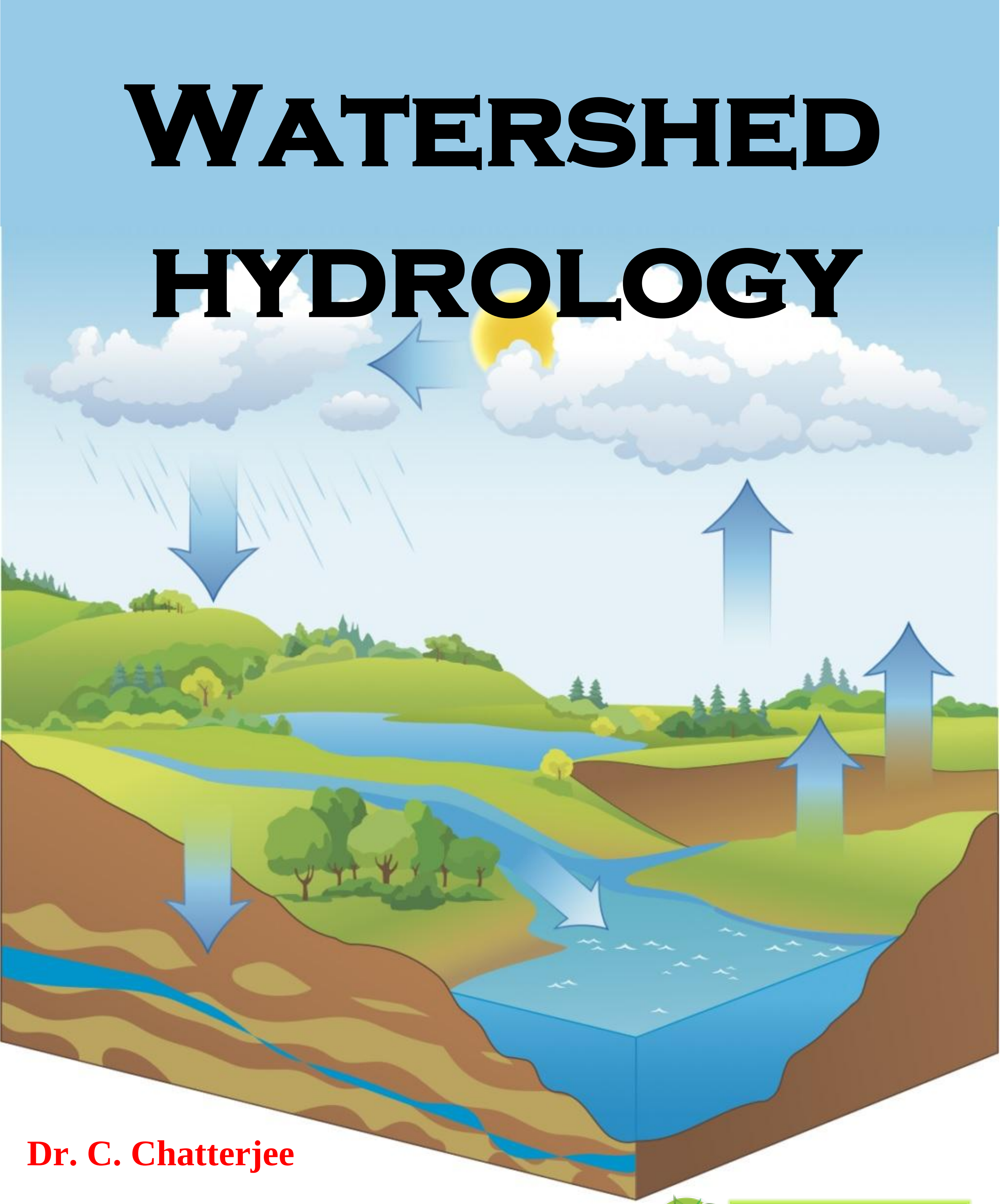
# WATERSHED HYDROLOGY



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# Watershed hydrology

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# Index

**Module 7. Flood Routing**

## Module.1 Introduction of Water Resources and Hydrology

### Lesson 1 Water Resources Availability and Requirements of India

"Water is probably the only natural resource to touch all aspects of human civilization – from agricultural and industrial development to cultural and religious values embedded in society"(USSR, 1978).

### 1.1 Global Water Resources

Earth's water has been estimated to be 1.385 billion km$^3$, 96.5 % of which is in the oceans and is saline. Of the remaining 3.5% water on land, approximately 1% is contained in deep, saline ground waters or in saline lakes, leaving only 2.5% of the earth's water as freshwater. Of this freshwater, only 1.3% is mobile in the surface and atmospheric phases of the hydrologic cycle.

Table 1.1.Quantities of water in different phases of the hydrologic cycle (Source: USSR, 1978)

| Sl.No. | Item | Area $10^6$ km$^2$ | Volume (km$^3$) | Percent of total water | Percent of freshwater |
|---|---|---|---|---|---|
| 1. | Oceans | 361.3 | 1,338,000,000 | 96.5 | |
| 2. | Ground water:<br>Fresh<br>Saline | <br>134.8<br>134.8 | <br>10,530,000<br>12,870,000 | <br>0.76<br>0.93 | 30.1 |
| 3. | Soil moisture | 82.0 | 16,500 | 0.0012 | 0.05 |
| 4. | Polar ice | 16.0 | 24,023,500 | 1.7 | 68.6 |
| 5. | Other ice and snow | 0.3 | 340,600 | 0.025 | 1.0 |
| 6. | Lakes:<br>Fresh<br>Saline | 1.2<br>0.8 | 91,000<br>85,400 | 0.007<br>0.006 | 0.26 |

| | | | | | |
|---|---|---|---|---|---|
| 7. | Marshes | 2.7 | 11,470 | 0.0008 | 0.03 |
| 8. | Rivers | 148.8 | 2,120 | 0.0002 | 0.006 |
| 9. | Biological water | 510.0 | 1,120 | 0.0001 | 0.003 |
| 10. | Atmospheric water | 510.0 | 12,900 | 0.001 | 0.04 |
| 11. | Total water | 510.0 | 1,385,984,610 | 100 | |
| 12. | Freshwater | 148.8 | 35,029,210 | 2.5 | 100 |

As per WHO estimates, only 0.007% of all water on earth is readily available for human consumption.

## 1.2 Water Resources of India

India receives annual precipitation of about 4000 km$^3$. The rainfall in India shows very high spatial and temporal variability and paradox of the situation is that Mousinram, Cherapunji, which receives the highest rainfall in the world, also suffers from shortage of water during the non-rainy season, almost every year. The total average annual flow per year for the Indian rivers is estimated as 1953 km$^{3.}$ The total annual replenishable ground water resources are assessed as 432 km$^3$. The annual utilizable surface water and ground water resources of India are estimated as 690 km$^3$ and 396 km$^3$ per year, respectively. Thus, the total utilizable water resources of the country are assessed as 1086 km$^3$.

Rainfall in India is dependent on the South-West and North-East monsoons, on shallow cyclonic depressions and disturbances and on local storms. Most of it takes place under the influence of South-West monsoon between June to September except in Tamil Nadu, where it is under the influence of North-East monsoon during October and November.

India is gifted with a river system comprising of more than 20 major rivers with several tributaries. Many of these rivers are perennial and some of these are seasonal. The rivers like Ganges, Brahmaputra and Indus originate from the Himalayas and carry water throughout the year. The snow and ice melt of the Himalayas and the base flow contribute the flows during the lean season. More than 50% of water resources of India are located in various tributaries of these river systems. Average water yield per unit area of the Himalayan rivers is almost double that of the south peninsular rivers system indicating the importance of snow and glacier melt contribution from the high mountains.

The peninsular rivers such as Narmada, Mahanadi, Godavari, Krishna, Cauvery etc. carry flows from rainfall and are supported by base flow discharges. Thus, these rivers carry less water during the lean season of the non-monsoon period.

Apart from the water available in the various rivers of the country, the ground water is also an important source of water for drinking, irrigation and industrial uses etc. It accounts for about 80% of domestic water requirement and more than 45% of the total irrigation in the country. The ground water aquifers work like a regulating reservoir for storing water during the rainy season and releasing it during the lean season.

In spite of availability of substantial quantity of water in India, the actual utilizable quantity is limited and a freshwater crisis is gradually unfolding in India. The crisis unfolds by the way of lack of access to safe water supply to millions of people as a result of inadequate water planning and environmental degradation.

### 1.2.1 Monsoon in India

Normal duration of monsoon in India is about 100 to 120 days beginning from first June. In India, the two monsoon seasons (the southwest monsoon in June to September and the northeast monsoon in November-December) bring forth rains – many a times in intensities and amounts sufficient to cause serious floods creating hazardous situations.

### 1.2.2 Precipitation Variability

The long-term average annual rainfall for the country is 1160 mm, which is the highest anywhere in the world for a country of comparable size. The annual rainfall in India, however, fluctuates widely. The highest rainfall in India of about 11,690 mm is recorded at Mousinram, Cherrapunji in Meghalaya in the northeast. In this region rainfall as much as 1040 mm is recorded in a day. At the other extreme are places like Jaisalmer, in the west, which receives barely 150 mm of rain. Though the average rainfall is adequate, nearly three-quarters of the rain pours down in less than 120 days, from June to September. As much as 21 percent of the area of the country receives less than 750 mm of rain annually while 15 percent receives rainfall in excess of 1500 mm. Precipitation generally exceeds 1000 mm in areas to the east of Longitude $78^0$ E. It reaches nearly to 2500 mm along almost the entire west coast and over most of Assam and sub-Himalayan West Bengal.Large areas of peninsular India receive rainfall less than 600 mm. Annual rainfall of less than 500 mm is experienced in western Rajasthan and adjoining parts of Gujarat, Haryana and Punjab. Rainfall is equally low in the interior of the Deccan plateau, east of the Sahyadris. A third area of low precipitation is around Leh in Kashmir. Rest of the country receives moderate rainfall.

### 1.2.3 Surface Water Resources of India

National Commission for Integrated Water Resources Development (NCIWRD, 1999) estimated the basin-wise average annual flow in Indian river systems as 1953 km$^3$. Utilizable water resource is the quantum of withdrawable water from its place of natural occurrence. Within the limitations of physiographic conditions and socio-political environment, legal and constitutional constraints and the technology of development available at present, utilizable quantity of water from the surface flow has been assessed by various authorities differently. According to NCIWRD (1999), the utilizable annual surface water of the country is 690 km$^3$. There is considerable scope for increasing the utilization of water in Ganga-Brahmaputra basins by construction of storages at suitable locations in neighbouring countries.

Table 1.2.Basin-wise average flow and utilizable water (in km$^3$/ year) (Source:NCIWRD, 1999)

| S.No. | River Basin | Average Annual Flow | Utilizable Flow |
|---|---|---|---|
| 1 | Indus | 73.31 | 46 |
| 2 | Ganga-Brahmaputra-Meghna Basin | | |
| | (a) Ganga | 525.02 | 250 |
| | (b) Brahmaputra sub-basin | 629.05 | 24 |
| | (c) Meghna (Barak) sub-basin | 48.36 | |
| 3 | Subarnarekha | 12.37 | 6.81 |
| 4 | Brahmni-Baitarani | 28.48 | 18.3 |
| 5 | Mahanadi | 66.88 | 49.99 |
| 6 | Godavari | 110.54 | 76.3 |
| 7 | Krishna | 69.81 | 58 |
| 8 | Pennar | 6.32 | 6.86 |
| 9 | Cauvery | 21.36 | 19 |
| 10 | Tapi | 14.88 | 14.5 |
| 11 | Narmada | 45.64 | 34.5 |
| 12 | Mahi | 11.02 | 3.1 |

| | | | |
|---|---|---|---|
| 13 | Sabarmati | 3.81 | 1.93 |
| 14 | West flowing rivers of Kachchh and Saurashtra including Luni | 15.1 | 14.98 |
| 15 | West flowing rivers south of Tapi | 200.94 | 36.21 |
| 16 | East flowing rivers between Mahanadi and Godavari | 17.08 | 13.11 |
| 17 | East flowing rivers Between Godavari and Krishna | 1.81 | |
| 18 | East flowing rivers Between Krishna and Pennar | 3.63 | |
| 19 | East flowing rivers Between Pennar and Cauvery | 9.98 | 16.73 |
| 20 | East flowing rivers south of Cauvery | 6.48 | |
| 21 | Area of North Ladakh not draining into Indus | 0 | NA |
| 22 | Rivers draining into Bangladesh | 8.57 | NA |
| 23 | Rivers draining into Myanmar | 22.43 | NA |
| 24 | Drainage areas of Andman, Nicobar and Lakshadweep Islands | 0 | NA |
| | Total (Rounded) | 1953 | 690 |

### 1.2.4 Groundwater Resources

The annual potential natural ground water recharge from rainfall in India is about 342.43 km$^3$, which is 8.56% of total annual rainfall of the country. The annual potential ground water recharge augmentation from canal irrigation system is about 89.46 km$^3$. Thus, total replenishable groundwater resource of the country is assessed as 431.89 km$^3$ (342.43 km$^3$ + 89.46 km$^3$).After allotting 15% of this quantity for drinking, and 6 km$^3$ for industrial purposes, the remaining can be utilized for irrigation purposes. Thus, the available ground water resource for irrigation is 361 km$^3$ of which utilizable quantity (90%) is 325 km$^3$.

Table 1.3.Groundwater resources of India, in km$^3$(Source: CGWB, 1995)

| | |
|---|---|
| Total Replenishable Ground Water Resource | 432 |
| Provision for Domestic, Industrial and other Uses | 71 |
| Available Ground Water Resource for Irrigation | 361 |
| Utilizable Ground Water Resource for Irrigation (90% of the Sl. No. 3) | 325 |
| Total Utilizable Ground Water Resource (Sum of Sl. No. 2 and 4) | 396 |

Table 1.4.Ground water potential in river basins of India, in km$^3$/year (Source: IWRS, 1998)

| S.No. | Name of the basin | Total replenis-able ground water resources | Provision for domestic, industrial and other uses | Available ground water for irrigation | Net draft | Balance ground water potential | Level of ground water development (%) |
|---|---|---|---|---|---|---|---|
| 1 | Brahmani with Baitarni | 4.05 | 0.61 | 3.44 | 0.29 | 3.15 | 8.45 |
| 2 | Brahmaputra | 26.55 | 3.98 | 22.56 | 0.76 | 21.80 | 3.37 |
| 3 | Chambal Composite | 7.19 | 1.08 | 6.11 | 2.45 | 3.66 | 40.09 |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| 4 | Cauvery | 12.30 | 1.84 | 10.45 | 5.78 | 4.67 | 55.33 |
| 5 | Ganga | 170.99 | 26.03 | 144.96 | 48.59 | 96.37 | 33.52 |
| 6 | Godavari | 40.65 | 9.66 | 30.99 | 6.05 | 24.94 | 19.53 |
| 7 | Indus | 26.49 | 3.05 | 23.43 | 18.21 | 5.22 | 77.71 |
| 8 | Krishna | 26.41 | 5.58 | 20.83 | 6.33 | 14.50 | 30.39 |
| 9 | Kutch &Saurashtra Composite | 11.23 | 1.74 | 9.49 | 4.85 | 4.64 | 51.14 |
| 10 | Madras and South Tamil Nadu | 18.22 | 2.73 | 15.48 | 8.93 | 6.55 | 57.68 |
| 11 | Mahanadi | 16.46 | 2.47 | 13.99 | 0.97 | 13.02 | 6.95 |
| 12 | Meghna | 8.52 | 1.28 | 7.24 | 0.29 | 6.95 | 3.94 |
| 13 | Narmada | 10.83 | 1.65 | 9.17 | 1.99 | 7.18 | 21.74 |
| 14 | Northeast Composite | 18.84 | 2.83 | 16.02 | 2.76 | 13.26 | 17.20 |
| 15 | Pennar | 4.93 | 0.74 | 4.19 | 1.53 | 2.66 | 36.60 |
| 16 | Subarnarekha | 1.82 | 0.27 | 1.55 | 0.15 | 1.40 | 9.57 |
| 17 | Tapi | 8.27 | 2.34 | 5.93 | 1.96 | 3.97 | 33.05 |
| 18 | Western Ghat | 17.69 | 3.19 | 14.50 | 3.32 | 11.18 | 22.88 |
| | Total | 431.43 | 71.08 | 360.35 | 115.21 | 245.13 | 31.97 |

## 1.3 Water Requirements of India

Traditionally, India has been an agriculture-based economy. Hence, development of irrigation to increase agricultural production to make the country self-sustained and for poverty alleviation has been of crucial importance for the planners. Accordingly, irrigation sector was assigned a very high priority in the 5-year plans. Giant schemes like the BhakraNangal, Hirakud, Damodar Valley, Nagarjunasagar, Rajasthan Canal project etc. were taken up to increase irrigation potential and maximize agricultural production.

As per the international norms, if per-capita water availability is less than 1700 $m^3$ per year then the country is categorized as water stressed and if is less than 1000 $m^3$ per capita per year then the country is classified as water scarce. Table 1.5 provides details of the population of India and per capita water availability as well as utilizable surface water for some of the years from 1951 to 2050 (projected). The availability of water in India shows wide spatial and temporal variations. Most of rainfall is received in a few months, and that too within 100 hours of rainy days. Also, there are very large inter annual variations. Hence, the general situation of availability of per capita availability is much more alarming than what is depicted by the average figures.

Table 1.5.Per capita per year availability and utilizable surface water in India (in $m^3$) (Source:NCIWRD, 1999)

| Year | Population (in millions) | Per-capita surface water availability | Per-capita utilizable surface water |
|---|---|---|---|
| 1951 | 361 | 5410 | 1911 |
| 1955 | 395 | 4944 | 1746 |
| 1991 | 846 | 2309 | 816 |
| 2001 | 1027 | 1902 | 672 |
| 2025 (Projected) | (a) 1286 (Low growth)<br>(b) 1333 (High growth) | 1519<br>1465 | 495 |
| 2050 (Projected) | (a) 1346 (Low growth)<br>(b) 1581 (High growth) | 1451<br>1235 | 421 |

### 1.3.1 Domestic Use

Community water supply is the most important requirement and it is about 5% of the total water use. About 7 $km^3$ of surface water and 18 $km^3$ of ground water are being used for community water supply in urban and rural areas. Long-term planning has to account for the growth of population.

