, C_1 is the coefficient of discharge which is related to H/R as shown in Fig. 17.15, and r is the rise of the lower nappe above the theoretical sharp crest. The different steps involved in the trial method are as follows:

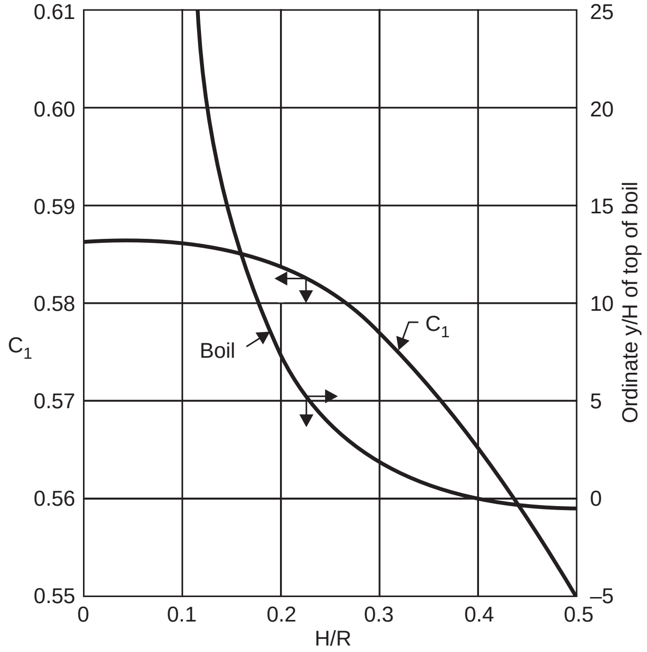


Fig. 17.15 Variations of C_1 and the ordinate y/H of the top of boil for standard-crested shaft spillway (6)

(*i*) Assume some suitable values of H and R and, hence, H/R, and obtain the value of C_1 from Fig. 17.15.

(*ii*) Determine the discharge intensity, q per unit length of the crest, *i.e.*,

$$q = C_1\sqrt{g}H^{3/2} \tag{17.12}$$

(*iii*) Obtain the required radius R from

$$R = \frac{Q}{2\pi q} \tag{17.13}$$

Compare this value of R with the assumed value of R in step (*i*). If these two values do not match, assume another value of R for the same assumed value of H and repeat the procedure until the value of R obtained in step (*iii*) matches with the assumed value of R.

(*iv*) Determine r from Eq. (17.10) and then obtain h from Eq. (17.11). If this value of h does not agree with the given value of h, one has to assume another value of H, and repeat steps (*i*) to (*iv*) until the agreement is reached.

Vittal (7) has obtained a direct solution for R and H by rewriting Eqs. (17.9) to (17.11) as follows:

From Eq. (17.9),

$$Q_* = \frac{Q}{g^{1/2} h^{5/2}} = 2\pi \frac{R}{h} C_1 \left(\frac{H}{h}\right)^{3/2} \tag{17.14}$$

Using Eq. (17.11), Eq. (17.10) can be rewritten in the following two forms:

$$\frac{r}{H} = \frac{H-h}{H} = 1 - \frac{h}{H} = 0.11 - 0.10\frac{H}{R}$$

$$\therefore \quad \frac{h}{H} = 0.89 + 0.10\frac{H}{R} \tag{17.15}$$

and

$$\frac{r}{H} = \frac{H-h}{H} = \frac{\frac{H}{R} - \frac{h}{R}}{\frac{H}{R}} = 0.11 - 0.10\frac{H}{R}$$

$$\therefore \quad \left(\frac{H}{R}\right)^2 + 8.9\frac{H}{R} - 10\frac{h}{R} = 0 \tag{17.16}$$

From Fig. 17.15, and Eqs. (17.15) and (17.16), one obtains the following functional relationships:

$$C_1 = f_1\left(\frac{H}{R}\right)$$

$$\frac{h}{H} = f_2\left(\frac{H}{R}\right)$$

$$\frac{H}{R} = f_3\left(\frac{h}{R}\right)$$

Using the above functional relationships, Vittal (7) obtained the following functional relation:

$$C_1\left(\frac{H}{h}\right)^{3/2} = f_4\left(\frac{h}{R}\right) \tag{17.17}$$

Actual relationship of Eq. (17.17) can be obtained by obtaining C_1 from Fig. 17.15, the values of $\frac{h}{H}$ from Eq. (17.15), and $\frac{h}{R}$ from Eq. (17.16) for different values of $\frac{H}{R}$ ranging from 0 to 0.5. One can, therefore, prepare a curve of $C_1\left(\frac{H}{h}\right)^{3/2}$ versus $\left(\frac{h}{R}\right)$. Vittal (7) obtained the following equation for this curve:

$$C_1\left(\frac{H}{h}\right)^{3/2} = 0.6988 - 0.0882\left(\frac{h}{R}\right) - 0.296\left(\frac{h}{R}\right)^2 \tag{17.18}$$

On combining Eq. (17.18) with Eq. (17.14), and solving the resulting quadratic equation, one obtains,

$$\left(\frac{R}{h}\right)^2 - (0.2280\, Q_* + 0.1263)\frac{R}{h} - 0.3861 = 0$$

$$\therefore \qquad \frac{R}{h} = (0.1140\, Q_* + 0.0632)\left[1 + \sqrt{1 + \frac{1.4432}{(0.2280\, Q_* + 0.1263)^2}}\right] \tag{17.19}$$

For large values of Q_* (say, greater than 25), Eq. (17.19) can be approximated to

$$\frac{R}{h} = 0.2280\, Q_* + 0.1264 \tag{17.20}$$

For known Q and h and, hence, Q_*, one can easily determine $\frac{R}{h}$ (and, hence, R) from one of the Eqs. (17.19) and (17.20). Using Eq. (17.16) one can determine $\frac{H}{R}$ and, hence, H. For example, values of Q and h equal to 851.2 m^3/s and 3.05 m, respectively, yield

$$Q_* = 16.73$$

$$\frac{R}{h} = 4.03 \quad \therefore \quad R = 12.29 \text{ m}$$

$$\frac{H}{R} = 0.27 \quad \therefore \quad H = 3.325 \text{ m}$$

The profile of the underside of the nappe over the circular sharp-crested weir can be determined from Fig. 17.16 which enables computation of x for a given value of y, and already computed R and H. If R_0 is the radius of the lower nappe at any given elevation y, then

$$R_0 = R - x$$

And x_0, representing the value of x for the upper side of the nappe at a given value of y, is obtained from (6)

$$x_0 = R - \sqrt{{R_0}^2 - \frac{Q}{\pi\sqrt{2g(y + 1.269\, H)}}} \tag{17.21}$$

Here, y + 1.269 H is the head available for vertical velocity. Proceeding in this manner, one can compute the value of x and x_0 for different values of y until x_0 equals R at which value of y the horizontal velocity ceases and its energy gets converted into a 'boil' as shown in Fig. 17.14. The ordinate $\frac{y}{H}$ of the top of the boil can be computed from the curve shown in Fig. 17.15. The point at which x_0 becomes equal to R is usually known as 'crotch'. The above analysis does not include the friction loss as it is difficult to be considered and is within permissible limits for the accuracy desired (6).

The diameter of the vertical shaft below the crotch continues to decrease until the size becomes such that the discharge, Q can be carried according to the head available. The radius of the transition shaft, R' at a given elevation y is obtained from

$$Q = \pi\, R'^2 \sqrt{2gh_v} \tag{17.22}$$

in which, $\qquad h_v = y + 1.269\, H - h_{L1} - h_{L2}$

where, h_{L1} and h_{L2} are the head losses due to friction, respectively, from the crest to the crotch and from the crotch to the elevation under consideration.

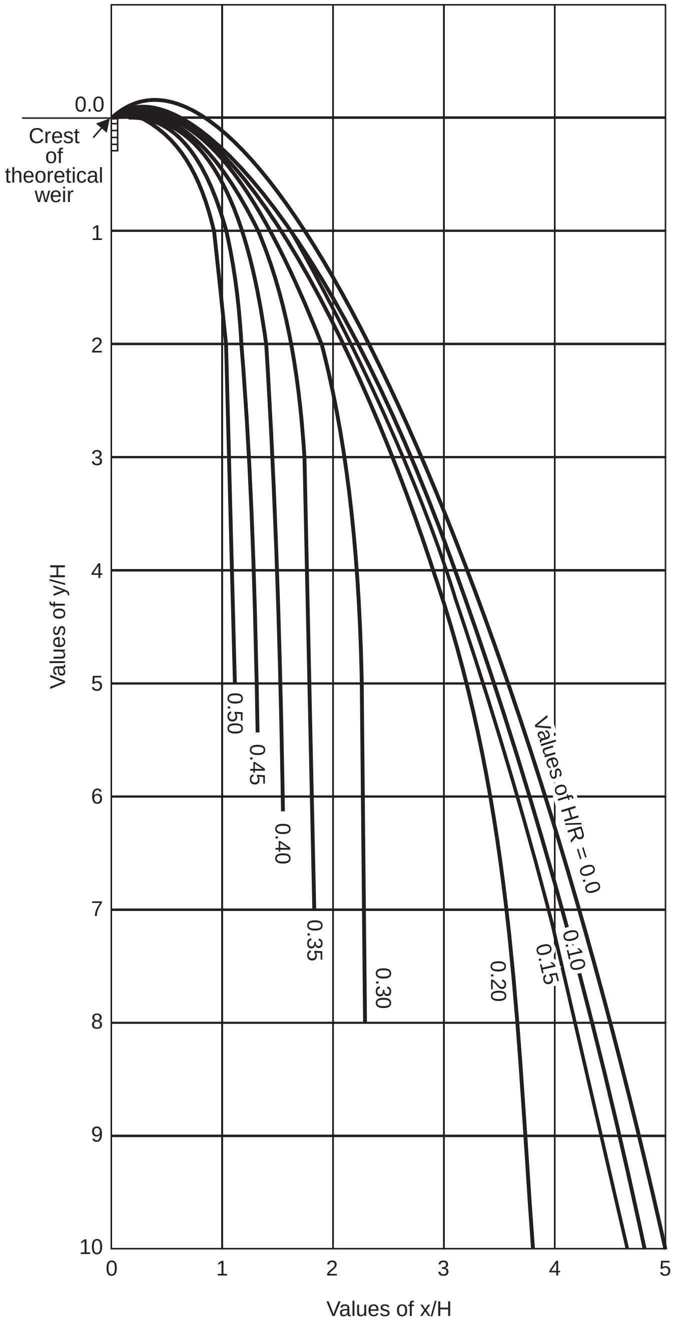


Fig. 17.16 Profile of the undernappe of the standard-crested shaft spillway (6)

17.4.6. Siphon Spillway

A siphon spillway (Fig. 17.17) is essentially a closed conduit system which uses the principle of siphonic action. The conduit system is the shape of an inverted U of unequal legs with its inlet end at or below normal reservoir storage level. When the reservoir water level rises above the normal level, the initial flow of water is similar to the flow over a weir. When the air in the bend has been exhausted, siphonic action starts and continuous flow is maintained until air enters the bend. The inlet end of the conduit is placed well below the normal reservoir water level to prevent ice and drift from entering the conduit. Therefore, once the siphonic action starts, the spillway continues to discharge even after the reservoir water level falls below the normal level. As such, a siphon-breaking air vent is always provided so that siphonic action can be broken once the reservoir water level has been drawn down to the normal level in the reservoir. Siphon spillways can be either constructed of concrete or formed of steel pipe. The thickness of the wall of the siphon structure should, however, be sufficiently strong to withstand the negative pressures which develop in the siphon. Pressure at the throat section (*i.e.*, section 2) can be determined by the use of Bernoulli's equation. Thus,

$$\frac{p_2}{\rho g} + \frac{{v_2}^2}{2g} + H_1 = \frac{{v_3}^2}{2g} + h_L$$

where, h_L is the head loss between sections 2 and 3. Therefore,

$$\frac{p_2}{\rho g} = -H_1 + \left(\frac{{v_3}^2}{2g} - \frac{{v_2}^2}{2g}\right) + h_L \tag{17.23}$$

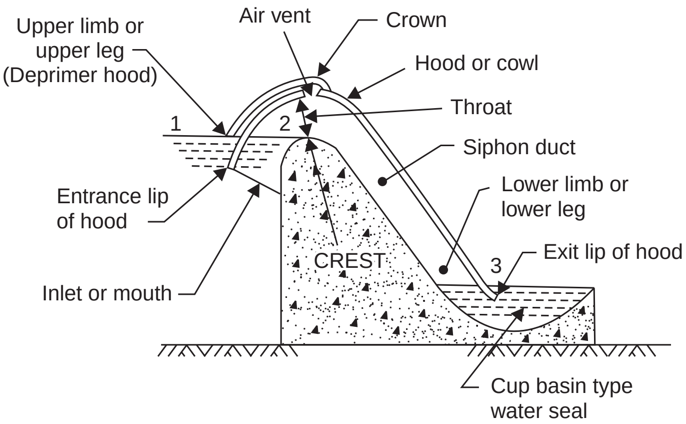


Fig. 17.17 Siphon spillway

If the cross-sectional areas at sections 2 and 3 are the same, $v_3 = v_2$ and $h_L < H_1$, the pressure at the throat is always negative. Besides, the pressure distribution is non-uniform due to the curvature of streamlines and the pressure is lower at the crest and higher at the crown. Keeping these in mind, the total drop of siphon structure should be limited to about 6 m so that the negative pressures do not reach cavitation pressures.