A number of individuals and agencies have estimated the likely population of India by the year 2025 and 2050. According to the estimates adopted by NCIWRD (1999), by the year 2025, the population is expected to be 1333 million in high growth scenario and 1286 million in low growth scenario. For the year 2050, high rate of population growth is likely to result in about 1581 million people while the low growth projections place the number at nearly 1346 million. Keeping in view the level of consumption, losses in storage and transport, seed requirement, and buffer stock, the projected food-grain and feed demand for 2025 would be 320 million tonnes (high demand scenario) and 308 million tonnes (low demand scenario). The requirement of food grains for the year 2050 would be 494 million tonnes (high demand scenario) and 420 million tonnes (low demand scenario).

Different organizations and individuals have given different norms for water supply in cities and rural areas. The figure adopted by the NCIWRD (1999) was 220 litre per capita per day (lpcd) for class I cities. For the cities other than class I, the norms are 165 for year 2025 and 220 lpcd for the year 2050. For rural areas, 70 lpcd and 150 lpcd have been recommended for the year 2025 and 2050. Based on these norms and projection of population, it is estimated that by the year 2050, water requirements per year for domestic use will be 90 km$^3$ for low demand scenario and 111 km$^3$ for high demand scenario.

It is expected that about 70% of urban water requirement and 30% percent of rural water requirement will be met by surface water sources and the remaining from ground water.

### 1.3.2 Irrigation

The irrigated area in the country was only 22.6 million hectare (M-ha) in 1950-51. Since the food production was much below the requirement of the country, due attention was paid for expansion of irrigation. The ultimate irrigation potential of India has been estimated as 140 M-ha. Out of this, 76 M-ha would come from surface water and 64 M-ha from ground water sources. The quantum of water used for irrigation by the last century was of the order of 300 km$^3$ of surface water and 128 km$^3$ of ground water, total 428 km$^3$. The estimates indicate that by the year 2025, the water requirement for irrigation would be 561 km$^3$ for low demand scenario and 611 km$^3$ for high demand scenario. These requirements are likely to further increase to 628 km$^3$ for low demand scenario and 807 km$^3$ for high demand scenario by the year 2050.

### 1.3.3 Hydroelectric Power

The hydropower potential of India has been estimated at 84044 MW at 60% load factor. At the time of independence (1947), the installed capacity of hydropower projects was 508 MW. By the end of 1998, the installed hydropower capacity was about 22000 MW (24.85 % of the total installed capacity of 88543 MW). The status of hydropower development in major basins is highly uneven.

### 1.3.4 Industrial Water Requirement

Rough estimates indicate that the present water use in the industrial sector is of the order of 15 km$^3$. The water use by thermal and nuclear power plants with installed capacities of 40000 MW and 1500 MW (1990 figures) respectively, is estimated to be about 19 km$^3$. In view of shortage of water, the industries are expected to switch over to water efficient technologies. If the present rate of water use continues, the water requirement for industries in the year 2050 would be 103 km$^3$; this is likely to be nearly 81 km$^3$ if water saving technologies are adopted on a large scale.

### 1.3.5 Total Water Requirements

Total annual requirement of water for various sectors has estimated and its break up is given following table.

Table 1.6.Annual water requirement for different uses, in km$^3$

(Source:NCIWRD 1999)

| Uses | Year 1997-98 | Year 2010 | | | Year 2025 | | | Year 2050 | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | Low | High | % | Low | High | % | Low | High | % |
| Surface Water | | | | | | | | | | |
| Irrigation | 318 | 330 | 339 | 48 | 325 | 366 | 43 | 375 | 463 | 39 |
| Domestic | 17 | 23 | 24 | 3 | 30 | 36 | 5 | 48 | 65 | 6 |
| Industries | 21 | 26 | 26 | 4 | 47 | 47 | 6 | 57 | 57 | 5 |
| Power | 7 | 14 | 15 | 2 | 25 | 26 | 3 | 50 | 56 | 5 |
| Inland Navigation | | 7 | 7 | 1 | 10 | 10 | 1 | 15 | 15 | 1 |
| Environment – Ecology | | 5 | 5 | 1 | 10 | 10 | 1 | 20 | 20 | 2 |
| Evaporation Losses | 36 | 42 | 42 | 6 | 50 | 50 | 6 | 76 | 76 | 6 |
| Total | 399 | 447 | 458 | 65 | 497 | 545 | 65 | 641 | 752 | 64 |

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Ground Water | | | | | | | | | | |
| Irrigation | 206 | 213 | 218 | 31 | 236 | 245 | 29 | 253 | 344 | 29 |
| Domestic | 13 | 19 | 19 | 2 | 25 | 26 | 3 | 42 | 46 | 4 |
| Industries | 9 | 11 | 11 | 1 | 20 | 20 | 2 | 24 | 24 | 2 |
| Power | 2 | 4 | 4 | 1 | 6 | 7 | 1 | 13 | 14 | 1 |
| Total | 230 | 247 | 252 | 35 | 287 | 298 | 35 | 332 | 428 | 36 |
| Grand total | 629 | 694 | 710 | 100 | 784 | 843 | 100 | 973 | 1180 | 100 |
| Total Water Use | | | | | | | | | | |
| Irrigation | 524 | 543 | 557 | 78 | 561 | 611 | 72 | 628 | 807 | 68 |
| Domestic | 30 | 42 | 43 | 6 | 55 | 62 | 7 | 90 | 111 | 9 |
| Industries | 30 | 37 | 37 | 5 | 67 | 67 | 8 | 81 | 81 | 7 |
| Power | 9 | 18 | 19 | 3 | 31 | 33 | 4 | 63 | 70 | 6 |
| Inland Navigation | 0 | 7 | 7 | 1 | 10 | 10 | 1 | 15 | 15 | 1 |
| Environment – Ecology | 0 | 5 | 5 | 1 | 10 | 10 | 1 | 20 | 20 | 2 |
| Evaporation Losses | 36 | 42 | 42 | 6 | 50 | 50 | 6 | 76 | 76 | 7 |
| Total | 629 | 694 | 710 | 100 | 784 | 843 | 100 | 973 | 1180 | 100 |

# Lesson 2 Fresh Water and its Management in India

India is likely to face a major challenge in the management of freshwater in view of rapidly rising population and increasing agricultural, industrial and other requirements. As the economy of the country is currently witnessing rapid growth, management of freshwater resources becomes all the more important.

## 2.1. Freshwater Related Problems in India

To safeguard the economic and social prosperity of the country, it is imperative that enough freshwater is available to meet the requirements of agriculture, industries, and the domestic sector in the coming years. Unfortunately, inadequate water planning, lack of water awareness and non-implementation of desired measures, have created a difficult-to-manage situation. As a result an alarming scenario of freshwater scarcity is gradually unfolding in India. The water scarcity is already evident in many parts of India, varying in scale and intensity at different times of the year. This situation is the result of natural factors and human actions. Intense competition among water users – agriculture, industry and domestic sector – is pushing the groundwater table deeper and deeper. Widespread pollution of surface water and groundwater is degrading the quality of freshwater resources.

The major issues related to freshwater problems in India are elaborated in the subsequent sections.

### 2.1.1 Uneven Distribution of Water Availability

Water availability in India has large variations– both spatial and temporal. The basin wise per-capita water availability varies between 13,393m$^3$/year for Brahmaputra-Barak basin to about300m$^3$/year for Sabarmati basin. As per the international norms, if water availability is less than 1700m$^3$ per capita/year thenthe country is categorized as water stressed and if is less than1000m$^3$ per capita/year then the country is classified as waterscarce. Growing water scarcity in India can be gauged from thefact that the available water per capita per year has decreasedfrom 6008m$^3$ in 1947 to 2384m$^3$ in 2000. Although India isabove the water stressed category, the real situation of per capitawater availability is more disturbing than what is depicted by theaverage figures.India receives nearly 75–80% of annualprecipitation during the four monsoon months. Of the remainingamount, a large fraction is received during the winter monsoon.Further, out of 8760 hours in a year, most of the precipitation isreceived in about 100 hours. Instances where 10% of annual rainfalls in just 3 hours are not uncommon. Such a high concentrationof precipitation and streamflows makes it imperative to regulaterivers. Moreover, the uneven distribution of rainfall across thecountry at different times of the year makes several parts of India fall under the water stressed, water scarcity and absolute water scarcity category, as shown in Fig. 2.1.

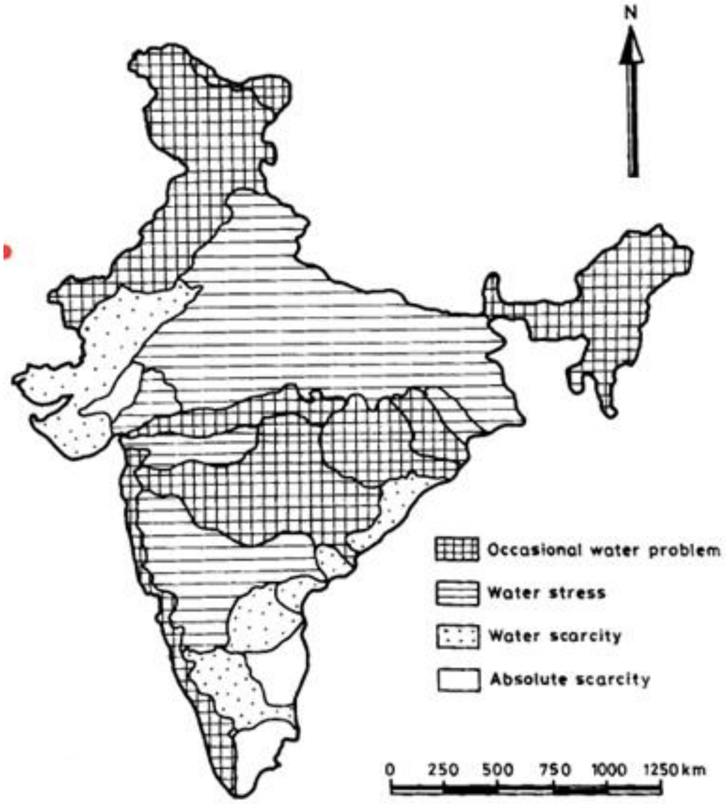


Fig.2.1.Water availability in Indian river basins. (Source: Chitale, 1992)

### 2.1.2 Water Pollution

Water pollution is acquiring serious dimensions in India as almost 70% of its surface water resources and a large proportion of groundwater reserves are already contaminated by biological, toxic organic and inorganic pollutants. Degradation of quality in turn leads towater scarcity as it limitswater availability for human use. Sources of water pollution are diverse: untreated sewage, industrial discharges, leaching from municipal waste, and drainage from the residues of agricultural fertilizers and pesticides. With burgeoning cities and increasing industrialization, the quantum of waste dumped into rivers has also increased.Water pollution varies in severity from one region to the other depending on the density of urban development, agricultural and industrial practices, and the systems for collecting and treating wastewater. Most of the polluted stretches exist in and around large urban areas.

Some of the agricultural, industrial and domestic sources of water pollution are described below.

a) Agriculture: The indiscriminate use of agro-chemicals has contributed significantly to the pollution of both surface water and groundwater resources. The consumption of pesticides, which rose from less than 1 million tonnes in 1948 to 66.36 million tonnes during 1994–95, was around 43.59 million tonnes during 2001–02. Some of the chemicals in these fertilizers and pesticides, which enter water bodies through runoff and leaching, are considered hazardous by the World Health Organization (WHO). Water quality studies on the Ganga River indicate the presence of chemicals such as HCH, DDT, dimethoate, endosulfan and malathion in quantities exceeding standards set by international organizations. Severe soil erosion and water quality degradation (in the form of increase in sediment load) due to improper land management practices are particularly noticeable in the mountainous regions in northern and western India.

b) Industry: Although the industrial sector accounts for about 4% of the annual water withdrawals, its contribution to water pollution, particularly in urban areas, is significant. Wastewater generation from this sector has been estimated at 55,000 million $m^3$ per day, out of which 68.5 million $m^3$ is discharged into river streams. Of the total pollution load, 40%–45% is contributed by the processing of industrial chemicals, while nearly 40% of the total organic pollution, expressed as BOD, arises from the food industries followed by industrial chemicals and the pulp and paper industry.

c) Domestic: The domestic sector is responsible for majority of the wastewater generation in India. About 50 million$m^3$ of untreated sewage discharged into rivers have contributed towards pollution of India's fourteen major river systems. The 22 largest cities in the country produce over 7300 million litres of domestic wastewater per day and only about 80% of it is collected for treatment. Inadequate treatment of human and animal wastes adds to the high incidence ofwater-related diseases.

### 2.1.3 Excessive Groundwater Exploitation

Large-scale extraction of groundwater has lead to overdraft and a drastic fall in water table in some basins. This in turn has created a chaotic situation especially in the water scarce hard-rock regions of southern India, where assured sources of surface irrigation are rare and rainfall is non-uniform. Currently, about 32% of the annual utilizable groundwater potential of 432 $km^3$ is actually exploited, and only 8% of the groundwater sources have been exploited above 85% of their potential. However, in states like Punjab, Rajasthan and Tamil Nadu, large areas fall under the dark category. Table 2.1 shows the ten states where the percentage of dark areas has increased considerably between 1984–85 and 1997–98. In coastal regions, e.g., in Tamil Nadu and Gujarat, regional decline in water table has resulted in saltwater encroachment in the aquifer systems. Groundwater sources have been classified in three categories depending upon the extent of exploitation. In the 1st category (termed 'white'), the level of exploitation is below 65% of the annual utilizable potential. The 2nd category (termed 'gray') includes areas and sources in the range of 65% to 85% exploitation levels and the third and the worst category (termed 'dark') has the level of exploitation exceeding 85%.

Table 2.1.Blocks with intensive exploitation of groundwater

(Source: Chaddha, 2002)

| State | Number of dark blocks | | |
|---|---|---|---|
| | 1984-1985 | 1992-1993 | 1997-1998 |
| Andhra Pradesh | 0 | 30 | 26 |
| Bihar | 14 | 1 | 11 |
| Gujarat | 6 | 26 | 28 |
| Haryana | 31 | 51 | 41 |
| Karnataka | 3 | 18 | 16 |
| Madhya Pradesh | 0 | 3 | 3 |
| Punjab | 64 | 70 | 83 |
| Rajasthan | 21 | 56 | 94 |
| Tamil Nadu | 61 | 97 | 103 |
| Uttar Pradesh | 53 | 31 | 40 |
| Total | 253 | 383 | 445 |

### 2.1.4 Threat to Biodiversity and Wetlands

About 6.5% and 12.5% of the world’s animal and plant species, respectively, can be found in India. Out of these almost 7,000 are endemic to the subcontinent. Unfortunately, habitat destruction in both freshwater and coastal areas has endangered many endemic species. Most vulnerable are the freshwater fish since they are more susceptible to water pollution and environmental change. Other endangered species include freshwater aquatic animals like the Gangetic dolphin and several species of aquatic birds, amphibians, reptiles and insects.

Wetlands in India cover a land area of about 4.1 million hectares. Most of these have become degraded due to pollution and development pressures, like conversion of wetlands for agriculture. This is threatening not only the local fauna but also the livelihood of the residents dependent on the wetland ecosystem. In coastal areas, industrial and domestic pollution has severely degraded estuarine and coastal environments.

To summarize, the root causes of the freshwater crisis inIndia are:

1. Rampant pollution of freshwater resources mainly by the agricultural, industrial and municipal activities.
    1. Inadequate attention to water conservation, efficiency in water use, water re-use, groundwater recharge, and ecosystem sustainability.
    2. Very low water prices which do not discourage wastage.
    3. Prevalent system ofwater rights which gives unlimited ownership of groundwater to the landowner, despite the fact that groundwater is a shared resource from common pool aquifers.
    4. Uncontrolled use of the bore-wells that has allowed extraction of groundwater at very high rates, often exceeding recharge.
    5. Communities are not partners in managing water resources.

### 2.2 Strategies for Freshwater Management in India

As per the National Water Policy (2002) of the Government of India, water allocation priorities in the planning and operation of systems should broadly be:

1. drinking water,
2. irrigation,
3. hydropower,
4. ecology,
5. agro-industries
6. non-agricultural industries and
7. navigation

In view of the current status of freshwater in India and the problems that are likely to arise in future, a well-planned long-term strategy is needed for sustainable water resources management in India. Some key aspects of such a strategy are proposed next.

#### 2.2.1 Water Conservation

Broadly speaking, water conservation implies improving the availability of water through augmentation by means of storage of water in surface reservoirs, tanks, soil, and groundwater zone. It emphasizes the need to modify the space and time availability ofwater to meet the demands. This concept also highlights the need for judicious use of water. If one looks at utilizable water resources in major river basins, these resources in Indus, Ganga, Brahmaputra, and Godavari basins are 73.31, 525.02, 629.05 and 110.54 km$^3$ per year, respectively.The storages available in these basins, including projects under construction, are 16.28, 54, 3.5, and 30.16 km$^3$. Thus, only a small fraction of available water is being regulated in these basins at present. These basins are subject to frequent flooding, making the argument of storage even stronger. Overall, out of 690 km$^3$ of utilizable surface water, storage capacity created so far is only 177 km$^3$, ongoing projects will add another 70 km$^3$and those under planning 132 km$^3$. Thus, even after completing the planned projects, 45% of the potential will remain unutilized. In viewof rapidly rising population and demands forwater, it will be necessary to conserve adequate quantity of water for later use.