To expedite the priming of siphon spillway, some kind of priming device is always used. The priming device could be a joggle (or step), a steel plate or some other suitable arrangement. A joggle, [Fig. 17.18 (*a*)] deflects the sheet of water flowing over the crest of the spillway to strike against the inner side of the hood thus forming a water seal which results in early priming of the spillway. The presence of the step, however, offers resistance to flow when the siphon ducts run full. A steel plate hinged at the spillway surface [Fig. 17.18 (*b*)] will also act as a priming device. Once the siphon duct starts running full, the plate is pressed downwards and is flush with the spillway surface so that there is no obstruction to the flow.

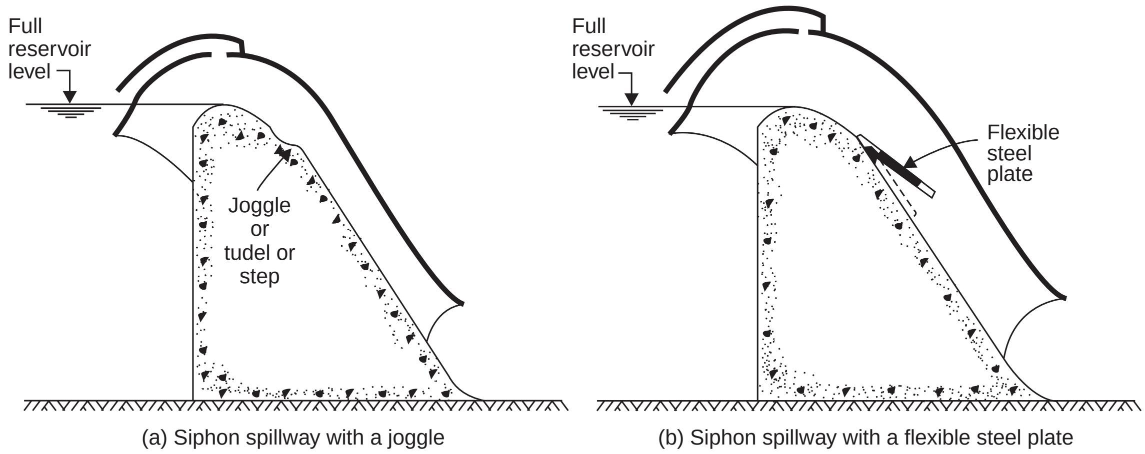


Fig. 17.18 Priming of siphon spillway

If the permissible negative head is h_0 and the radii of curvature of the crest and crown are r_0 and R_0, respectively, the unit discharge, q, through a siphon spillway, shown in Fig. 17.17, can be worked out on the assumption of free vortex conditions which are approximately obtained. If v is the velocity of flow at radius r, then in a free vortex flow

$$vr = v_0 \, r_0$$

where, v_0 is the velocity at radius $r = r_0$ (*i.e.*, the crest) and equals $\sqrt{2gh_0}$.

$$\therefore \qquad v = \frac{r_0}{r}\sqrt{2gh_0}$$

$$\text{Thus,} \qquad q = \int_{r_0}^{R_0} v\,dr$$

$$= \int_{r_0}^{R_0} r_0\sqrt{2gh_0}\,\frac{dr}{r}$$

$$\text{or} \qquad q = r_0\,\sqrt{2gh_0}\,\ln\frac{R_0}{r_0} \tag{17.24}$$

The main advantages of siphon spillway are: (*i*) its automatic operation without any mechanical device, and (*ii*) its ability to pass higher discharges at relatively low surcharge head resulting in lower height of dam as well as less surrounding area to be acquired for reservoir submergence.

Besides being an expensive structure and of limited capacity, it has a serious disadvantage due to the occurrence of sudden surges and stoppages of outflow as a result of erratic siphonic action, thus causing severe fluctuations in the downstream river stage. A minor crack in the cover of the siphon would interfere with the siphon. Therefore, siphon spillway is usually constructed in batteries so that the entire spillway is not affected even if cracks have developed in one or few units. In addition, the structure and foundation have to be strong enough to resist vibration stresses. Further, there exists a possibility of clogging of the siphon due to debris and floating material. Like other types of closed conduit spillway, a siphon spillway too is incapable of handling flows appreciably greater than the designed capacity. As such, siphon spillway, whenever provided, is used as a service spillway in conjunction with an auxiliary or emergency spillway. In canyons of small width and small flood discharge, the suitability of a siphon spillway should always be examined.

17.4.7. Cascade Spillway

In case of very high dams the kinetic energy at the toe of the dam will be very high and the tail-water depth in the river may not be adequate for a single-fall hydraulic jump or roller bucket stilling basin. Narrow and curved canyons consisting of fractured rock would not be suitable for trajectory buckets. In such situations, especially for high earth and rockfill dams for which spillway is a major structure, possibility of providing a cascade of falls with a stilling basin at each fall (Fig. 17.19) must be considered. The cascade spillway (2, 8) is likely to be an ideal choice for a high rockfill dam for which the material has been obtained from a quarry located downstream of the dam so that the flood waters may be discharged over the quarry face. As the quarry would usually be excavated in benches, they may as well form the steps in the cascade. A cascade spillway has been planned at the proposed 218 m high Tehri dam on the Bhagirathi river in the Ganga valley of the central Himalayas. At Darmouth dam in Australia, the spillway to pass 2700 m^3/s of flood discharge is an unlined cascade in granite. The benches of the cascade will be 5 m high and of varying widths to suit the topography of the site. Although, a cascade spillway would be an attraction during floods, it may not be always acceptable for environmental reasons which demand that the quarries be always located upstream of the dam and below the normal water surface level of the reservoir so as to cause minimum disfigurement of the land.

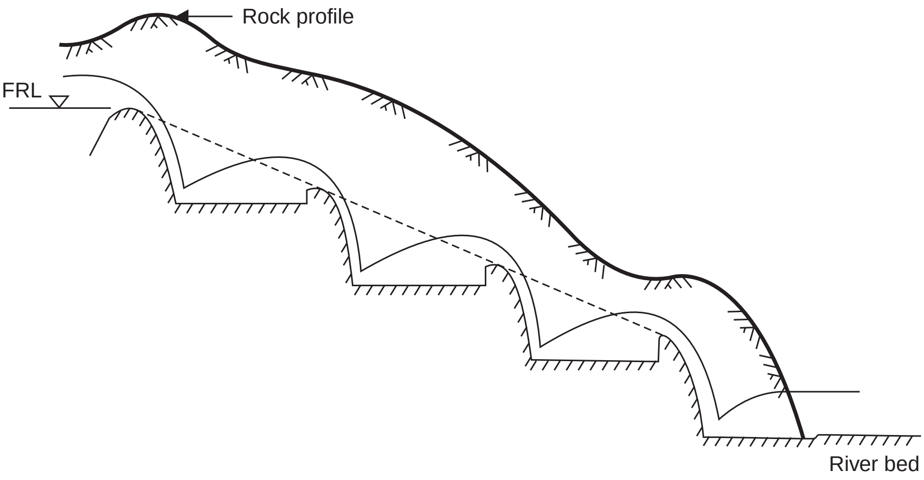


Fig. 17.19 Cascade spillway

17.4.8. Tunnel Spillway

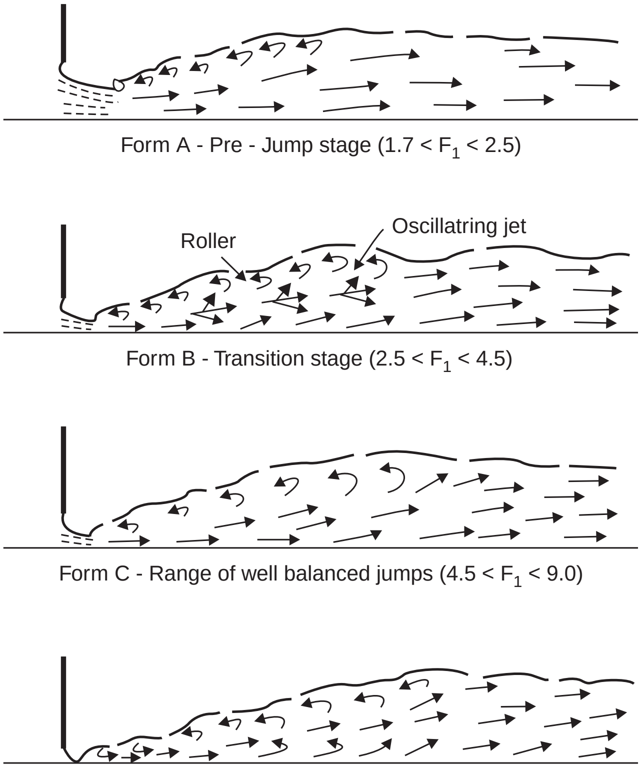
A tunnel spillway discharges water through closed channels or tunnels laid around or under a dam. The closed channels can be in the form of a vertical or inclined shaft, a conduit constructed in an open cut and back-filled with earth materials, or a horizontal tunnel through earth or rock. In narrow canyons with steep abutments as well as in wide valleys with abutments far away from the stream channel, tunnel spillways may prove to be advantageous. In such situations, the conduit of the spillway can be easily located under the dam near the stream bed.

17.5. TERMINAL STRUCTURES FOR SPILLWAYS

Some kind of energy dissipation is usually required before the spillway discharge is returned to the downstream river channel. An energy dissipator at the toe of the spillway is necessary to avoid or minimise erosion of river bed on the downstream side of the dam. A hydraulic jump is one of the best means to dissipate the excess energy of the falling water. Hence, wherever the tail-water conditions are suitable for the formation of a hydraulic jump, it is usual to provide hydraulic jump-type stilling basins which have been comprehensively studied by USBR (9). Alternatively, bucket type energy dissipators are provided.

17.5.1. Hydraulic Jump-Type Stilling Basins

Selection of a suitable type of stilling basin depends upon the characteristics of the hydraulic jump that would form. The characteristics of the jump, in turn, depend on the Froude number of the incoming flow as has been illustrated in Fig. 17.20. For Froude numbers between 1.0 and 1.7, the depth of incoming flow is only slightly less than the critical depth. The change from supercritical stage to subcritical stage is gradual and the water surface is only slightly ruffled. For Froude numbers between 1.7 and 2.5, surface rollers are formed but the flow is still relatively smooth (form A of Fig. 17.20). For Froude numbers between 2.5 and 4.5, the incoming jet intermittently flows near the bottom and along the water surface and thus forms



Form A - Pre - Jump stage ($1.7 < F_1 < 2.5$)

Form B - Transition stage ($2.5 < F_1 < 4.5$)

Form C - Range of well balanced jumps ($4.5 < F_1 < 9.0$)

Form D - Effective jump but rough surface downstream ($F_1 > 9.0$)

Fig. 17.20 Different forms of hydraulic jump

an oscillating hydraulic jump (form B of Fig. 17.20). The oscillating flow causes objectionable surface waves. A stable and well-balanced jump forms when the Froude number of the incoming flow lies between 4.5 and 9.0 (form C of Fig. 17.20). Water surface downstrem of the jump is relatively smooth and the action of the turbulence is confined within the body of the jump. When the Froude number exceeds 9.0, the surface roller and the turbulence are very active resulting in a rough water surface with strong surface waves downstream of the jump (form D of Fig. 17.20).

Of the different types of USBR stilling basins, Type III (Fig. 17.21) is commonly used. This basin is suitable when the Froude number of the incoming flow exceeds 4.5, and the velocity of incoming flow does not exceed 15.0 m/s. The purpose of providing accessories, such as baffle blocks, chute blocks, and sill is to ensure the formation of the jump even in conditions of inadequate tail-water depth, and thus permit shortening of the basin length. Energy dissipation is due to the turbulence in the jump and also by the impact on blocks. Because of the large impact forces on the baffle blocks and owing to the possibility of cavitation along the

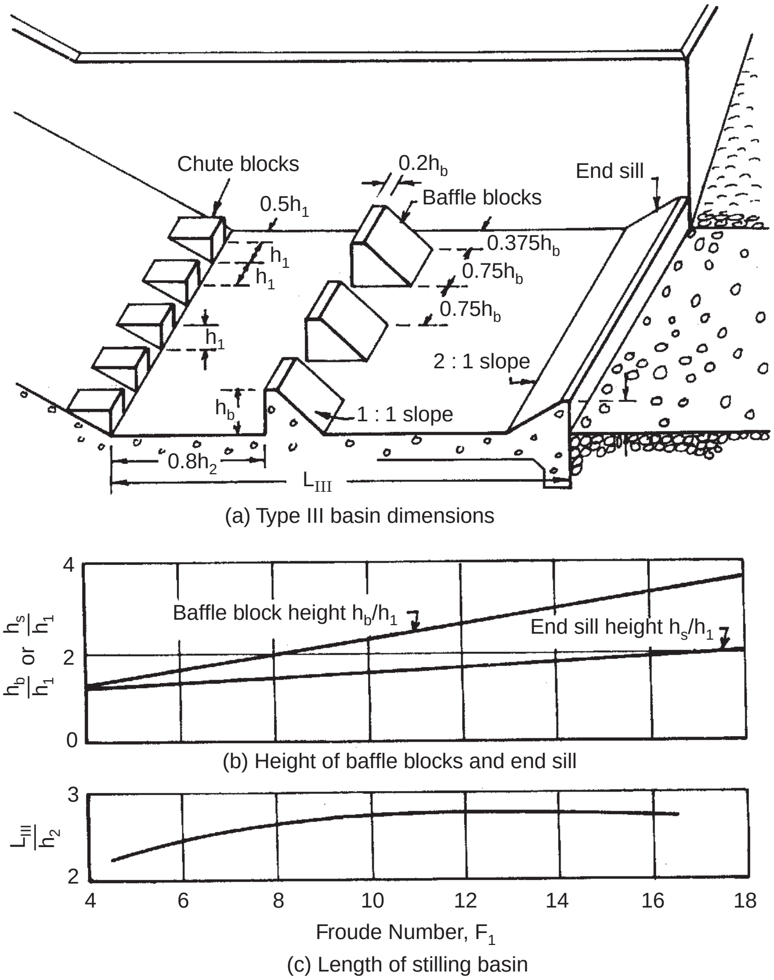
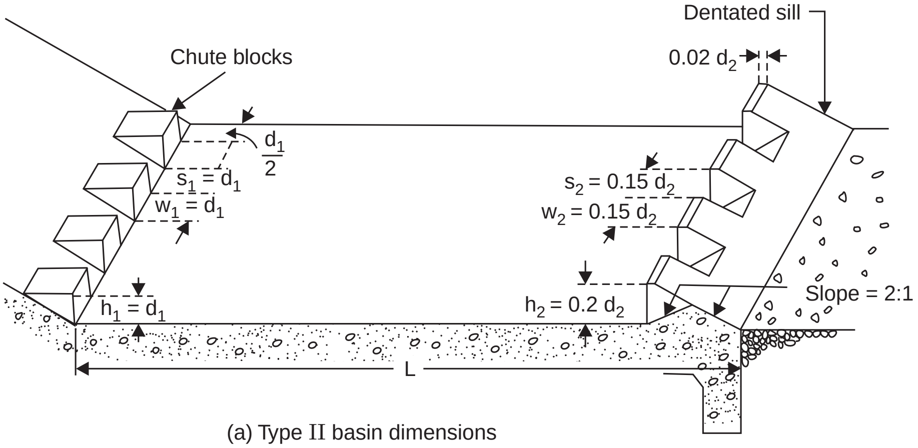