No matter how freshwater is used – whether for agriculture, industry, or domestic purposes – there is a great potential for better conservation and management. On the demand side, a variety of economic, administrative and community-based measures can help conserve water. Side-by-side, it is necessary to control thegrowth of population since large population is putting massive stress on all natural resources. Since agriculture accounts for about 83% of all water withdrawn, the greatest potential for conservation lies in increasingirrigation efficiencies.

In urban water supply, for example, almost 30% of the water is wasted due to leakage and other losses, while most metro cities face deficit in supply of water. It is, therefore, imperative to prevent wastage.

### 2.2.2 Watershed Management

Watershed management aims to establish aworkable and efficient framework for the integrated use, regulation and development of land and water resources in a watershed for socio-economicgrowth. Typical objectives of watershed development programsinclude:

a) Raising the productivity of rain-fed agriculture and non-arable lands

b) Encouraging the sustainable management and optimal useof surface and groundwater,

c) Reducing soil erosion,

d) Conserving forests and other natural vegetation,

e) Creating employment (both directly and indirectly), and

f) Promoting increased individual and collective responsibility for natural resources management and strengthening the social institutions.

### 2.2.3 Water Quality Conservation and Environment Restoration

To preserve our water and environment, we need to make systematic changes in the way we grow our food, manufacture the goods, and dispose off the waste.

a) Water quality: In India, agriculture is the biggest user and polluter of water. If pollution by agriculture is reduced, it would improve water quality and would also eliminate cost incurred for treatment of diseases. This would entail learning how to use less chemicals while boosting yields, e.g., eliminating use of fungicides by planting more diverse varieties of grains and switching to organic farming so that fewer chemicals are introduced on farms. Industries need to carefully treat their waste discharges. Manufacturers may reduce water pollution by reusing materials and chemicals and switching over to less toxic alternatives. Industrial symbiosis, in which the unusable wastes from one product/firm become the input for another, is an attractive solution.

b) Environment Restoration: Environmental improvement and restoration should be planned and implemented such that the freshwater resources are protected and their quality is maintained. Model efforts in this direction include the capture, storage and safe release of water and the prevention of accelerated soil erosion through hydraulic structures and vegetation. While utilizing water and land resources, their ability to serve other uses is often degraded either inadvertently or due to carelessness. Efforts should be made to restore landscapes and ecosystems to more efficiently protect water quality, aquatic andwildlife. On the legislative front, we require laws to check litteringas well as to implement “polluter pays” principle.

### 2.2.4 Inter-basin Water Transfer (IBWT)

The vast variation, both in space and time, in the availability of water in different regions of India has created a food-drought flood syndrome with some areas suffering from flood damages and other areas facing acute water shortage. The drought prone area assessed in the country is of the order of 51.12 million hectare, while the area susceptible to floods is around 40 million hectares. The States of Karnataka, Tamil Nadu, Rajasthan, Gujarat, Andhra Pradesh and Maharashtra are the worst drought prone States. The States of Uttar Pradesh, Bihar, West Bengal, Orissa and Assam face severe flood problems. Inter basin transfer of water in India is a long-term option to partly overcome the spatial and temporal imbalance of availability and demand of water resources. The transfer of water from surplus areas to deficit areas is not a new concept. Many such schemes have been implemented all over theworld. In India too, projects like Periyar –Vaigai system, Indira Gandhi canal, and Telugu Ganga stand as classic examples of inter-basin water transfer. In the seventies, the Garland Canal proposal of Captain Dastur and the Ganga – Cauvery Canal proposal of Rao (1973) were received with considerable attention. A National Perspective Plan (NPP) for water resources development was formulated by the Government of India in 1980s. The distinctive feature of the NPP is that the transfer of water is essentially by gravity and only in small reaches by lift pumping (not exceeding 120 m). This plan comprises of two components:

1. Himalayan Rivers Development, and
   2.Peninsular Rivers Development

While the second component will be an inter-state venture, the first will involve neighbouring countries too and thus will be an international venture. Some of the major benefits expected from inter linking of the rivers are:

1. Irrigation potential is to increase from 140 to 175 million ha,
   1. Drinking water availability is to increase by about 12 km3,
   2. Peak flood discharge to get reduced by about 30% due to construction of reservoirs,
   3. Generation of 34000MW of electricity, and
   4. Possibilities of inland navigation to provide cheap transport.

While planning interbasin transfer of water, it may be noted that water has to be shared in two ways:

1. between its different uses (energy, cities, food production, environment, etc.), and
2. between users (administrative blocks or states sharing a river basin or aquifer).

Many regions and cities rely on upstream users for water flow and any downstream user will be dependent on the action of the upstream users. Measures used to allocate water between competing uses may include:national strategy and/or legislation on inter-sectoral allocations, tariff disincentives and targeted subsidies, abstraction management, application and enforcement of water-quality objectives, reservoir operating rules, multi-use reservoir management, multi-reservoir system management and reservoir compensation flow releases (UN, 2003). A diagram depicting how IBWT will lead to increased utilization of water resources is shown in Fig. 2.2.

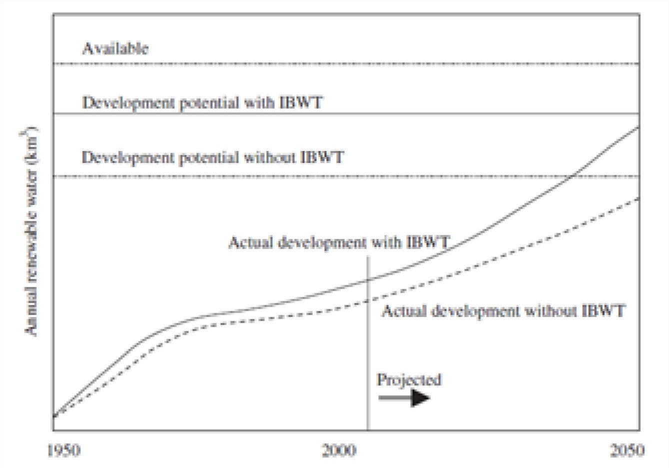


Fig.2.2. Diagram showing the effect of IBWT on utilization (Curve for the future depict expected scenario) of water resources. (Source: Thatte,2003)

### 2.2.5 Groundwater Management

To protect the aquifers from overexploitation, an effective groundwater management policy oriented towards promotion of efficiency, equity, and sustainability is required. The exploitation of groundwater resources should be regulated so as not to exceed the recharging possibilities, as well as to ensure social equity. Integrated and coordinated development of surface water and groundwater resources and their conjunctive use should be envisaged right from the project planning stage and should form an integral part of the project implementation. Over exploitation of groundwater should be avoided, especially near the coasts to prevent ingress of seawater into freshwater aquifers.

The government can initiate a variety of programs and controls for recharge and discharge and implement regulatory measures such as well spacing norms, control drilling of new wells by issuing permits, regulation of water intensive crops, and pricing of electricity for lifting groundwater. To the extent possible, conjunctive use of surface water and groundwater should form an integral part of groundwater management policy. In critically overexploited areas, bore-well drilling should be regulated till the water table attains the desired elevation. Artificial recharge measures need to be urgently implemented in these areas. Amongst the various recharge techniques, percolation tanks are least expensive in terms of initial construction costs. Many such tanks already exist but a vast majority of these structures have silted up. In such cases, cleaning of the bed of the tanks will make them reusable.

### 2.2.6 Rainwater Harvesting

Rainwater harvesting is the process to capture and store rainfall for its efficient utilization and conservation to control its runoff, evaporation and seepage. Some of the benefits of rainwater harvesting are:

1. It increases water availability,
2. It checks the declining water table,
3. It is environmentally friendly,
    1. It improves the quality of groundwater through dilution, mainly of fluoride, nitrate, and salinity, and
    2. It prevents soil erosion and flooding, especially in the urban areas.

### 2.2.7 Recycle and Reuse of Water

Another way through which we can improve freshwater availability is by recycle and reuse of water. Use of water of lesser quality, such as reclaimed wastewater, for cooling and fire fighting is an attractive option for large and complex industries to reduce their water costs, increase production and decrease the consumption of energy. This conserves better quality waters for potable uses. Currently, recycling of water is not practiced on a large scale in India and there is considerable scope and incentive to use this alternative.

### 2.2.8 Desalination of Water

Since 1970, there has been significant commercial development using various desalination technologies, including distillation, reverse osmosis, and electrolysis. This technology is suitable for use in areas where freshwater is scarce, but saline water is available and energy is cheap. Compared to water recycling technologies, desalination presents fewer health risks.

### 2.2.9 Environmental Flow Requirement (EFR)

An environmental flow (EFR) is the water regime provided within a river, wetland or coastal zone to maintain ecosystem and their benefits where there are competing water uses and where flows are regulated. Environmental flows normally include the flow requirements in rivers and estuaries for maintenance of riverine ecology.

### 2.2.10 Dealing with Climate Change

Climate change is likely to result in hydrologic conditions and extremes of a nature that will be different from those for which the existing projects were designed. Some recommendations to cope up with the problems in a systematic and a planned manner are:

1. A nationwide climate monitoring program should be developed;
2. While formulating new projects that influence climate, it should be ensured that no action is taken which causes irreversible harmful impact on the climate;
3. Improved methods for accounting of climate related uncertainty should be developed and made part of decision making process;
4. Existing systems should be examined to determine how they will perform under the climate situations that are likely to arise;

5. Water availability and demands in all regions, particularly in water scarce regions should be reassessed in the new climate scenario;
6. Are-examination of the operating rules should be taken up to see how these need to be updated to handle likely extremes.

# Lesson 3 Introduction to Hydrology

## 3.1 Definition of Hydrology

Hydro means water, and logy means study. Therefore, in a broad sense hydrology can be defined as a study of water. A more practical definition is given below:

Hydrology can be defined as the science as the science that deals with the space-time characteristics of the quantity and quality of the waters of the earth, encompassing their occurrence, movement, distribution, circulation, storage, exploration, development, and management. These characteristics are determined by the relation of water to the earth.

The definition of hydrology encompasses some aspects of a multitude of disciplines involving agriculture, biology, chemistry, geography, geology, glaciology, meteorology, oceanography, physics, volcanology and many other disciplines. Many branches of hydrology have been distinguished (Fig. 3.1) because of the close association of water with the atmosphere and the earth.

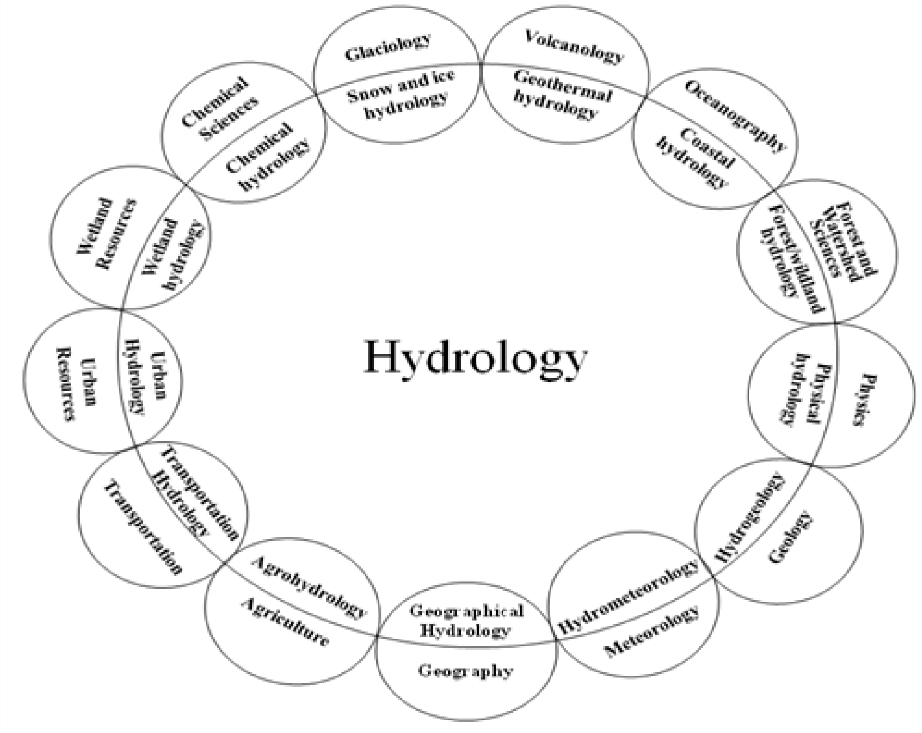


Fig.3.1.Classification of hydrology according to its association with other branches of science. (Source: Singh, 1994)

The techniques for solving hydrologic problems are borrowed from several disciplines such as mathematics, statistics, probability theory, operation research, control theory, information theory, and others. Based on the treatment of hydrologic data by these techniques, hydrology can be sub-divided into different branches (Fig 3.2).

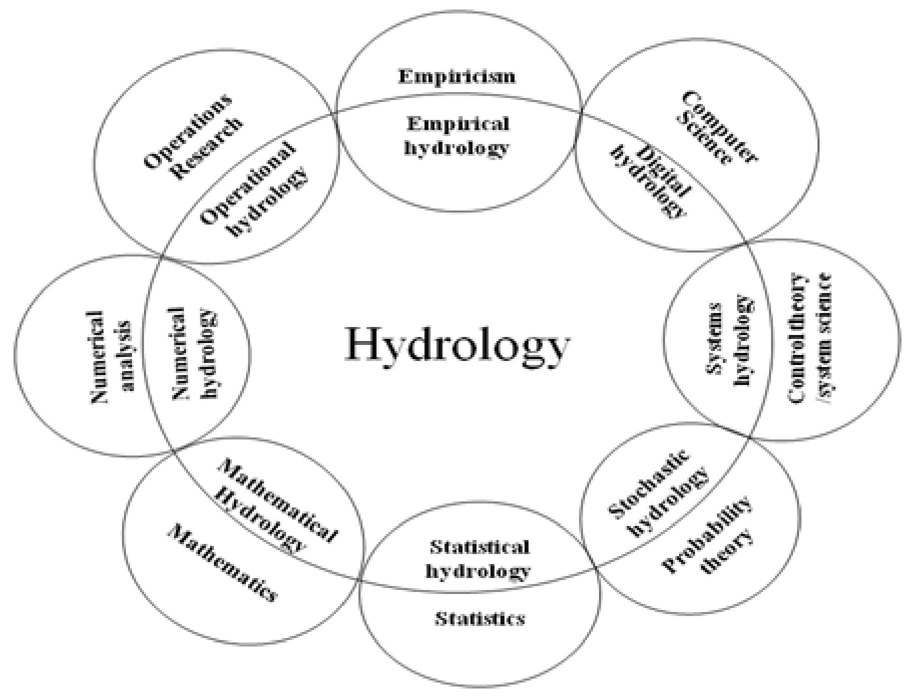


Fig. 3.2.Classification of hydrology according to methods of solution.(Source: Singh, 1994)

## 3.2 Hydrologic Cycle

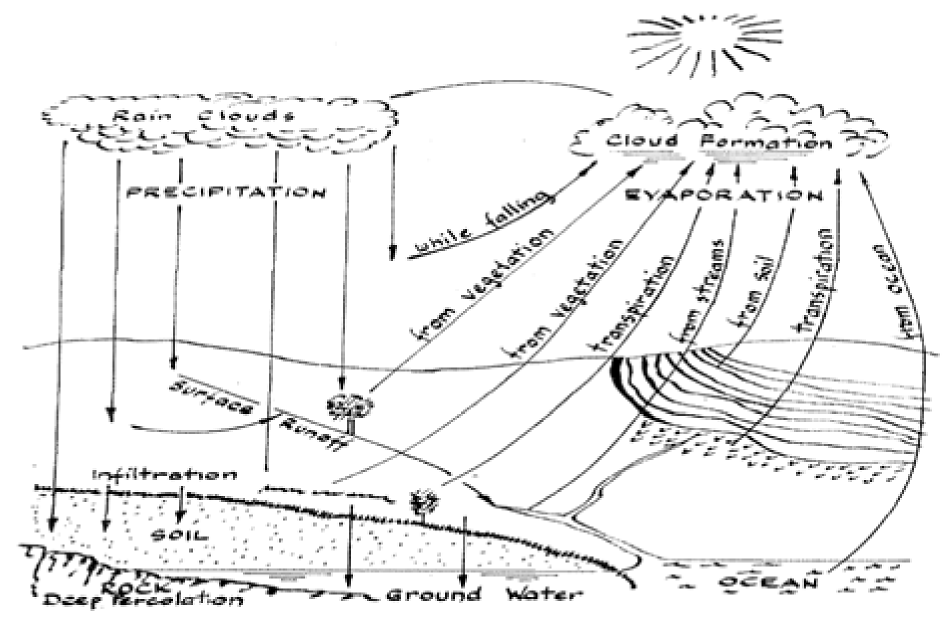


Fig. 3.3.Descriptive representation of the hydrologic cycle.

(Source: Ackermann et. al., 1955)

Water on the earth exists in a space called the hydrosphere which extends about 15 km into the atmosphere and about 1 km down into the lithosphere, the crust of earth. Water circulates in the atmosphere through maze of paths constituting the hydrologic cycle. This cycle has no beginning or end, and many processes (known as hydrological processes) occur continuously. A schematic of hydrologic cycle is given in Fig. 3.3.

Water evaporates from the water bodies (lakes, rivers, oceans and other free water bodies), land surface, and transpires from vegetation (forests, cropland and other vegetation) due to the energy from the sun, thereby providing supply of vapor to the atmosphere. Water vapor is transported and lifted in the atmosphere until it condenses and precipitates on the land or oceans; precipitated water may be intercepted by vegetation, become overland flow over the ground surface, infiltrate into the ground, flow through the soil as subsurface flow, discharge into streams as surface runoff. Much of the intercepted water and surface runoff returns to the atmosphere through evaporation and transpiration. The infiltrated water may percolate deeper to recharge ground water, later emerging as springs or seeping into streams to form surface runoff, and finally flowing to the sea or evaporating into the atmosphere as the hydrologic cycle continues.