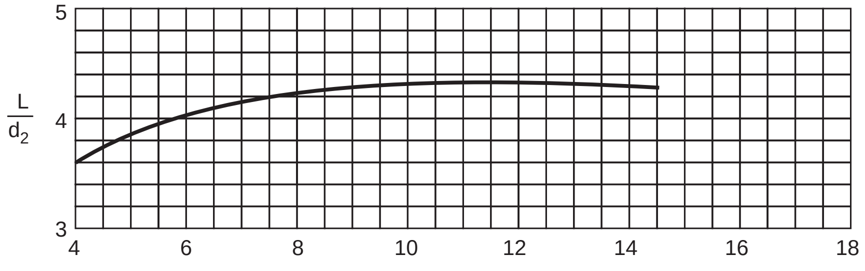
Fig. 17.21. USBR stilling basin Type III

surface of the blocks and floor, the use of this basin is limited to such conditions in which the velocity of incoming flow does not exceed 15.0 m/s. For good hydraulic performance, the side walls of a stilling basin are kept vertical or as nearly vertical as is practicable. For known conditions of incoming flow, the parameters of stilling basin can be determined from the curves of Fig. 17.21. A freeboard of 1.5 to 3.0 m should always be provided to allow for surging and wave action in the stilling basin.

When the velocity of the incoming flow exceeds 15 m/s, or when baffle blocks are not to be used, the USBR stilling basin, designated as Type II and shown in Fig. 17.22, should be adopted. Since the energy dissipation is accomplished mainly by hydraulic jump action, the basin length is bound to be longer than required for Type III basin. Also, the water depth in the basin should be about 5 per cent larger than the computed value of the post-jump conjugate depth.



(a) Type II basin dimensions



(b) Length of jump

Fig. 17.22 USBR stilling basin Type II

Example 17.4 Design a stilling basin of USBR type III for an ogee spillway with the following data:

Design discharge	= 13875 m^3/s
Tail-water level at the design discharge	= 180.30 m
Crest length of spillway	= 183 m

Tail-water depths (h_t) and post-jump depths (h_2) for different discharges are as follows:

Q (m^3/s)	0	3000	6000	9000	12000	13875
h_t (m)	0	6.90	9.40	11.30	12.90	13.80
h_2(m)	0	8.47	12.11	14.99	17.41	18.79

Solution : For the design discharge of 13875 m^3/s, h_2 = 18.79 m

$$F_2 = \frac{13875/(183 \times 18.79)}{\sqrt{9.81 \times 18.79}} = 0.297$$

$$\therefore \quad h_1 = \frac{h_2}{2}\left[\sqrt{1+8F_2^{\,2}} - 1\right]$$

$$= \frac{18.79}{2}\left[\sqrt{1+8\,(0.297)^2} - 1\right] = 2.875 \text{ m}$$

$$F_1 = \frac{13875/(183 \times 2.875)}{\sqrt{9.81 \times 2.875}} = 4.966$$

From Figs. 17.21 (*b*) and (*c*)

$$\frac{h_b}{h_1} = 1.49$$

$$\therefore \quad h_b = 4.28 \text{ m}$$

$$\frac{h_s}{h_1} = 1.30$$

$$\therefore \quad h_s = 3.74 \text{ m}$$

$$\frac{L_{III}}{h_2} = 2.32$$

$$\therefore \quad L_{III} = 43.59 \text{ m}$$

Other dimensions of the stilling basin as well as chute and baffle blocks can be determined using Fig. 17.21 (*a*).

17.5.2. Bucket-Type Energy Dissipators

When the tail-water depth is either too small or too large for the formation of hydraulic jump, the high amount of energy at the toe of spillway can be dissipated by the use of bucket-type energy dissipators. These can be either: (*i*) a trajectory (or deflector) bucket, or (*ii*) a roller (or submerged) bucket energy dissipator.

(*i*) *Trajectory* (*or Deflector*) *Bucket:* When the tail-water depth is lesser than the depth required for the jump formation and the bed of the river channel is composed of sound rock capable of withstanding the impact of the trajectory jet, a trajectory bucket (also known as a flip or ski-jump bucket) (Fig. 17.23) is generally used as an energy dissipator. The incoming jet of water leaves the bucket as a free-discharging upturned jet and falls into the stream channel some distance downstream of the end of the spillway. The upturned jet gets split into a number of bubbles or smaller jets. The energy is dissipated on account of the increased air resistance (because of splitting of the jet) as well as the impact against water and the channel bed downstream.

With the end of the lip as the origin of the coordinate system, the path of the trajectory is given by the equation (10):

$$y = x \tan\theta - \frac{x^2}{4k_1 E \cos^2\theta} \tag{17.25}$$

where, θ, known as the lip angle, is the angle between the curve of the bucket at the lip and the horizontal, E the specific energy, and k_1 is assigned a value of 0.85 to compensate for loss of energy and velocity reduction due to the effect of the air resistance, internal turbulence, and disintegration of the jet.