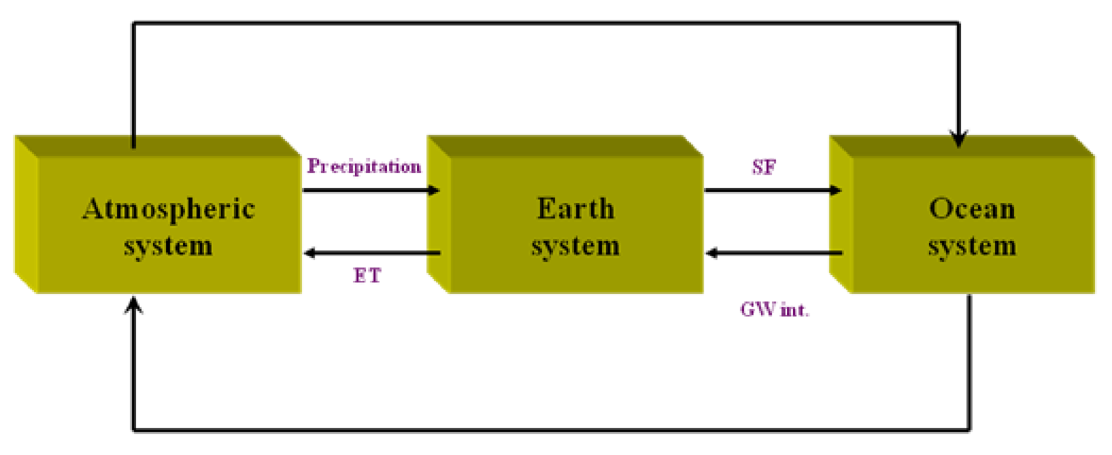


Fig. 3.4.A global schematic of the hydrologic cycle.(Source: Singh, 1994)

Figs. 3.4 through 3.7shows hydrologic cycle at global scale and various hydrologic processes involved in the earth and land systems. The concept of the hydrologic cycle is simple, but the phenomenon is enormously complex and intricate. It is not just one large cycle but rather is composed of several interrelated cycles of continental, regional, and local extent.

Furthermore, the total volume of water in the global hydrologic cycle remains essentially constant (conservation of mass), the distribution of this water is continually changing in both space and time.

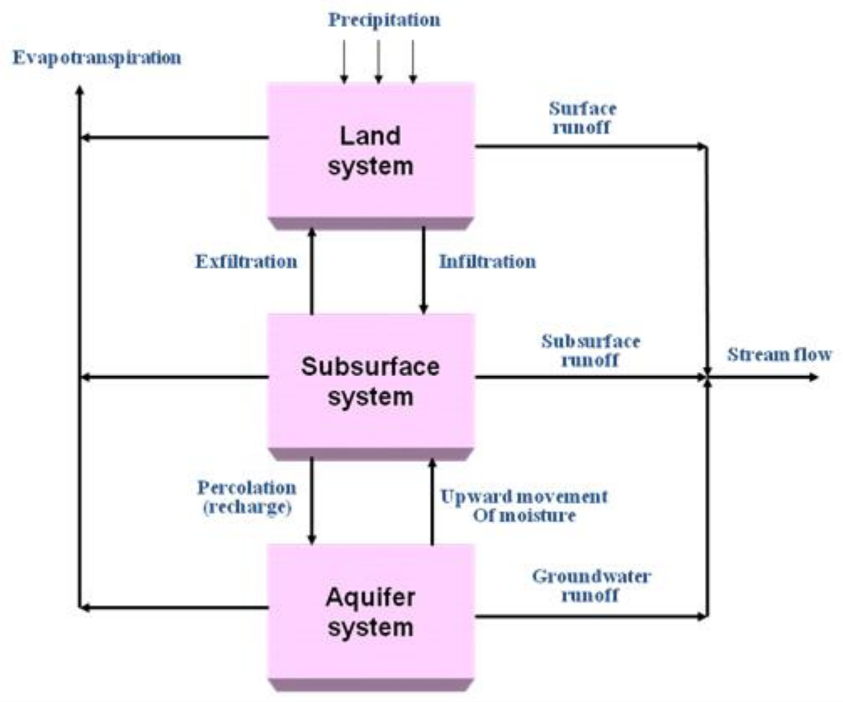


Fig. 3.5.A schematic of the hydrologic cycle in the earth system.

(Source: Singh, 1994)

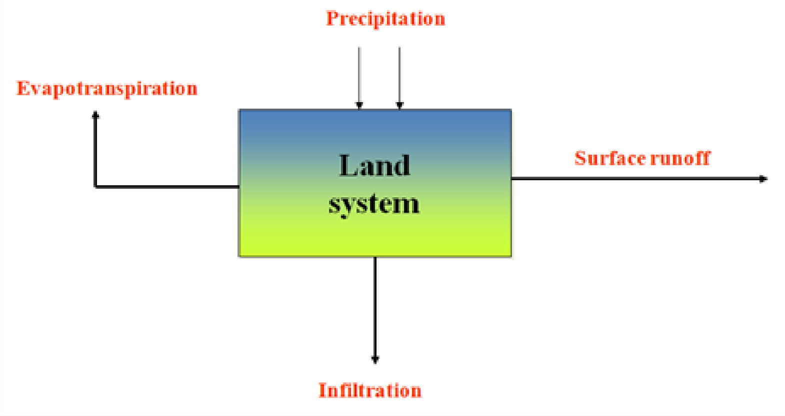


Fig. 3.6.A schematic of the hydrologic cycle in the land system.

(Source: Singh, 1994)

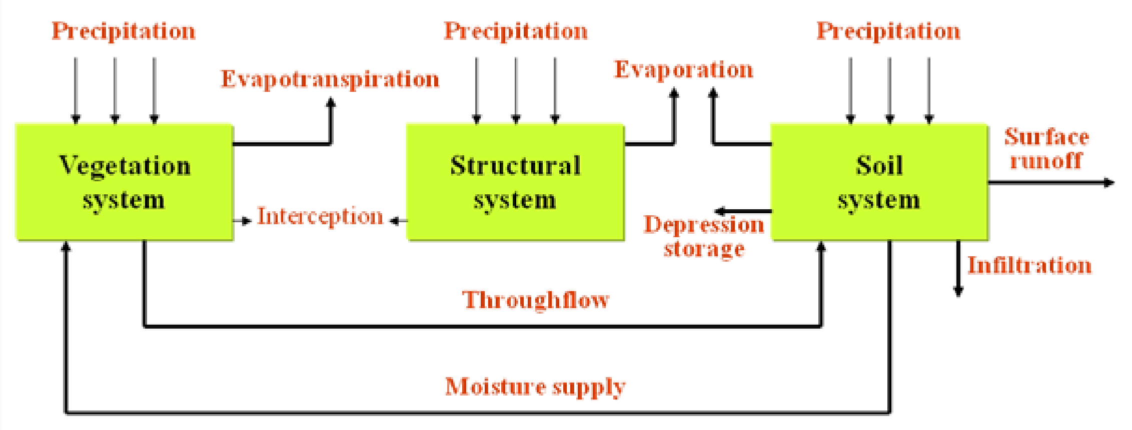


Fig. 3.7.A more complete schematic of the hydrologic cycle in the land system.(Source:Singh, 1994)

The hydrology of a region is determined by its weather patterns and by physical factors such as topography, geology, and vegetation. Also, as civilization progresses, human activities gradually encroach on the natural water environments, altering the dynamic equilibrium of the hydrologic cycle and initiating new processes and events.

## 3.3 Applications of Hydrology

The most important uses of hydrology are flood control, drought mitigation, water supply, pollution control, urban and industrial development, design of hydraulic works, agricultural production, land conservation, environmental impact assessment, land use change, navigation, recreation and fisheries.

### 3.3.1 Flood Control

**Floods occurs when**

1. water body (lake, reservoir, or channel) is unable to contain the amount of the water it receives,
    1. there is inadequate drainage provision to drain excess water, and
    2. Hydraulic structures such dams, levees and dykes are failed.

The hydrologic inputs needed to design flood mitigation and control projects (structural, e.g. dams, levees, diversions, channels, and non-structural e.g. flood plain management, flood roofing or both) includes:

1. Peak discharge and its frequency of occurrence,
    1. Duration and volume of flood hydrograph and their probability of occurrence, and
    2. Arrival of next flooding.

### 3.3.2 Drought Mitigation

Drought occurs when there is a shortage of water by comparison with the demand for it. Droughts are usually distinguished as agricultural, hydrological and meteorological. The problem of drought is defined by its areal extent, duration, severity, and the onset of next drought. From hydrological perspective, low discharge (defined over a period) and its frequency of occurrence, duration and volume of low flow, and the probability of occurrence of the next drought are useful to design drought mitigation projects.

Construction of water impoundments, ground water pumpage, interbasin transfer, water conservation, and even augmentation of atmospheric precipitation through cloud seeding are some of the ways to mitigate droughts.

### 1. Design of Hydraulic Works

Dams, culverts, spillways, bridge crossings, dykes, levees, diversions, drainage works etc. are typical hydraulic works required for water resources development and management. Design of these works requires an estimate of peak discharge of given frequency. Also estimated using hydrology is the environmental consequence of these works.

### 2. Agricultural Production

Agriculture is the largest water user and for sustainable agricultural production efficient water management is essential. Proper agricultural practices conserve precipitation for crop use, prevent the loss of precious soil, and preserve the quality of stream that drains the land. Crop production involves moisture forecasting, supply of water to farms, management of irrigation water, application of chemical and fertilizers, drainage of excess water, soil conservation etc.

Hydrology is used to determine irrigation scheduling, soil erosion and sediment transport, migration of chemical and their impact on water quality. It also used to design a network of wells for a farm, drainage system to remove excess water, and water conveyance system network (dams, canals, and ditches) based on soil properties, land slope, location of the water table, climate and other factors.

### 3. Land Conservation

Careless farming method, deforestation, over-grazing can speed up runoff of rainfall, resulting in erosion of soil. This increases the danger of flooding downstream and also increases reservoir sedimentation rate. Loss of fertile lands due to erosion and of coastal areas has been growing concern.

Hydrology is used to determine the space-time history of erosion and to develop scenarios for prevention of erosion through, soil conservation, appropriate farm practices, vegetation management, water diversion, afforestation, reduced flooding and controlled land use.

Module.2 Precipitation

## Lesson 4 Forms and Types of Precipitation

### 4.1 Forms of Precipitation

Precipitation occurs in many forms e.g. drizzle, rain, glaze, sleet, snow, hail, dew and frost, depending upon the causes and temperature at the time of formation. Dew is condensation on the ground of atmospheric vapor caused by radiational cooling of the lower layers of atmosphere, usually at night. Frost is dew formed under freezing conditions. Dew and frost are quantitatively unimportant and rarely measured.

1. Drizzle: Drop size < 0.5 mm in dia. and intensity is usually < 1 mm/hr and generally occurs in conjunction with warm frontal lifting.
2. Rain: Drop size is between 0.5 to 6 mm in dia. Drops bigger than 6 mm tend to break up as they fell. It is formed by condensation and coalescence of cloud droplets at temperatures above the freezing point.
3. Glaze: It is the ice coating formed when drizzle or rain freezes as it comes in contact with cold objects on the ground.
4. Sleet: It is frozen raindrops cooled to ice stage while falling through air at sub-freezing temperature.
5. Snow: It is a precipitation in the form of ice crystals resulting from sublimation, i.e., directly from water vapor to ice.
6. Snow Flake: It is made of a number of ice crystals fused to gather.
7. Hail: It is precipitation in the form of balls or lumps of ice over 5 mm diameter formed by alternate freezing and melting as they are carried up and down in highly turbulent air currents.

### 4.2 Mechanisms for Production of Rainfall

The following four conditions are necessary for the production of rainfall.

4.2.1 Mechanism to produce cooling of the air – The pressure reduction due to ascending air from surface to upper levels in the atmosphere is the only known mechanism capable of producing large drops in the temperature.

4.2.2 Mechanism to produce condensation – Condensation in the atmosphere takes place on "hygroscopic nuclei" small particles of substances that have an affinity for water.

4.2.3 Mechanism for droplet growth – A tendency for the droplets to remain small and therefore to resist falling is called "colloidal stability". The most effective processes for droplet enlargement are,

1. the difference in speeds between large and small droplets, and
2. The co-existence of ice crystals and water droplets.

4.2.4 to produce accumulation of moisture of sufficient intensity to account for the observed rates of rainfall – Regardless of whether or not the other conditions for precipitation are fulfilled, continuity considerations demand that there must be a good amount of moisture present in the atmosphere so that evaporation losses between ground and cloud be compensated, if there is to be appreciable rain.

### 4.3 Types of Precipitation

There are three major types of precipitation: cyclonic, convective, and orographic. Each type represents a different method of lifting an air mass, resulting in cooling and condensation of atmospheric water vapor.

4.3.1 Cyclonic Precipitation: It is caused by lifting associated with the horizontal convergence of inflowing atmosphere into an area of low pressure. There are two kinds of cyclonic precipitation. Non-frontal precipitation involves only this convergence and lifting. Frontal precipitation results when one air mass is lifted over another. A front is defined as the boundary between two air masses of different temperatures and densities. The types of fronts and their commonly associated precipitation are described below.

A warm front is the result of a warm air mass overriding a cold air mass, causing extensive areas of cloudiness and precipitation. As the warm front approaches a given area, the precipitation becomes more continuous and intense. Warm fronts move at a speed of 15-50 km/h (10-30 mph).

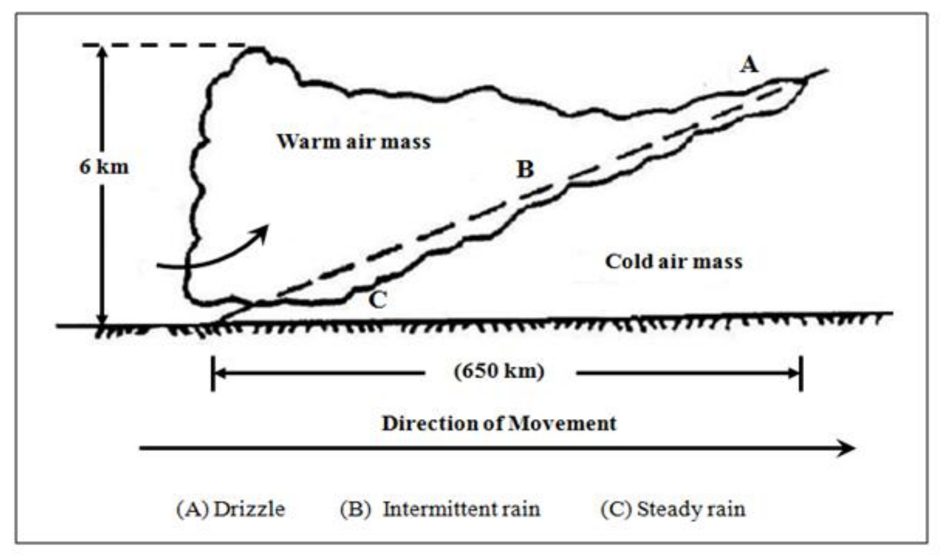


Fig.4.1.Warm Front.(Source:Singh, 1994)

A cold front results from a strong push of a cold air mass against and beneath a warm air mass. At the front towering clouds develop together with intense short duration precipitation. Cold fronts move at a speed of 30-80 km/h (20-50 mph).

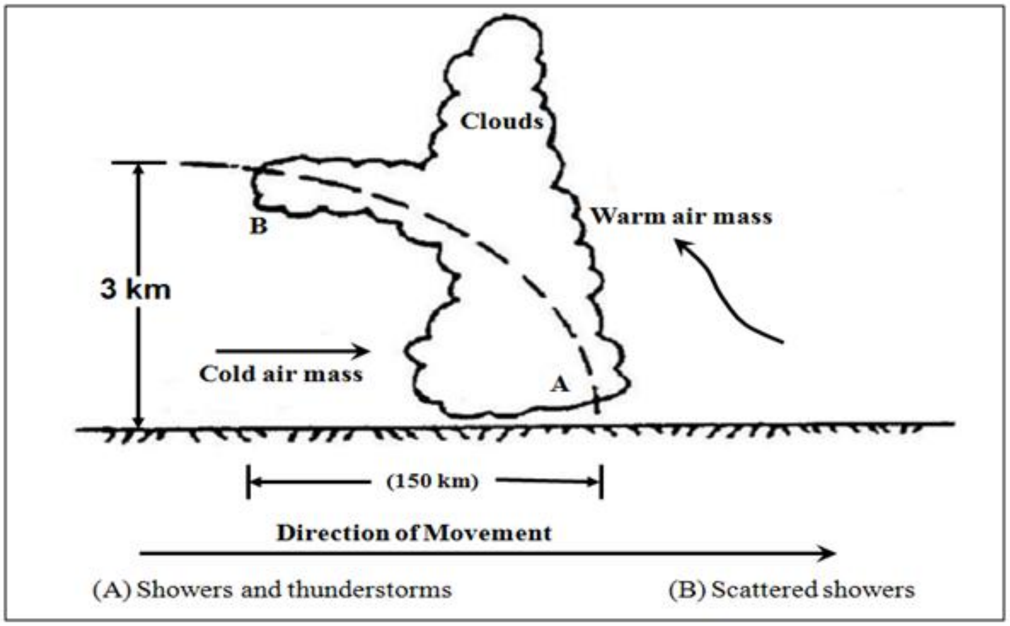


Fig.4.2.Cold Front.(Source:Singh, 1994)

An occluded front occurs when a cold front overtakes a warm front. The precipitation pattern is a combination of both warm and cold frontal distribution. Occluded fronts move at a speed of from 8-50 km/h (5-30 mph).

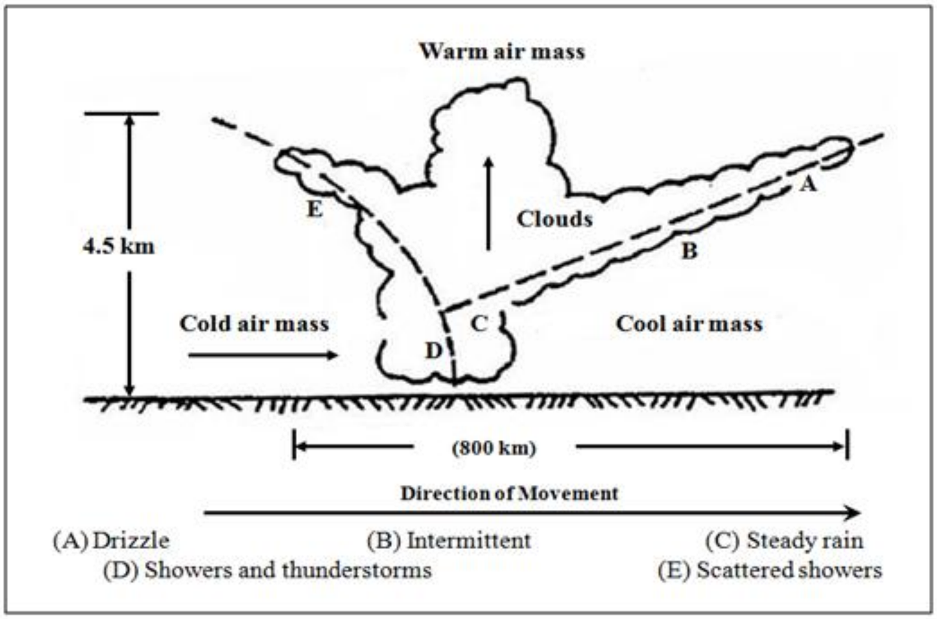


Fig.4.3.Occluded Front.(Source: Singh, 1994)

### 4.3.2 Convective Precipitation:

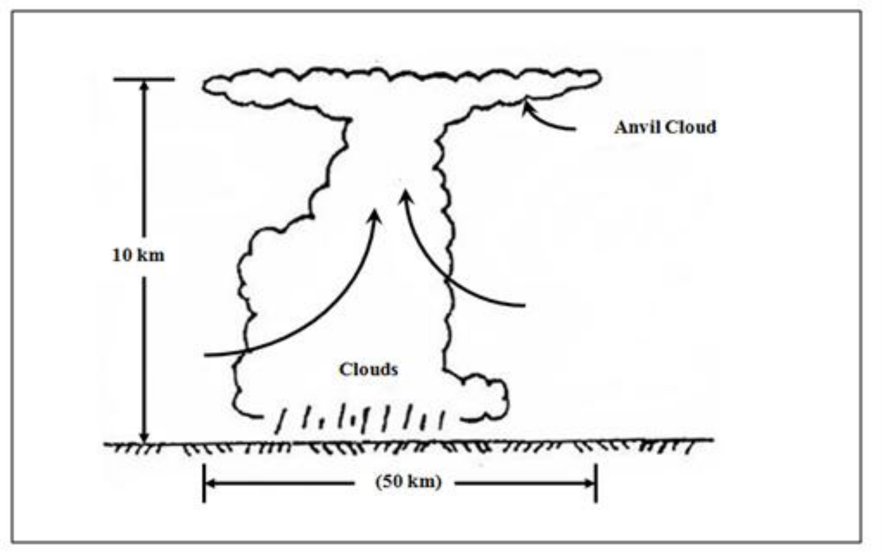


Fig.4.4.Thunderstorm.(Source:Singh, 1994)

It results when air that is warmer than its surrounding rises and cools. The precipitation is of a shower type, varying from light showers to cloudbursts. The typical thunderstorms resulting from heating of the atmosphere in the afternoon hours is the best example of convective rainfall. Thunderstorms occur throughout the world, especially in the summer. They are the characteristic form of rain in the tropics, wherever cyclonic circulation does not operate. A cross section through a typical thunderstorm is shown in the Fig 4.4

4.3.3 Orographic Precipitation: It is caused when air masses are lifted as they move over mountain barriers. Such orographic barriers tend to increase both cyclonic and orographic precipitation due to the increased lifting involved. Precipitation is generally heavier on the windward slope than on the leeward slope.

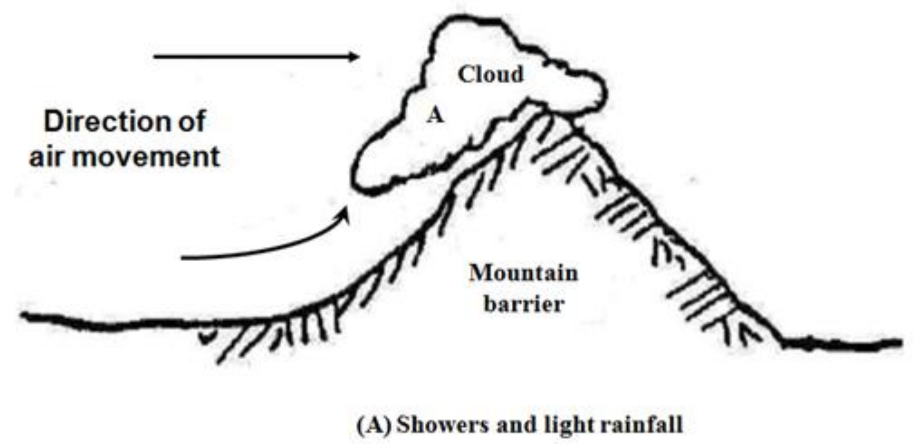


Fig.4.5.Orographic Precipitation.(Source:Singh, 1994)

## Lesson 5 Rainfall Measurement

### 5.1 Introduction

Mostly estimates of runoff are made based on rainfall data, amount and intensity of rainfall should be known by measurement. Rainfall data is also required for calculating irrigation requirements.

5.1.1 Raingage: The purpose of the raingage is to measure the depth and intensity of rain falling on a flat surface without considering infiltration, runoff or evaporation. The problems of measurements include effects of topography, nearby vegetation and the design of gage itself.

5.2 Types of Raingages: There are mainly two types of raingages (non-recording and recording).

### 5.2.1 Non-recording Gage

The standard raingage, known as Symon's gage is recommended and installed by the Indian Meteorological Department. This is a vertical, cylindrical container with top opening 127 cm in diameter. A funnel shaped hood is inserted to minimize evaporation losses. The water is funneled into an inner cylinder.

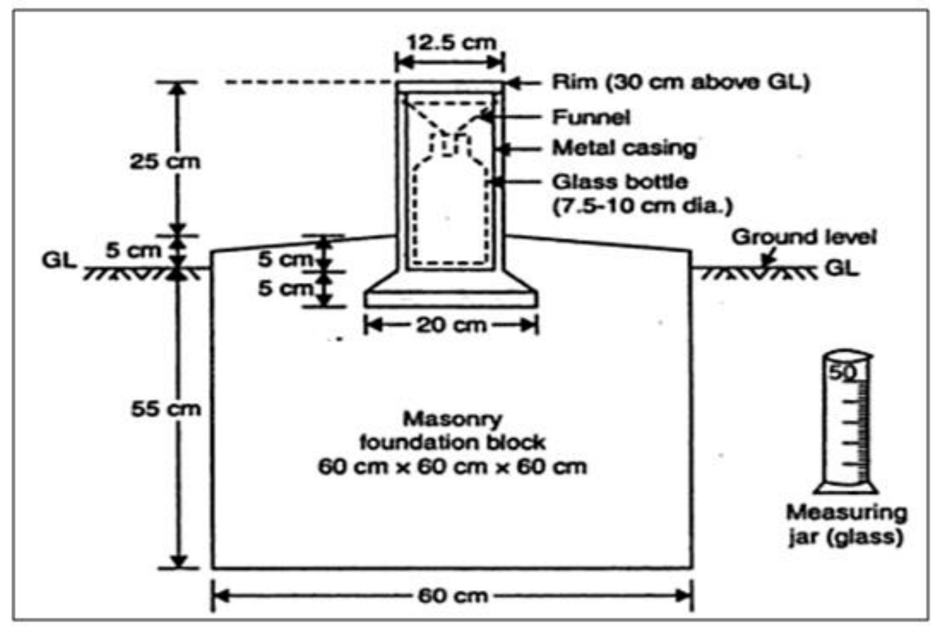


Fig. 5.1.Symon'sRaingage. (Source: Raghunath, 2006)

Considerations for Installation

1. The site should be an open place,
2. The distance between the raingage and the nearest object should be at least twice the height of the object,
3. As for as possible it should be a level ground,
4. In the hills, the site should be so chosen where it is best shielded from high winds and wind does not cause eddies, and
5. If a fence is erected, it should be atleast at a distance of twice the height.

### 5.2.2 Recording or Automatic Raingage

5.2.2.1 Weighting Bucket Type Raingage - This gage weighs the rain, which falls into a bucket set on a platform of a spring or level balance. The increasing weight of bucket and its counts are recorded on the chart held by a clock driven drum. The record shows the accumulation of precipitation with time in the shape of a mass curve of precipitation. The gage must be serviced about once a week when the clock is re-wound and the chart is replaced. For high rainfall, the recording mechanism reverses the direction of record immediately on reaching the upper edge of the recording chart.

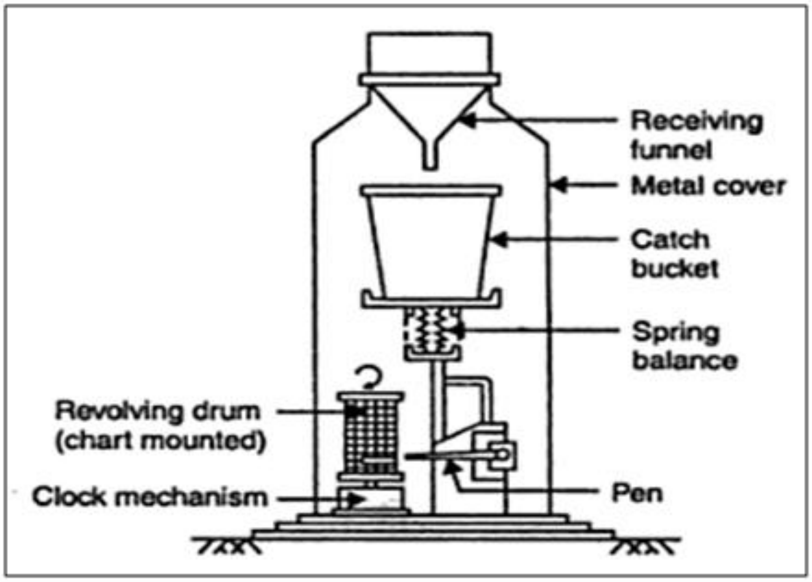


Fig. 5.2.Weighing bucket type rain gauge.(Source: Raghunath,2006)

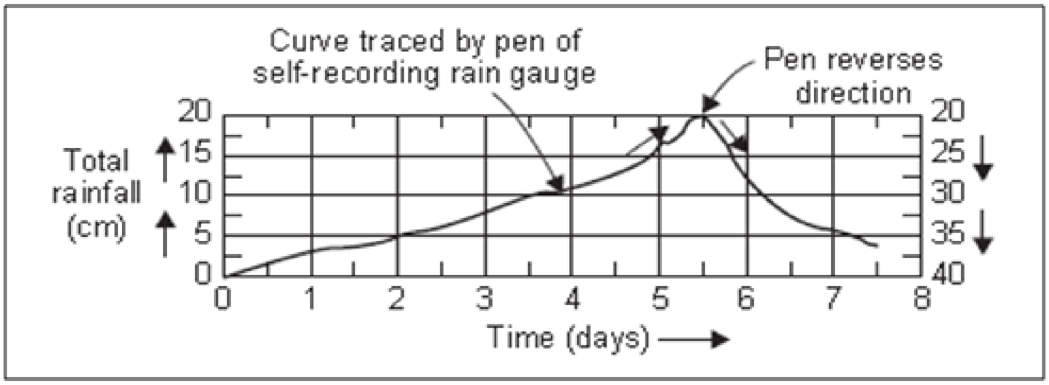


Fig. 5.3.Recorded mass curve of precipitation in weighing bucket type rain gage.(Source: Raghunath, 2006)

**5.2.2.2 Tipping Bucket Type Raingage** - The tipping bucket raingage consists of a 30 cm diasharp edge receiver. At the end of the receiver a funnel is provided. A pair of buckets are pivoted under the funnel in such a way that when one bucket receives 0.25 mm of rainfall it tips, discharging its contents in to a tank bringing the other bucket under the funnel. Tipping of the bucket completes an electric circuit causing the movement of a pen to mark on a clock driven revolving drum which carries a record sheet.

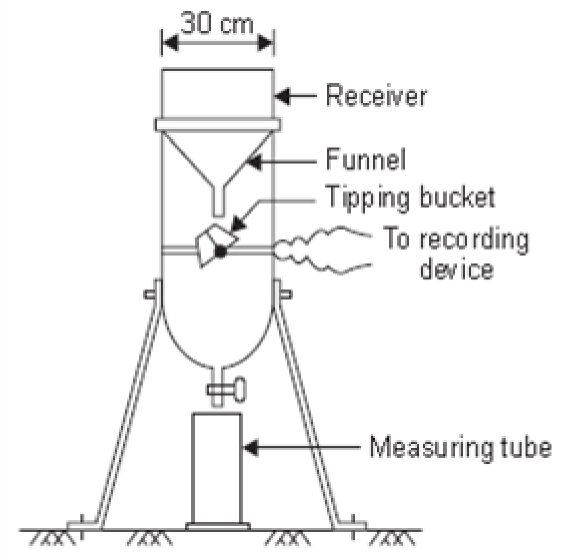


Fig. 5.4. Tipping bucket type raingage.(Source: Raghunath, 2006)

a. Siphon Type Automatic Rainfall Recorder - In the siphon gage, also known as the float type of recording raingage, the rain is fed into a float chamber containing a light, hallow float. The vertical movement of the float, as the level of water rises, is transmitted by a suitable mechanism in to the movement of the pen on a revolving chart. By suitably adjusting the dimensions of the receiving funnel, float and float chamber, any desired scale value on the chart can be obtained. Siphoning arrangement is provided for emptying the float chamber quickly whenever it becomes full, the pen returns to the bottom of the chart.

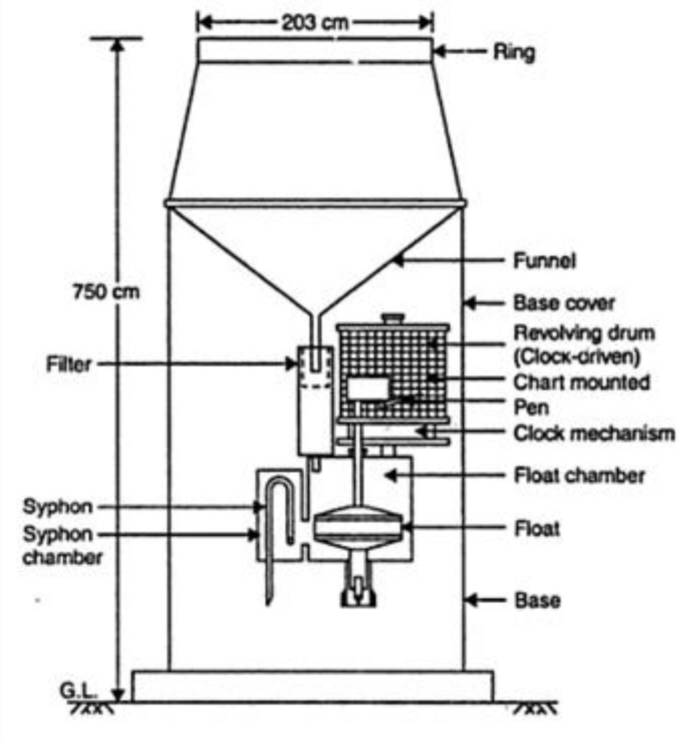


Fig. 5.5.Siphon type automatic rainfall recorder.

(Source: Raghunath, 2006)

### 5.3 Errors in Rainfall Measurements

There are three main sources of errors in rainfall measurements –

a) instrumental defects,

b) improper sitting (location) of the gage, and

c) human errors

Each recording type gage has inherent errors caused by mechanical parts of the instrument. In addition to mechanical errors, some precipitation is also lost in wetting the collecting funnel and measuring cylinder surface if the gage is dry before it begins to collect measurable amount of water (approximately 25 mm per year). Similarly, evaporation from a non-recording gage could cause a small loss of measurable water over the year.

The improper location of the raingage can tend to either over- or under-catch rainfall. The largest errors in all gages are the effect of wind on the entrance of rain or snow into the instrument. Errors due to wind are greater for light rain than for heavy rains.

In order to avoid erroneous conclusions it is important to give the proper interpretation to precipitation data, which often cannot be accepted at face value. For example, a mean annual precipitation value for a station may have little significance if the gage site has been changed significantly during the period for which average is computed. Also, there are several ways of computing average precipitation over an area; each may give a different answer.

## 5.4 Raingage Network

There is no single answer to determining the mean areal rainfall because it is affected by so many factors. However, the denser the gage network, the more accurate is the representation. Gages are not evenly spaced, high variability areas have more gages and relatively uniform rainfall areas have fewer gages. In addition, costs of installation, maintenance of the network, as well as its accessibility to the observer, are also important consideration.

In general, the sampling errors of rainfall tends to increase with increasing mean areal rainfall, and decrease with increasing network density, duration of rainfall, and areal extend. Accordingly, larger average errors are produced by a particular network for storm rainfall than for monthly, seasonal or annual rainfall.