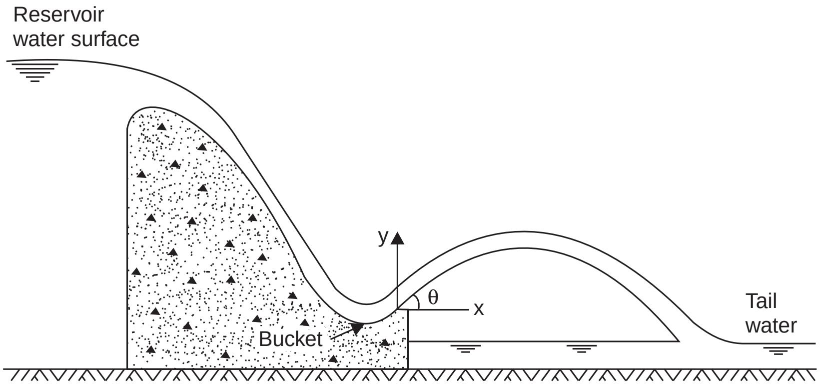


Fig. 17.23 Trajectory bucket

The horizontal range of the jet at the level of the lip (*i.e.*, $y = 0$) is given by

$$x = 4k_1E \tan\theta \cos^2\theta$$
$$= 2k_1E \sin 2\theta$$

The maximum value of x will be $2\,k_1E$ when θ is 45°. The lip angle depends on the radius of the bucket and the height of the lip above the bucket invert. It usually varies from 20° to 45°. The bucket radius should be large enough to ensure a concentric flow along the bucket (10).

(*ii*) *Roller* (*or Submerged*) *Bucket:* A roller bucket is used when the tail-water depth is greater than 1.1 times the required conjugate depth for the formation of the hydraulic jump and the river bed rock is good (10). Roller buckets can be either solid or slotted (Fig. 17.24). The

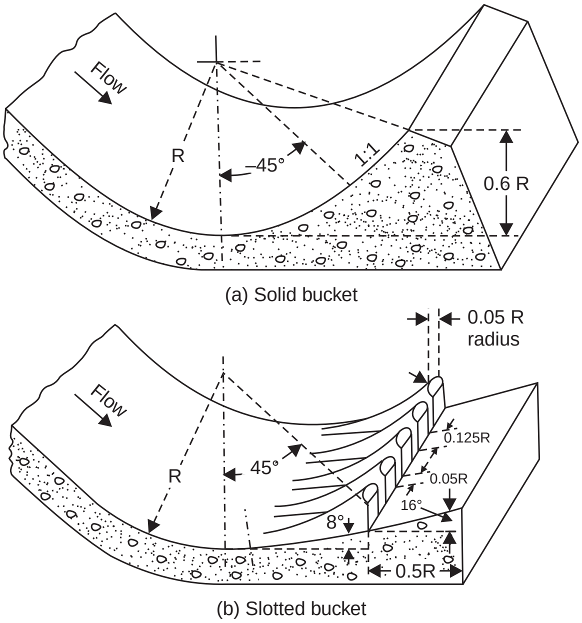


Fig. 17.24 Roller buckets

general hydraulic action responsible for the dissipation of energy is shown in Fig. 17.25. The energy dissipates through the formation of two rollers; one is on the surface of the bucket and moves anticlockwise (considering that the flow is to the right), and the other is a ground roller moving in a clockwise direction and which forms immediately downstream of the bucket (Fig. 17.25). The intermingling of the incoming flow with the roller as well as the movements of the latter dissipate the energy of the water effectively and prevent excessive scouring downstream of the bucket.

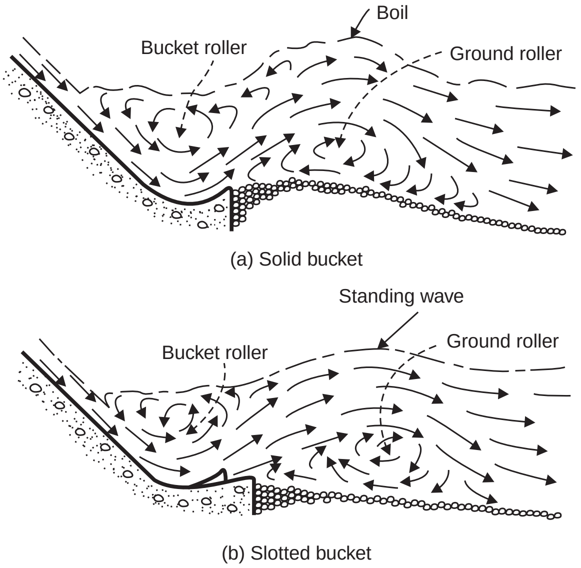


Fig. 17.25 Roller formation in roller buckets

Although the hydraulic action of the two buckets is similar, there are some differing flow features as well. The deflector lip of a solid bucket directs upward the high-velocity flow and thus creates a high boil on the water surface and a violent ground roller moving clockwise immediately downstream of the bucket. This ground roller picks up the loose material and keeps some of it in a constant state of agitation. There may be unwanted abrasion on the concrete surfaces because of the loose material which is brought back towards the lip of the bucket by the ground roller. Further, the more violent water surface created by surface boil is carried downstream causing objectionable eddy currents which may adversely affect the river banks. In the slotted bucket, the high-velocity jet leaves the lip of the bucket at a relatively flatter angle and only a part of the high-velocity flow finds its way to the water surface. Thus, a relatively less violent surface boil forms and there is better dispersion of the flow in the region above the ground roller. Therefore, there is less concentration of high-energy flow throughout the bucket and a smoother downstream flow.

17.6. TYPES OF GATES FOR SPILLWAY CRESTS

Free or uncontrolled overflow crest of a spillway is the simplest form of control as it automatically releases water whenever the reservoir water level rises above the crest level. For such crests, there is no need of constant attendance and regulation of the control devices by an operator. Besides, the problems of maintenance and repair of the controlling device also do not arise. However, when sufficiently long uncontrolled crest or large surcharge head for the required spillway capacity cannot be obtained, a regulation gate may be necessary. Such regulating devices enable the spillway to release storages even when the water level in the reservoir is below the normal reservoir water surface. Gates can be provided on all types of spillways except the siphon spillway. The installation of gates involves additional expenditure on initial cost, and on their repair and maintenance. The selection of type and size of the controlling device depends on several factors, such as: (*i*) discharge characteristics of the device, (*ii*) climate, (*iii*) frequency and nature of floods, (*iv*) winter storage requirements, (*v*) the need for handling ice and debris, and (*vi*) special operating requirements such as presence of operator during periods of flood, the availability of electricity, operating mechanism, and so on. In addition, economy, reliability, efficiency, and adaptability of the regulating device must also the looked into.

The following types of regulating devices are generally used:

(*i*) Flashboards and stoplogs,

(*ii*) Rectangular lift gates,

(*iii*) Radial gates, and

(*iv*) Drum gates.

These may be controlled either manually or automatically through mechanical or hydraulic operations.