The adequacy of an existing rain-gage network of a watershed is assessed statistically. The optimum number of raingages corresponding to an assigned percentage of error in estimation of mean areal rainfall can be obtained as:

$$N = \left(\frac{CV}{\varepsilon}\right)^2 \qquad (5.1)$$

Where, N is the optimum number of raingages, CV is the coefficient of variation of the rainfall values of the gages, and  is the assigned percentage of error in estimation of mean areal rainfall.

$$CV = \frac{100S}{\overline{P}} \qquad (5.2)$$

In which  is the mean rainfall defined as

$$\overline{P} = \frac{1}{m}\sum_{i=1}^{m} P_i \qquad (5.3)$$

and S is the standard deviation of rainfall computed as

$$S = \frac{1}{m-1}\left[\sum_{i=1}^{m} P_i^2 - \frac{\left(\sum_{i=1}^{m} P_i\right)^2}{m}\right]^{0.5} \qquad (5.4)$$

Where, m is the number of raingages in the watershed recording $P_1$, $P_2$… $P_m$ values of rainfall for fixed time interval. Generally, value of   is taken as 10%.

Example: A catchment has six raingage stations. In a year, the annual rainfalls recorded by the gages are as follows:

| Stations | A | B | C | D | E | F |
|---|---|---|---|---|---|---|
| Rainfall (cm) | 82.6 | 102.9 | 180.3 | 110.3 | 98.8 | 136.7 |

For a 10% error in the estimation of mean rainfall, calculate optimum number of stations in the catchment.

Solution: Number of stations (m) = 6,

Mean precipitation = 118.6 cm

Standard deviation of precipitation (S) = 35.04

Error (ε) = 10%

$$CV = \frac{100\,(35.04)}{118.6} = 29.54$$

$$N = \left(\frac{29.54}{10}\right)^2 = 8.7 \cong 9 \text{ stations}$$

### 5.4.1 WMO Recommended Precipitation Network Density

- One station per 600 to 900 km$^2$ – in flat regions of temperate, Mediterranean and tropical zone.
- One station per 100 to 250 km$^2$ – in mountainous regions of temperate, Mediterranean and tropical zone.
- One station per 25 km$^2$ – in small mountainous land with irregular precipitation.
- One station per 1500 to 10,000 km$^2$ – arid and polar zones.

### 5.4.2 Indian Standard Recommendation

- One station per 520 km$^2$ –in plains.
- One station per 260-390 km$^2$– in regions of average elevation of 1000 m.
- One station per 130 km$^2$ – in predominantly hilly areas with heavy rainfall.

## Lesson 6 Presentation of Rainfall Data

A few commonly used methods of presentation of rainfall data which have beenfound to be useful in interpretation and analysis of such data are given below:

### 6.1 Mass Curve of Rainfall

The mass curve of rainfall is a plot of the accumulated precipitation against time,plotted in chronological order. Records of float type and weighing-bucket typegauges are of this form. A typical mass curve of rainfall at a station during a storm is shown in fig 6.1. Mass curves of rainfall are very useful in extractingthe information on the duration and magnitude of astorm. Also, intensities atvarious time intervals in a storm can be obtained by the slope of the curve.Fornon-recording raingauges, mass curves are prepared from knowledge of theapproximate beginning and end of a storm and by using the mass curves ofadjacent recording gauge stations as a guide

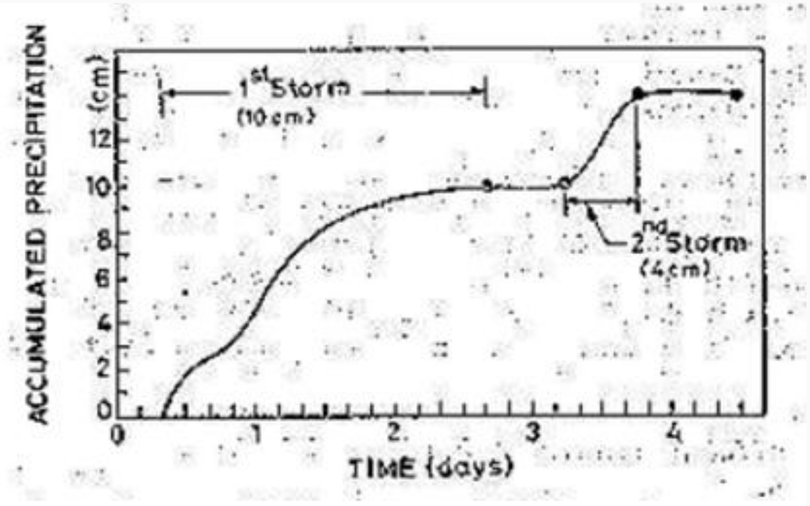


Fig. 6.1.Mass curve of rainfall.(Source: Subramanya, 2006)

### 6.2 Hyetograph

A hyetograph is a plot of the intensity of rainfall against the time in the hyetograph is derived from the mass curve and is usually represented as a bar chart (Fig. 6.2). It is a very convenient way of represents characteristics of a storm and is particularly important in thedevelopment of a design storms to predict extreme floods. The area under a hyetograph represents the total precipitation received in that period. The time in used depends on the purpose; in urban-drainage problems small durations are used while in flood-flow computations in larger catchments the intervals are of about 6 hours.

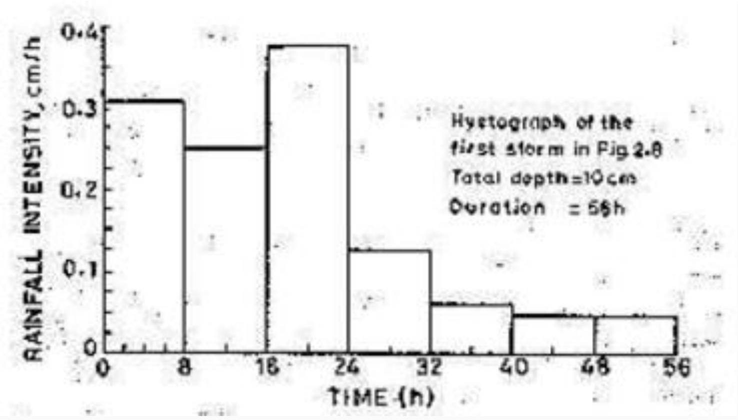


Fig. 6.2.Hyetograph of a storm.(Source: Subramanya, 2006)

### 6.3 Depth-Area-Duration Relationships

The areal distribution characteristics of a storm of given duration is reflected in its depth-area-relationship.

### 6.3.1 Depth-Area Relation

For a rainfall of a given duration, the average depth decreases with the area in an exponential fashion given by

$$\overline{P} = P_0 \exp(-KA^n)$$

Where,  is average depth in cms over an area A km$^2$,is highest amount ofrainfall in cm at the storm centre and K and n are constant for a given region.

On the basis of 42 severe most storms in north India, Dhar and Bhattacharya(1975) have obtained the following values for K and n for storms of different duration.

| Duration | K | n |
|---|---|---|
| 1 day | 0.0008526 | 0.6614 |
| 2 days | 0.0009877 | 0.6306 |
| 3 days | 0.0017454 | 0.5961 |

Since it is very unlikely that the storm centre coincides over a raingauge station, the exact determination of  is not possible. Hence in the analysis of large area storms the highest station rainfall is taken as the average depth over an area of 25 km$^2$.

### 6.3.2 Maximum Depth-Area-Duration Curves

In many hydrological studies involving estimation of severe floods, it is necessary to have information on the maximum amount of rainfall of various duration occurring over various sizes of areas. The development of relationship, between maximum depth-area-duration for a region is known as DAD analysis and forms an important aspect of hydro-meteorological study. A brief description for the DAD analysis is given below.

First the severe most rainstorms that have occurred in the region in the question are considered. Isohyetal maps and mass curves of the storm are compiled. A depth area curve of a given duration of the storm is prepared. Then from a study of the mass curve of rainfall, various durations and the maximum depth of rainfall in these durations are noted. The maximum depth-area curve for a given duration D is prepared by assuming the area distribution of rainfall for smaller duration to be similar to the total storm. The procedure is then repeated for different storms and the envelope curve of maximum depth-area for duration D is obtained. A similar procedure for various values of D results in a family of envelopes curve of maximum depth vs area, with duration as the third parameter (Fig 6.3).These curves are called DAD curves.

Fig 6.3 shows typical DAD curves for a catchment. In this the average depth denotes the depth averaged over the area under consideration. It may be seen that the maximum depth for a given storm decreases with the area; for a given area the maximum depth increases with the duration.

Preparation of DAD curves involves considerable computational effort and requires meteorological and topographical information of the region. Detailed data on severe most storms in the past are needed. DAD curves are essential to develop design storms for use in computing the design flood in the hydrological design of major structures such as dams.

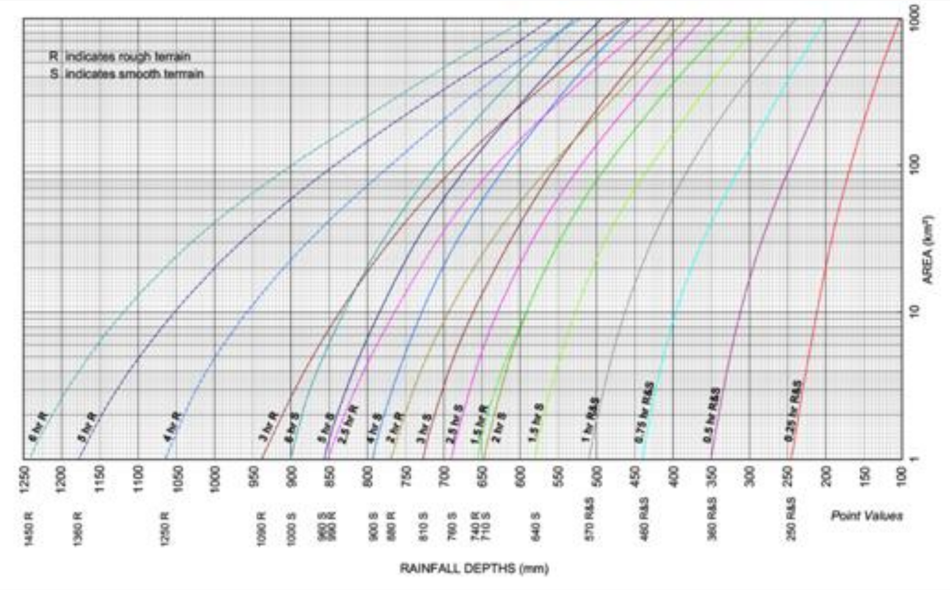
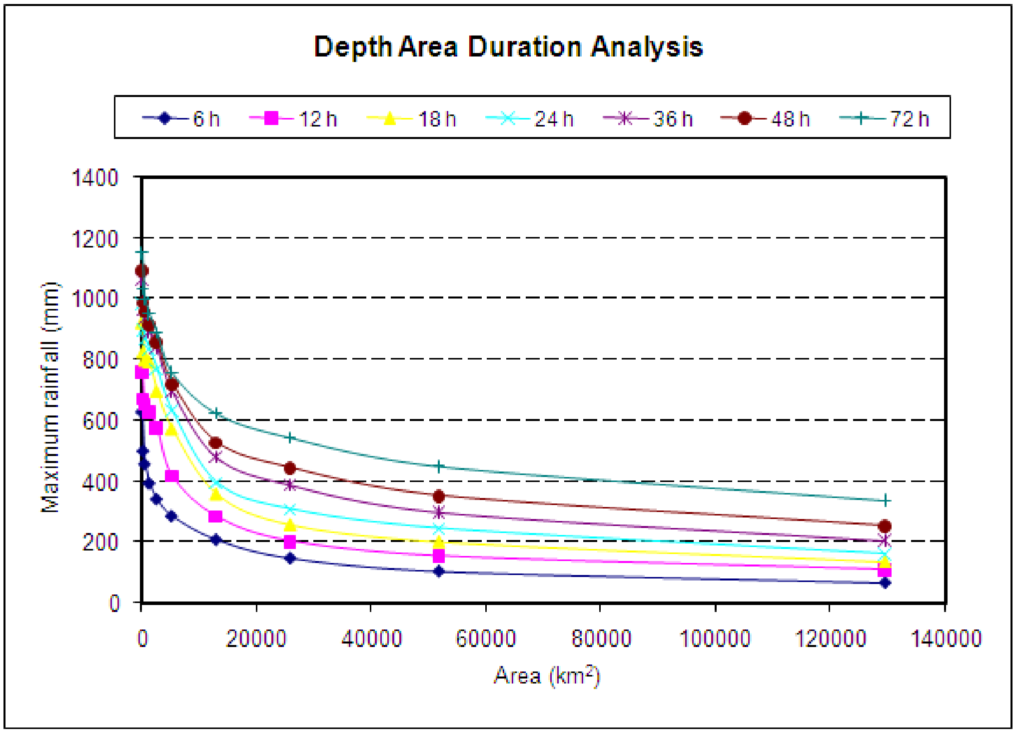


Fig. 6.3.Typical DAD Curves. (Source: Raghunath, 2006)

Solved Example

Construct maximum depth-area-duration curve for the data given below:

| Area (km²) | Maximum rainfall (mm)* | | | | | | |
|---|---|---|---|---|---|---|---|
| | Duration (h) | | | | | | |
| | 6 | 12 | 18 | 24 | 36 | 48 | 72 |
| 25 | 635 | 765 | 930 | 991 | 1070 | 1103 | 1156 |
| 258 | 506 | 676 | 834 | 902 | 971 | 996 | 1039 |
| 517 | 463 | 658 | 806 | 877 | 940 | 966 | 1004 |
| 1294 | 399 | 633 | 802 | 839 | 897 | 922 | 955 |
| 2589 | 348 | 582 | 704 | 775 | 844 | 864 | 894 |
| 5179 | 292 | 423 | 580 | 638 | 701 | 729 | 762 |
| 12945 | 214 | 290 | 366 | 402 | 483 | 534 | 628 |
| 25895 | 153 | 209 | 265 | 315 | 392 | 450 | 549 |
| 51795 | 110 | 160 | 209 | 252 | 303 | 359 | 455 |
| 129495 | 72 | 115 | 143 | 168 | 209 | 259 | 343 |
| 258995 | 51 | 72 | 97 | 117 | 160 | 176 | 234 |


Depth Area Duration Analysis
6 h
12 h
18 h
24 h
36 h
48 h
72 h
Maximum rainfall (mm)
1400
1200
1000
800
600
400
200
0
0
20000
40000
60000
80000
100000
120000
140000
Area (km²)

## Lesson 7 Estimation of Missing Rainfall Data

### 7.1 Estimating Missing Data

The point observation from a precipitation gage may have a short break in the record because of instrument failure or absence of the observer. Thus, it is often necessary to estimate the missing record using data from the neighboring station. The following methods are most commonly used for estimating the missing records.

Simple Arithmetic Method

1. Normal Ratio Method
2. Modified normal ratio method
3. Inverse distance method
4. Linear programming method

For m stations, 1, 2, 3, …,m, the annual precipitation values are P1, P2, P3, …, Pm, respectively. At station x (not included in the above m stations), the missing annual precipitation (Px) should be found out. The normal annual precipitation N1, N2, N3, …,Ni at each of the above (m+1) stations including the station x is known.

**7.1.1 Normal Precipitation -** It is the average value of precipitation at a particular date, month or year over a specified 30 year period. Thus, the term normal annual precipitation at station A means the average annual precipitation at A based on a specified 30 year of record.

**7.1.2 Simple Arithmetic Average -** The missing precipitation Px can be determined using simple arithmetic average, if the normal annual precipitation at various stations are within 10% of the normal precipitation at station, x, as follows:

$$Px=\frac{1}{m}\left[P1+P2+\cdots+Pm\right] \qquad (7.1)$$

**7.1.3 Normal Ratio Method -** If the normal precipitations vary considerably then Px is estimated by weighting the precipitation at various stations by the ratios of normal annual precipitation. The normal ration method gives Px as:

$$Px \quad =\frac{Nx}{m}\left[\frac{P1}{N1}+\frac{P2}{N2}+\cdots+\frac{Pm}{Nm}\right] \qquad (7.2)$$

This method is based selecting m (m is usually 3) stations that are near and approximately evenly spaced around the station with the missing record.

Example 7.1 The normal annual rainfall at stations A, B, C and D in a basin are 80.97, 67.59, 76.28, and 92.01 cm, respectively. In the year 1975, the station D was inoperative and the stations A, B, and C recorded annual rainfall of 91.11, 72.23, and 79.89 cm, respectively. Estimate the rainfall at station D in that year.

Solution: As the normal rainfall values vary by more than 10%, the ration method is adopted.

$$Px = \frac{Nx}{m}\left[\frac{P1}{N1} + \frac{P2}{N2} + \cdots + \frac{Pm}{Nm}\right]$$

$$Pd = \frac{92.01}{3}\left[\frac{91.11}{80.97} + \frac{72.23}{67.59} + \frac{79.89}{76.28}\right] = 99.41\,cm$$

### 7.1.4 Modified Normal Ratio Method

Normal ratio method is modified to incorporate the effect of distance in the estimation of missing rainfall.

$$r_x = \frac{\sum_{i=1}^{n} D_i^{\,1/b}\left(\frac{\overline{r_x}}{\overline{r_i}}\right) r_i}{\sum_{i=1}^{n} D_i^{\,1/b}} \qquad (7.3)$$

Where  is normal rainfall,  is the distance between the index station i and the gauge station with missing data or ungaged station, n is the number of index stations and b is the constant by which the distance is weighted (normally 1.5-2.0) commonly used $D^{0.5}$

### 7.1.5Inverse Distance Method

The inverse distance method has been advocated to be the most accurate method as compare to other two methods discussed above.

Amount of rainfall to be estimated at a location is a function of;

1. rainfall measured at the surrounding index stations
2. distance to each index station from the ungauged location

Rainfall $r_x$ at station x is given by;

$$r_x = \frac{\sum_{i=1}^{n}\left(\frac{r_i}{D_i^{\,b}}\right)}{\sum_{i=1}^{n}\left(\frac{1}{D_i^{\,b}}\right)} \qquad (7.4)$$

b = 2 is commonly used.

As in inverse distance method the weighting is strictly based on distance, hence this method is not satisfactory for hilly regions.

Example 7.2 Data for the base station and 5 surrounding stations are tabulated below. Find missing data at 'A' using (i) modified normal ratio method and (ii) inverse distance method.

| Station | Distance (D) from the base station (km) | Rainfall (cm|) | Normal Rainfall (cm) |
|---|---|---|---|
| A (base station) | - | ? | 102 |
| B | 1.5 | 2.5 | 114 |
| C | 1.21 | 3.4 | 122 |
| D | 0.85 | 1.5 | 95 |
| E | 1.3 | 2.2 | 106 |
| F | 2.11 | 1.8 | 104 |

Solution: (i) Using modified normal ratio method

Weight calculation for different stations other than the base station is shown in following table, using b = 2

| Station | | Weight (a) |
|---|---|---|
| B | 1.22 | |
| C | 1.1 | |
| D | 0.92 | |
| E | 1.14 | |
| F | 1.45 | |
| Sum | 5.83 | 1.0 |

Hence,

$$r_A = \frac{\overline{r_A}}{\overline{r_B}} r_B a_B + \frac{\overline{r_A}}{\overline{r_C}} r_C a_C + \frac{\overline{r_A}}{\overline{r_D}} r_D a_D + \frac{\overline{r_A}}{\overline{r_E}} r_E a_E + \frac{\overline{r_A}}{\overline{r_F}} r_F a_F$$

$$= \frac{102}{114} 2.5(0.21) + \frac{102}{122} 3.4(0.19) + \frac{102}{95} 1.5(0.15) + \frac{102}{106} 2.2(0.20) + \frac{102}{104} 1.8(0.25)$$

= 2.12 cm

(ii) Using Inverse Distance Method

| Station | | Weight (a) |
|---|---|---|
| B | 0.44 | |
| C | 0.68 | |
| D | 1.38 | |
| E | 0.59 | |
| F | 0.22 | |
| Sum | 3.31 | 1.0 |

Hence,

$$r_A = r_B a_B + r_C a_C + r_D a_D + r_E a_E + r_F a_F$$

$$= 2.5(0.13) + 3.4(0.21) + 1.5(0.41) + 2.2(0.18) + 1.8(0.07)$$

$$= 2.18 \text{ cm}$$

### 7.1.6Linear Programming Method

Linear programming (LP) method selects a base station and several surrounding index stations and determines optimal weighting factor by minimizing the deviation between observed and computed rainfall at a base station for a number of rainfall events.

Thus it determines optimal weighting factors for the base station and associated index stations.

This method can be formulated as,Objective is to minimize sum of deviation for k events

i.e., Minimize $\sum_{j=1}^{k}(U_j + V_j)$

Subjected to

$$\sum_{i=1}^{n} a_i r_{ij} - U_j + V_j = r_{bj} \text{( j=1,2,.......k )}$$

$$\sum_{i=1}^{n} a_i = 1.0 \qquad \text{(sum of weights is 1)}$$

$$a_i \geq 0, \qquad U_j \geq 0, \qquad V_j \geq 0$$

(Non-negativity constraints)

Where, i = index for "index station"

j = index for rainfall events

= observed rainfall at base station 'b' for event 'j'

$\sum_{i=1}^{n} a_i r_{ij}$ = computed rainfall at base station for event 'j'

For any event, computed rain – observed rain = deviation

Deviation could be either positive or negative value (unrestricted in sign), such variables are replaced by the difference of two non-negative variables (LP requirement) i.e., U.

Solved Example

Assume that rainfall is not known at the station D. The normal precipitations of the three neighbouring gauging stations are as follows:

| Station | Station co-ordinate | Normal Annual Precipitation (cm) | Precipitation (cm) | |
|---|---|---|---|---|
| A | (1,2.5) | 28 | 25 | |
| B | (4,1) | 15 | 10 | |
| C | (3,5) | 30 | 25 | |
| D | (3,3) | 25 | ? | |

Compute the rainfall at this point using

a. Simple Arithmetic Method

b. Normal

Ratio Method

c. Modified Normal Ratio Method

d. Inverse Distance Method

Answer

a. Simple Arithmetic Method

$$P_D = \frac{1}{3}[P_A + P_B + P_C]$$

$$P_D = \frac{1}{3}[25 + 10 + 25]$$

$$P_D = 20 \text{ cm}$$

b. Normal Ratio Method

$$Px = \frac{Nx}{m}\left[\frac{P1}{N1} + \frac{P2}{N2} + \cdots + \frac{Pm}{Nm}\right]$$

$$P_D = \frac{N_D}{3}\left[\frac{P_A}{N_A} + \frac{P_B}{N_B} + \frac{P_C}{N_C}\right]$$

$$P_D = \frac{25}{3}\left[\frac{25}{28} + \frac{10}{15} + \frac{25}{30}\right]$$

$$P_D = 19.94 \text{ cm}$$

c. Modified Normal Ratio Method

$$r_x = \frac{\sum_{i=1}^{n} D_i^{\,1/b}\left(\frac{N_D}{N_i}\right)P_i}{\sum_{i=1}^{n} D_i^{\,1/b}}$$

$$P_D = \frac{D_A^{\,1/b}\left(\frac{N_D}{N_A}\right)P_A + D_B^{\,1/b}\left(\frac{N_D}{N_B}\right)P_B + D_C^{\,1/b}\left(\frac{N_D}{N_C}\right)P_C}{D_A^{\,1/b} + D_B^{\,1/b} + D_C^{\,1/b}}$$

$$P_D = \frac{2.06^{1/2}\left(\frac{25}{28}\right)25 + 2.24^{1/2}\left(\frac{25}{15}\right)10 + 2^{1/2}\left(\frac{25}{30}\right)25}{2.06^{1/2} + 2.24^{1/b} + 2^{1/2}}$$

$$P_D = \frac{2.06^{1/2}\left(\frac{25}{28}\right)25 + 2.24^{1/2}\left(\frac{25}{15}\right)10 + 2^{1/2}\left(\frac{25}{30}\right)25}{2.06^{1/2} + 2.24^{1/b} + 2^{1/2}}$$

$$P_D = \frac{2.06^{1/2}\left(\frac{25}{28}\right)25 + 2.24^{1/2}\left(\frac{25}{15}\right)10 + 2^{1/2}\left(\frac{25}{30}\right)25}{2.06^{1/2} + 2.24^{1/b} + 2^{1/2}}$$

$$P_D = 19.88 \ \text{cm}$$

d. Inverse Distance Method

$$P_D = \frac{\left(\frac{P_A}{D_A{}^{b}}\right) + \left(\frac{P_B}{D_B{}^{b}}\right) + \left(\frac{P_C}{D_C{}^{b}}\right)}{\left(\frac{1}{D_A{}^{b}}\right) + \left(\frac{1}{D_B{}^{b}}\right) + \left(\frac{1}{D_C{}^{b}}\right)}$$

$$P_D = \frac{\left(\frac{25}{2.06^2}\right) + \left(\frac{10}{2.24^2}\right) + \left(\frac{25}{2^2}\right)}{\left(\frac{1}{2.06^2}\right) + \left(\frac{1}{2.24^2}\right) + \left(\frac{1}{2^2}\right)}$$

$$P_D = \frac{\left(\frac{25}{2.06^2}\right) + \left(\frac{10}{2.24^2}\right) + \left(\frac{25}{2^2}\right)}{\left(\frac{1}{2.06^2}\right) + \left(\frac{1}{2.24^2}\right) + \left(\frac{1}{2^2}\right)}$$

$$P_D = 20.63 \text{ cm}$$

## Lesson 8 Consistency of Rainfall Record

Precipitation puts water in the watershed. Precipitation measurements help determine water availability for evaporation and streamflow, and the risk of forest fires, landslides, and soil erosion. Precipitation is liquid (rain) or frozen (sleet, hail, graupel, snow) water or a combination of both falling from the sky. Measurement of the accumulation and disappearance of frozen precipitation on the ground is discussed in the following subsection. Precipitation is measured as depth of water that would accumulate on a horizontal surface. Typical recorded values for precipitation are the daily, monthly, and annual totals (depth of water in millimeters), as well as storm total, maximum intensity (millimeters per minute or per hour), and duration (hours). Precipitation is measured at a point using a manual or automatic recording gauge and over an area using meteorological radar (e.g., Doppler) or satellite images. Measurement of rainfall with a gauge is less prone to error than the measurement of solid precipitation because it is less susceptible to the influence of wind on “catch” by gauges. Some gauges are suitable only for measuring rainfall, and others are used only for solid precipitation. Very few can measure both forms reliably. Precipitation is usually not uniform in spatial distribution, intensity, or duration within a storm. Wind flow interacting with watershed topography also affects the distribution of precipitation. The standard technique for measuring rainfall is a plastic or metal cylinder with a sharp edge and funnel-like cover to minimize evaporation.

Rainfall is measured to the nearest 0.2 mm; an accumulation of less than this is called a “trace.” If these gauges are not measured daily, then a small amount of mineral oil or kerosene is added after each measurement to cover the surface of the water and reduce evaporative loss.

The depth of fluid is measured with a ruler. Precipitation intensity or amount can be measured automatically using weighing gauges or tipping buckets. The latter method is usually restricted to rainfall measurement. The tipping bucket gauge has a pair of buckets that pivot under a funnel such that when one bucket fills with about 0.25 mm of rain, it tips, discharging its contents and bringing the other bucket under the funnel. Tipping activates a switch that sends a pulse to the recording device. The weighing gauges can have a clock-driven chart to record weight or send an electronic signal to a data logger from a pressure transducer. Weighing gauges used to measure snow, contain antifreeze to melt the snow.

Many studies require long term rainfall data, therefore, a test must be conducted to check homogeneity or self consistency of the rainfall record. This is necessary because over a period of time, it may happen that there be a some obstructions (trees, buildings) may have emerged after the installation of gage or its location might have changed or observational procedure might have changed. The inconsistency of rainfall record can be checked by graphical or statistical methods including double mass curve, the von Neumann ratio test, cumulative deviation, run test. Double mass curve method described below is one of the most common and widely accepted methods for checking the consistency of rainfall record.

## 8.1 Double-mass Curve

This method is based on the assumption that the mean accumulated precipitation for a large group of stations is not significantly affected by a change or changes in individual stations. If we plot the mean accumulated precipitation for several stations against the accumulated precipitation of the record for the station that needs to be adjusted, any change in slope will indicate a "break" in the station record. This is shown by point B in the Fig. for the year 1974.

If the slope of the line AB (year 1970-74 in the example) is a and of the line AC (1974-85 in example fig.) is b, the adjustment of the inconsistent data is made by the ration of the slope of two line segments. Two ways of adjustments are possible.

1. The dataadjusted to reflect the conditions that existed prior to the indicated break. This is done by multiplying each recent precipitation value after break point B of station X (being tested) by the ratio a/b.
2. The data are adjusted toreflect recent conditions following the break. This is achieved by multiplying each value of the precipitation before the break by the ratio b/a. Adjustment of this type is generally made.

This is essentially a simple graphical method but statistical concepts and tests can be also utilized. Let us assume that we wish to check whether the data y1, y2 ,..., yN (N= sample size) are consistent data or not. For this purpose we will use another data set N x1, x2... xN, which is known to be reliable. The latter data set could be data measured at another gage or more generally the average of the data records available at several sites located in the same region as the suspected gage y. We will define the cumulative partial sums as

$$S_t(Y)=S_{t-1}(Y)+Y_t \qquad , t = 1,2,\ldots,N$$

$$S_t(X) = S_{t-1}(X) + X_t \quad , t = 1,2,\ldots,N$$

in which . Thus we have two sequences of partial sums, namely: and . The double mass plot is constructed as shown in the sketch below.

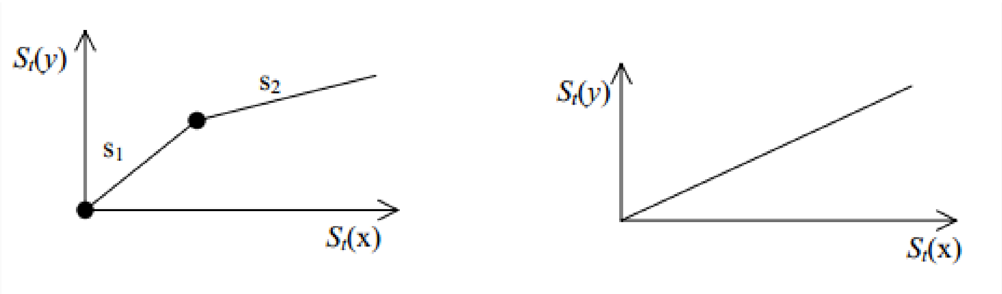


Fig. 8.1(a) Fig. 8.1(b)

Fig.8.1. (Source:Singh,1994)

Figure 1(a) reflects the inconsistency in the data while Fig.1(b) reflects that series yt has a pattern consistent with that of series xt. Referring to Fig.1(a) the point where the slope changes from s1 to s2 marks the time where the inconsistency occurs. Therefore part of the record of y must be adjusted. Which part of the record must be adjusted (before the break point or after) depends on the particular case. For example, if the inconsistency in y has occurred because the gage was moved from the original location to another location (the current location), then it is logical to adjust the record before the break point.

## 8.2 Limitations and Cautions

It is possible that an apparent change in slope is noticed because of a natural variation in the data which is not associated with changes in gage location, gage environment or observation procedure. In the case of doubt, a test of hypothesis should be performed by the Fisher distribution on two data sets (before and after the break) to check whether the data are homogeneous and the break is purely by chance.

If the fewer than 10 stations are grouped together to check the consistency of a station, the record of each station should be tested by double mass analysis for consistency by plotting it against the group of all other stations, and those records that are inconsistent should be eliminated from the group.

This method should seldom be used in mountainous areas. Furthermore, it is also not suitable for adjusting daily or storm precipitation.

Example

Test the consistency of the 22 years of data of the annual precipitation measuredat stationA. Rainfall data for station A as well as the averageannual rainfall measured for a group of eight neighboring stations located in a meteorologicallyhomogeneous region are given below as follows:

| Sl. No. | Year | Annual Rainfall of Station A | Average Annual Rainfall (AAR) of 8 Station Group |
|---|---|---|---|
| | | (mm) | (mm) |
| 1 | 1946 | 177 | 143 |
| 2 | 1947 | 144 | 132 |
| 3 | 1948 | 178 | 146 |
| 4 | 1949 | 162 | 147 |
| 5 | 1950 | 194 | 161 |

| | | | |
|---|---|---|---|
| 6 | 1951 | 168 | 155 |
| 7 | 1952 | 196 | 152 |
| 8 | 1953 | 144 | 117 |
| 9 | 1954 | 150 | 130 |
| 10 | 1955 | 130 | 190 |
| 11 | 1956 | 141 | 170 |
| 12 | 1957 | 160 | 190 |
| 13 | 1958 | 155 | 166 |
| 14 | 1959 | 140 | 150 |
| 15 | 1960 | 120 | 125 |
| 16 | 1961 | 148 | 140 |
| 17 | 1962 | 142 | 163 |
| 18 | 1963 | 140 | 145 |
| 19 | 1964 | 130 | 143 |
| 20 | 1965 | 137 | 135 |
| 21 | 1966 | 130 | 150 |
| 22 | 1967 | 163 | 165 |

a) In what year is a change in regime indicated?