17.6.1. Flashboards and Stoplogs

Flashboards and stoplogs raise the reservoir storage level above a fixed spillway crest level when the spillway is not required to release flood. The flashboards usually consist of individual boards or panels of 1.0 to 1.25 m height. These are hinged at the bottom and are supported against water pressure by struts. Stoplogs are individual beams or girders set one upon the other to form a bulkhead supported in grooves at each end of the span. To increase the spillway capacity, the flashboards or stoplogs are removed prior to the flood. Alternatively, they are designed and arranged so that they can be removed while being overtopped. Flashboards and stoplogs are simple and economical type of regulating devices which provide an unobstructed crest when removed. However, they have the following disadvantages (1):

(*i*) They present a hazard if not removed in time to pass floods, especially where the reservoir area is small and the stream is subject to flash floods,

(*ii*) They require attendance of an operator or crew to remove them, unless designed to fall automatically,

(*iii*) Ordinarily they cannot be restored to position while water flows over the crest,

(*iv*) If they are designed to fail when the water reaches certain stage, their operation is uncertain, and when they fail they release sudden and undesirably large outflows, and

(*v*) If the spillway functions frequently, the repeated replacement of flashboards may be costly.

17.6.2. Vertical Lift Gates

These are usually rectangular in shape and made of steel which span horizontally between guide grooves in supporting piers and move vertically in their own plane. The gates are raised or lowered by an overhead hoist and water is released by undershot orifice flow for all gate openings. Sliding gates offer large sliding friction due to water pressure and, therefore, require a large hoisting capacity. The use of wheels (along each side of the gate) would reduce the amount of sliding friction and thereby permit the use of a smaller hoist. Vertical lift gates have been used for spans and heights of the order of 20 m and 15 m, respectively. At larger heights, however, the problem of a raised operating platform becomes important.

17.6.3. Radial (or Tainter) Gates

These are made of steel plates which form a segment of a cylinder which itself is attached to supporting bearing by radial arms. The cylindrical plate is kept concentric to the supporting pins so that the entire thrust of the waterload passes through the pins and only a small amount of moment needs to be overcome in raising or lowering the gate. The hoisting loads then include only the weight of the gate, the sliding friction, and the frictional resistance at the pins. The small hoisting effort required for the operation of the radial gates makes hand operations at small installations possible. Besides, they require lesser head rooms than required by vertical lift gates. All these advantages make the radial gates more adaptable.

17.6.4. Drum Gates

Drum gates are hollow (and, therefore, buoyant), triangular in section, and made of steel plates. The drum gate is hinged at the upstream lip of a hydraulic chamber in the weir structure in which the gate floats. Water introduced into or drawn from the hydraulic chamber causes the gate to swing upwards or downwards. The inflow or outflow of water to the chamber is governed by controls located in the piers adjacent to the chambers.

17.7. CAVITATION EROSION ON SPILLWAY SURFACE

Surface roughnesses of a spillway causes separation of the boundary layer from the surface, thereby forming a lower pressure zone immediately downstream of the roughness. If the pressure in the wake region of the roughness falls to the vapour pressure of water, vapour bubbles are formed and carried downstream. When these bubbles reach a high pressure region, they suddenly collapse giving rise to extremely high pressures. These extremely high localised pressure cause damage to any surface adjacent to the collapsing bubbles (or cavities). The continued process of this nature, named as cavitation pitting or cavitation erosion, may cause serious damage to the structure in due course of time.

With the increasing demand for water, high dams are being proposed, planned, and constructed. The flow velocities over the spillways of such projects often exceed 30-40 m/s which are enough to cause cavitation pitting of the normal concrete surface of the spillway. The methods to prevent cavitation, used till recently, consisted of either using cavitation-resistant materials (such as steel lining, epoxy concretes, epoxy mortars, and fibrous concrete) for the construction of the surface or the adoption of specification criteria for limits of construction finish. The first method, being costly, is reserved for small areas such as near the outlet gates, or for repairing damaged surfaces. The second method defines objectionable irregularities. However, the standards are difficult to obtain especially when the flow velocities exceed 20 m/s. Besides, the method is relatively uncertain because of the defects which may develop on the surface as a result of climatic, atmospheric or chemical conditions.

More recently, a new method has been developed to protect spillway surfaces by aeration devices. This method is based on the known fact that the presence of air bubbles hinders cavitation. Hence, present-day design of spillways with high velocities envisages provision of aeration devices, known as spillway aerators, across the spillway face. Three basic types of aerators, *viz.*, groove, deflector, and offset (Fig. 17.26) have been used. An aerator of the groove-type has been constructed for the Karjan dam in the Narmada basin.

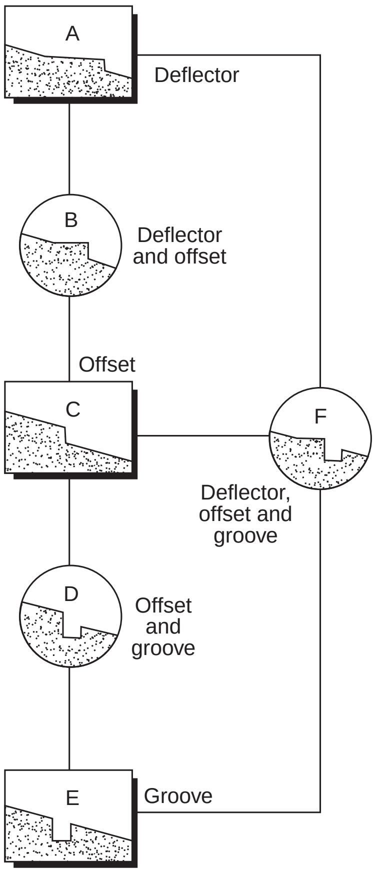


Fig. 17.26 Spillway aerators

The aerators are relatively cheap and have proved successful in preventing cavitation. These devices produce a local pressure drop which causes the flow to suck air. The compressibility of the air-water mixture reduces considerably the collapsing pressure of vapour bubbles and thus protects the concrete surface from cavitation erosion.

EXERCISES

17.1 Explain the functions of different components of a spillway and suitability of different types of spillway for different site conditions and other requirements.

17.2 Which types of gate are generally used for spillway crests ?

17.3 Why is the cavitation erosion of a spillway surface caused ? What are the possible measures to prevent or reduce it ?

17.4 Determine the profile of an overfall spillway with vertical upstream face using the following data:

Spillway discharge = 1600 m^3/s

Effective length of spillway = 100.00 m

Coefficient of discharge for spillway = 2.0

Spillway crest elevation = 205.00 m

River bed elevation = 100.00 m

If the tail-water depth for discharge of 1600 m^3/s is 10.5 m, design a suitable stilling basin and sketch it.

17.5 A flood comes into a reservoir at zero hour. The ordinates of the flood hydrograph are as follows:

Time in hours	0	1	2	3	4	5	6	7
Discharge (m^3/s)	0	100	200	300	400	500	400	300

The area elevation curve is also linear and is given by the following ordinates:

Height above spillway crest (m)	0	1	2	3	4
Area (10^5 sq.m)	7	8	9	10	11

The discharge formula for the spillway is $Q = 300\ H^{3/2}$ where, Q is in m^3/s and H is in metres. Find the reservoir level at 4 hours after the start of the flood assuming that water is up to the crest level at zero hour.

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Appendix 1

INTERLINKING OF RIVERS IN INDIA*

After air, water (required for domestic, irrigation, municipal and industrial purposes) is the next primary need of mankind. Prosperity, health and material progress of any country is very much linked to its ability to store and direct water. India, with its geographical area of 329 million hectares and ever-growing population (present population growth rate being 1.93%) of more than a billion at present, is blessed with many large river basins totaling to around 280 million hectares of catchment area and runoff of about 4 per cent of the total average annual runoff of the world. The average annual rainfall of around 1143 mm in the country is quite erratic in its spatial variation (varying from 11489 mm around Cherrapunji in Assam to only 217 mm around Jaisalmer in Rajasthan) and also temporal variation (75% to 90% of the annual rainfall occurring during 25–60 rainy days of the four monsoon moths from June to September). As a result, there are many river basins in India where the water resources are excessively surplus (largely unutilized) and give rise to problems of floods and continued submergence of crops. At the same time, there are other river basins where the available water resource is too small resulting in draught conditions.

It is primarily due to the rapid development in irrigation during the last five decades that India is presently self-sufficient in meeting its entire food requirement. However, unfortunately, safe drinking water needs are not adequately met even at present. The situation (for both food and drinking water) is likely to worsen in future due to the ever-increasing population expected to be around 1500 million in 2050. The country would then need to produce around 400 million tonnes of food (against the present production of around 200 million tonnes) for which irrigation potential would have to be increased by at least about 50 per cent. Total water needs (in 2050) would be around 1100 cubic km (against the present water need of around 600 cubic km). Therefore, alongwith integrated water resources development and management, some well-planned drastic measures have to be initiated and implemented well in time so as to increase the utilizable component of nation's water resources.

It is heartening to note that the Govt. of India has initiated a laudable gigantic programme of interlinking some major rivers of the country to effect inter-basins to increase the utilizable component of the nation's water resources and solve the problems of shortages and excesses of water in some parts of the country. The execution of such a programme, presently estimated to cost Rs. Six hundred thousand crores, is quite challenging and its implementation would have long-lasting impact (irrigation expansion, enhancement in the hydropower, fulfillment of water demands en route, flood control by moderation of flood peaks due to construction of new dams, and ecosystems on account of bio-deversity and climatic changes) on the well-being of the Indian population.

In the past, several inter-state water disputes have resulted much less than optimum utilization of water even after harnessing the available water resources. Several projects have

*Asawa, GL, Interlinking of Rivers in India, The Indian Society for Hydraulics, ISH–News, Vol. 12, No. 1, June 2003.

taken too long a time for completion and have experienced several hindrances including legal, political, rehabilitation, environmental *etc*. As a result, the original cost estimates and anticipated benefits have gone awry much beyond any reasonable expectation. Several kinds of hindrances are anticipated for the proposed interlinking of rivers as well. Therefore, it is necessary to take all suitable steps, well in advance, including legislative ones, to eliminate any possibility of delays in implementation of the proposed interlinking of rivers. People's active participation at all stages of the programme must be ensured. In addition, the need and benefits of the proposal as well as rehabilitation (of the displaced people) programmes must be given wide publicity in order to build favourable public opinion for the proposal.

Rivers are known to change their behaviour drastically when they are tinkered. Interlinking of rivers would , obviously, involve construction of several hydraulic structures of all kinds and complexities. Therefore, the interlinking of rivers, when implemented, may cause tremendous changes in river morphology and which may not be predicted correctly even with the best of the mathematical models available for the purpose due to so many uncertainties and variations involved. Therefore, while it is important that the schedule of completion is not unduly delayed, no details should ever be missed at any of the stages of planning, design, construction and operation. It needs to be emphasized that any wrong decision in the matter (either to meet the deadline or otherwise) would be disastrous and, to a large extent, irreversible. Therefore, it is imperative that the feasibility (planning, design, construction, operation and maintenance) of each of the several components of the proposal (and also other alternatives even if fulfilling only part needs) and its impact on environment (changing recharge patterns, reduced outflow to sea *etc*.) are properly assessed and critically examined from technical point of view in an unbiased manner. It may be worthwhile considering to start with only few relatively minor links and gain experience on their impact and, thereafter, proceed further with major links.

While irrigation facilities were being developed in the country in the past, the drainage of the irrigation water was usually not considered as the necessary complementary component of irrigation. As a result, many of the irrigation commands have been adversely affected due to waterlogging. Also, the present proposal may necessitate water level in the interlinking canals to be higher than the surrounding ground level in several stretches. This is likely to induce waterlogging around the region of those stretches. The present proposal should include adequate precautions and drainage measures to avoid waterlogging in the irrigated commands to be added to the existing ones. Water level in the interlinking canals should be kept below the surrounding ground level to the extent possible.

Alternatives (such as taking surplus water from rivers before its outflow into the sea and transporting through ship and highway/railway tankers to the storage reservoir in water-scarce regions or increasing recharge in water-surplus regions so as to effect inter-basin transfer through aquifers or rain water harvesting or effective watershed management *etc.*), meeting only part of the needs, must also be examined and implemented, if found feasible technically and otherwise.

In our country, the food production per unit of either supplied irrigation water or area under cultivation is far short of its potential and world averages. Likewise, full potential of the developed water resource system is also not fully exploited. These are obvious indicators of poor management of the developed water resources and inadequate on-farm operations. Steps (such as switching over to more efficient irrigation methods like sprinkler and drip irrigation, improved on-farm management, training and education of farmers for efficient use of water *etc*.) should be initiated so that these shortcomings are removed for the existing systems without delay. Necessary measures (like developing infrastructure and educating farmers of the concerned region) must be incorporated so that there is no delay in efficient utilization of the full potential of the proposed project (or its several components) when it is completed. Strategies should be evolved for improving water and land productivity, and eliminating the possibility of polluting water resources.

It is common knowledge and experience that the technologies or expertise or consultancy services or equipment bought from the developed countries by paying huge sums of hard-earned foreign exchange are, at times, either the obsolete ones or inferior to what are indigenously available. Therefore, such purchases should be resorted to only if there is no alternative and only the latest and the best must be procured.

While the water availability per unit of land in India is one of the highest, the land and water availability per capita is one of the lowest in the world. This and many other problems of India are primarily due to its huge population. These problems would only multiply with increasing population even after taking measures such as the proposed ones. Therefore, stern steps are immediately needed for reducing the population growth rate considerably in the country through education, persuasion and even mild coercion, if required.

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Author Index

Subject Index