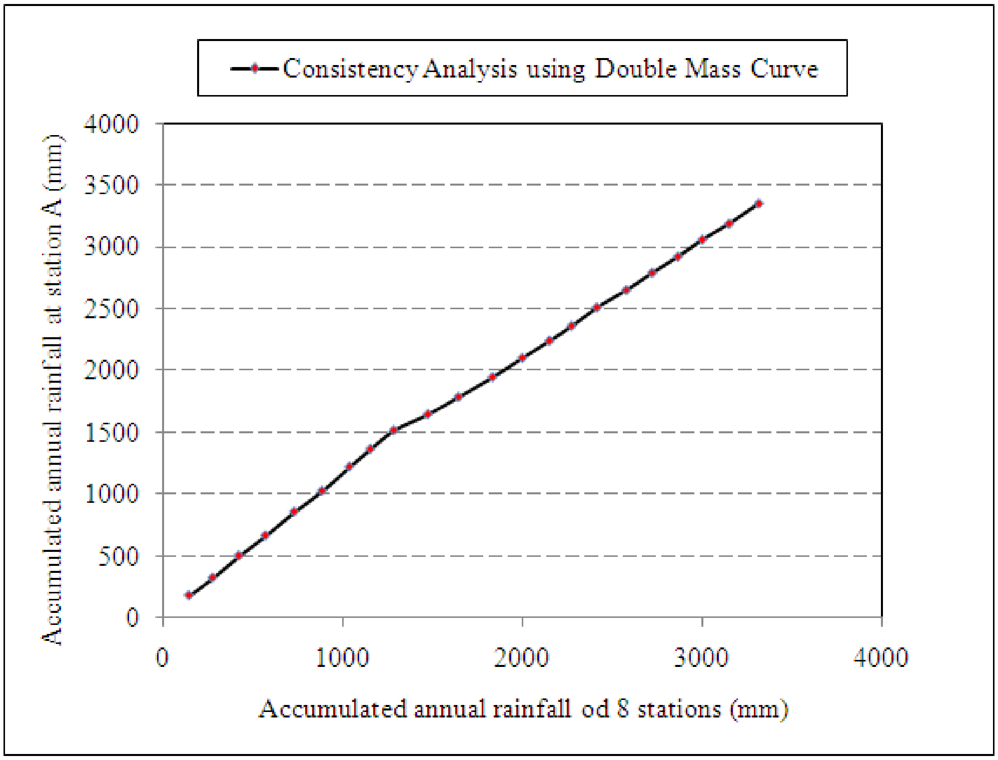
b) Adjust the record data at a station A and determine the mean annual precipitation.

| | | Col 3 | Col 4 | Col 5 | Col 6 | | |
|---|---|---|---|---|---|---|---|
| **Sl. No.** | **Year** | **Annual Rainfall of Station A** | **Average Annual Rainfall (AAR) of 8 Station groups** | **Cumulative Station A rainfall** | **Cumulative of 8 Station A.A.R.** | **Correction factor** | **Adjusted rainfall at station A** |
| | | **(mm)** | **(mm)** | **(mm)** | **(mm)** | | |
| 1 | 1946 | 177 | 143 | 177 | 143 | 1.18 | 140.8471 |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| 2 | 1947 | 144 | 132 | 321 | 275 | | 114.5875 |
| 3 | 1948 | 178 | 146 | 499 | 421 | | 141.6428 |
| 4 | 1949 | 162 | 147 | 661 | 568 | | 128.9109 |
| 5 | 1950 | 194 | 161 | 855 | 729 | | 154.3748 |
| 6 | 1951 | 168 | 155 | 1023 | 884 | | 133.6854 |
| 7 | 1952 | 196 | 152 | 1219 | 1036 | | 155.9663 |
| 8 | 1953 | 144 | 117 | 1363 | 1153 | | 114.5875 |
| 9 | 1954 | 150 | 130 | 1513 | 1283 | | 119.3619 |
| 10 | 1955 | 130 | 190 | 1643 | 1473 | 0.94 | 196 |
| 11 | 1956 | 141 | 170 | 1784 | 1643 | | 141 |
| 12 | 1957 | 160 | 190 | 1944 | 1833 | | 158 |
| 13 | 1958 | 155 | 166 | 2099 | 1999 | | 145 |
| 14 | 1959 | 140 | 150 | 2239 | 2149 | | 132 |
| 15 | 1960 | 120 | 125 | 2359 | 2274 | | 95 |
| 16 | 1961 | 148 | 140 | 2507 | 2414 | | 148 |
| 17 | 1962 | 142 | 163 | 2649 | 2577 | | 142 |
| 18 | 1963 | 140 | 145 | 2789 | 2722 | | 140 |
| 19 | 1964 | 130 | 143 | 2919 | 2865 | | 130 |
| 20 | 1965 | 137 | 135 | 3056 | 3000 | | 137 |
| 21 | 1966 | 130 | 150 | 3186 | 3150 | | 130 |
| 22 | 1967 | 163 | 165 | 3349 | 3315 | | 163 |
| | | | | | | | **139.134** |

Correction factor: (0.935/1.175)= 0.7957= 0.796 or 0.8

| Mean Annual Precipitation | 139.134 |
|---|---|



## Lesson 9 Estimation of Mean Areal Rainfall

A single point precipitation measurement is quite often not representative of the volume of precipitation falling over a given catchment area. The representative precipitation over a defined area is required in many engineering applications, whereas the gaged observation pertains to the point precipitation. A dense network of point measurements and/or radar estimates can provide a better representation of the true volume over a given area. A network of precipitation measurement points can be converted to areal estimates using any of the following techniques:

1. Arithmetic or Station Average Method
2. Thiessen Polygon Method
3. Isohyetal Method.

### 9.1 Arithmetic Average Method

This method consists of computing the arithmetic average of the values of the precipitation for all stations within the area. Since this method assigns equal weight to all stations irrespective of their relative location and other factors, it should be adopted in area where rainfall is uniformly distributed.

$$\overline{P} = \frac{1}{n}\sum_{i=1}^{n} P_i \qquad (9.1)$$

Where average precipitation is over an area, P is the precipitations at individual station i, and n is the number of stations.

### 9.2 Thiessen Polygon Method

This is a graphical technique which calculates station weights based on the relative areas of each measurement station in the Thiessen polygon network.Rainfall varies in intensity and duration from one place to other; hence rainfall recorded by each station should be weighed according to the area (polygons) it is assumed to influence. The individual weights are multiplied by the station observation and the values are summed to obtain the areal average precipitation.This method is useful for areas, which are more or less plain and are of intermediate size (500 to 5000 km$^2$). This method is also used when there are a few raingage stations compared to size. The polygons are formed as follows:

1. The stations are plotted on a map of the area drawn to a scale.
2. The adjoining stations are connected by the dashed lines.
3. Perpendicular bisectors are constructed on each of these dashed lines.

4. These bisectors form polygons around each station (effective area for the station within the polygon). For stations close to the boundary, the boundary lines form the closing limit of the polygons.

Area of each polygon ($A_i$)is determined and the average precipitation is calculated using the following equation:

$$\overline{P} = \frac{\sum_{i-1}^{N} P_i A_i}{A}$$

(9.2) Where A is the total area of the watershed.

### 9.3 Isohyetal Method

This is a graphical technique which involves drawing estimated lines of equal rainfall over an area based on point measurements. Then multiply the area between each contour by the average precipitation in the area to get the rainfall volume in the area. Sum these volumes to get the total rainfall volume, and then divide the total rainfall volume by the area of the watershed to get the average areal precipitation in the watershed.

Let’s take it step by step:

Step1: Determine what contours of equal precipitation (called isohytes) you will use.

This varies from situation to situation, but you want to have as many contours as necessary to get an accurate model, but not so many that your construction becomes cluttered.

Step2: Draw a line between gauges that will be separated by isohyets.

Step3: Plot points on those lines that correspond to the isohyets determined in Step 2.

Step4: Now sketch the isohyets.

Step5: Redraw the construction onto graph paper with the isohyetal lines. Then count the boxes between each of the isohyetal lines.

Step6: Find the actual watershed area between each isohyet. These areas will be lettered starting with A at the top and moving alphabetically toward the bottom of the construction.

Step7: Multiply the areas found in Step 6 by the average precipitation in the area.

Step8: Divide the sum of the values found in Step 7 by the total area of the watershed to get the average rainfall in the area.

### 9.4 Raingage Network

There is no single answer to determining the mean areal rainfall because it is affected by so many factors. However, the denser the gage network, the more accurate is the representation. Gages are not evenly spaced, high variability areas have more gages and relatively uniform rainfall areas have fewer gages. In addition, costs of installation, maintenance of the network, as well as its accessibility to the observer, are also important consideration.

In general, the sampling errors of rainfall tends to increase with increasing mean areal rainfall, and decrease with increasing network density, duration of rainfall, and areal extend. Accordingly, larger average errors are produced by a particular network for storm rainfall than for monthly, seasonal or annual rainfall.

The adequacy of an existing rain-gage network of a watershed is assessed statistically. The optimum number of raingages corresponding to an assigned percentage of error in estimation of mean areal rainfall can be obtained as:

$$N = \left(\frac{CV}{\varepsilon}\right)^2 \qquad (9.3)$$

Where, N is the optimum number of raingages, CV is the coefficient of variation of the rainfall values of the gages, and  is the assigned percentage of error in estimation of mean areal rainfall.

$$CV = \frac{100S}{\overline{P}} \qquad (9.4)$$

In which  is the mean rainfall defined as

$$\overline{P} = \frac{1}{m}\sum_{i=1}^{m} P_i \quad (9.5)$$

and S is the standard deviation of rainfall computed as

$$S = \frac{1}{m-1}\left[\sum_{i=1}^{m} P_i^2 - \frac{\left(\sum_{i=1}^{m} P_i\right)^2}{m}\right]^{0.5} \qquad (9.6)$$

Where, m is the number of raingages in the watershed recording $P_1, P_2, \ldots, P_m$ values of rainfall for fixed time interval. Generally, value of  is taken as 10%.

Example: A catchment has six raingage stations. In a year, the annual rainfalls recorded by the gages are as follows:

| Stations : | A | B | C | D | E | F |
|---|---|---|---|---|---|---|
| Rainfall (cm): | 82.6 | 102.9 | 180.3 | 110.3 | 98.8 | 136.7 |

For a 10% error in the estimation of mean rainfall, calculate optimum number of stations in the catchment.

Solution:

Number of stations (m) = 6,

Mean precipitation $(\bar{P})$ =118.6 cm

Standard deviation of precipitation (S) = 35.04

Error (ε) =10%

$$CV = \frac{100(35.04)}{118.6} = 29.54$$

$$N = \left(\frac{29.54}{10}\right)^2 = 8.7 = 9 \text{ stations}$$

### 9.4.1 WMO Recommended Precipitation Network Density

- One station per 600 to 900 km$^2$ - in flat regions of temperate, Mediterranean and tropical zone.
- One station per 100 to 250 km$^2$ – in mountainous regions of temperate, Mediterranean and tropical zone.
- One station per 25 km$^2$ – in small mountainous land with irregular precipitation.
- One station per 1500 to 10,000 km$^2$ – arid and polar zones.

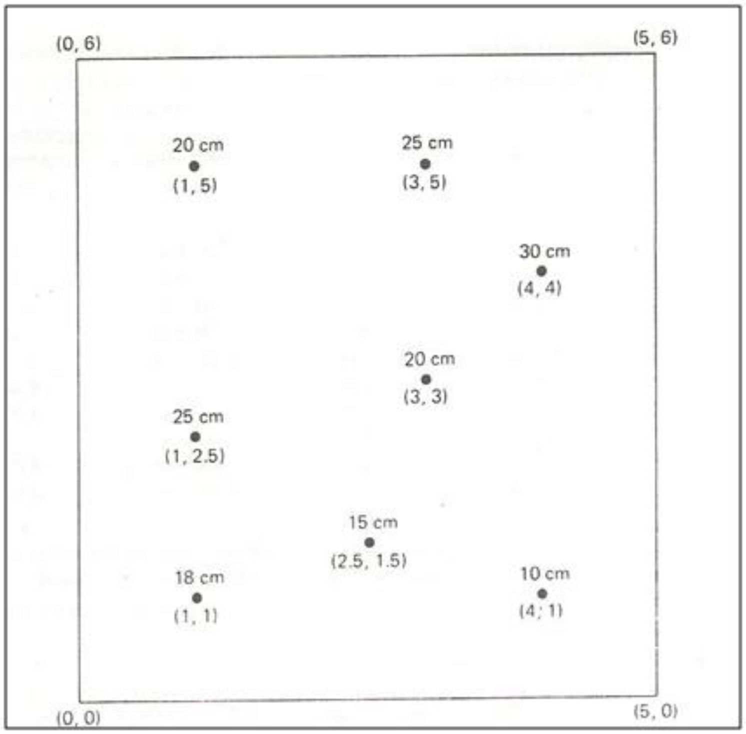
### 9.4.2 Indian Standard Recommendation

- One station per 520 km$^2$ - in plains.
- One station per 260-390 km$^2$– in regions of average elevation of 1000 m.
- One station per 130 km$^2$– in predominantly hilly areas with heavy rainfall.

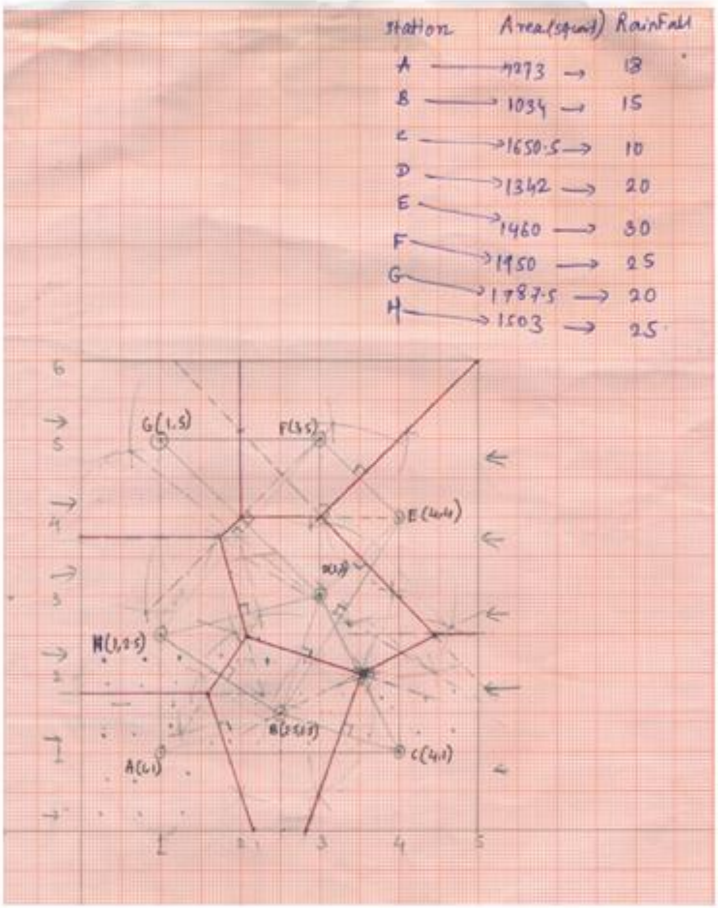
Solved Problems

Example 1

Estimate the mean areal rainfall for the area shown in Fig. 3.1, using the Thiessen polygon method.



Answer



| station | Area(sq.unit) | RainFall |
|---|---|---|
| A | 1273 | 18 |
| B | 1034 | 15 |
| C | 1650.5 | 10 |
| D | 1342 | 20 |
| E | 1460 | 30 |
| F | 1950 | 25 |
| G | 1787.5 | 20 |
| H | 1503 | 25 |

$$\bar{P} = \frac{P_A A_A + P_B A_B + P_C A_C + P_D A_D + P_E A_E + P_F A_F + P_G A_G + P_H A_H}{A_A + A_B + A_C + A_D + A_E + A_F + A_G + A_H}$$

$\bar{P}$ = 20.637 cm$\bar{P}$

$$= \frac{18 \times 1273 + 15 \times 1034 + 10 \times 1650.5 + 20 \times 1342 + 30 \times 1460 + 25 \times 1950 + 20 \times 1787.5 + 25 \times 1503}{1273 + 1034 + 1650.5 + 1342 + 1460 + 1950 + 1787.5 + 1503}$$

Example 2

A catchment area has seven raingauge stations. In a year the annual rainfall recorded by the gauges are as follows:

| Station | P | Q | R | S | T | U | V |
|---|---|---|---|---|---|---|---|
| Rainfall (cm) | 130.0 | 142.1 | 118.2 | 108.5 | 165.2 | 102.1 | 146.9 |

For a 5% error in the estimation of mean rainfall, calculate the minimum number of additional stations required to be established in the catchment.

Answer

|  | P | P-$P_{avg}$ | (P-$P_{avg}$)$^2$ |
|---|---|---|---|
| P | 130 | -0.43 | 0.18 |
| Q | 142.1 | 11.67 | 136.18 |
| R | 118.2 | -12.23 | 149.57 |
| S | 108.5 | -21.93 | 480.92 |
| T | 165.2 | 34.77 | 1208.95 |
| U | 102.1 | -28.33 | 802.58 |
| V | 146.9 | 16.47 | 271.26 |

| | | | |
|---|---|---|---|
| MEAN | 130.43 | | 3049.67 |
| Standard Deviation | 20.87 | | 22.54 |
| | | | 130.42 |
| Coefficient of Variation | | | |
| | | | 0.17 |
| Coefficient of Variation (%) | | | 17.28 |
| Percentage error (%) | | | 5 |
| | | | |
| Optimum number of rain gauge stations | 12 | | 11.95 |
| | | | |
| Answer | 12-7=5 | | |

## Lesson 10 Frequency Analysis of Point Rainfall

Storms of high intensity and varying durations occur from time to time. However, the probability of these heavy rainfalls varies with locality. The first step in designing engineering projects dealing with flood control, gully control etc. is to determine the probability of occurrence of a particular extreme rainfall. This information is determined by the frequency analysis of point rainfall data.

Frequency analysis deals with the chance of occurrence of an event over a specified period of time. Suppose, P is the probability of occurrence of an event (rainfall) whose magnitude is equal to or in excess of a specified magnitude X. The recurrence interval (return period) is related to P as follows:

### 10.1 Plotting Position Formulae

There are two methods for performing frequency analysis

- Empirical method
- Frequency factor method

The excedence probability of the event is obtained by the use of empirical formula, known as plotting position.

Several plotting position formulas were developed and some of them are given below:

Table 10.1. Plotting position formulae (Source: Subramanya, 2006)

| Method | P (probability) |
|---|---|
| California | $\frac{m}{N}$ |
| Hazen | $\frac{m-0.5}{N}$ |
| Weibull | $\frac{m}{N+1}$ |
| Chegodayev | $\frac{m-0.3}{N+0.4}$ |
| Blom | $\frac{m-0.44}{N+0.12}$ |

m = rank assigned to the data after arranging them in descending order of magnitude

Thus the maximum value is assigned m =1, the second largest value (m =2), and the lowest value m =N.

N = number of records.

Weibull formula is the most commonly used plotting position formula.

- Having calculated P and T for all the events in the series, the variation of rainfall magnitude is plotted against the corresponding T on semi-log or log-log paper.
- The rainfall magnitude for any recurrence interval can be determined by extrapolating the plot between magnitude and recurrence interval.
- Empirical procedures can give good results for small extrapolations but the errors increased with the amount of extrapolation.
- For more accurate results, analytical methods using frequency factor are used.

Example

For a station A, the recorded annual 24 h maximum rainfall is given below.

| Year | Rainfall (cm) | Year | Rainfall (cm) | Year | Rainfall (cm) |
|---|---|---|---|---|---|
| 1950 | 13.0 | 1957 | 12.5 | 1964 | 8.5 |
| 1951 | 12.0 | 1958 | 11.2 | 1965 | 7.5 |
| 1952 | 7.6 | 1959 | 8.9 | 1966 | 6.0 |
| 1953 | 14.3 | 1960 | 8.9 | 1967 | 8.4 |
| 1954 | 16.0 | 1961 | 7.8 | 1968 | 10.8 |
| 1955 | 9.6 | 1962 | 9.0 | 1969 | 10.6 |
| 1956 | 8.0 | 1963 | 10.2 | 1970 | 8.3 |
| | | | | 1971 | 9.5 |

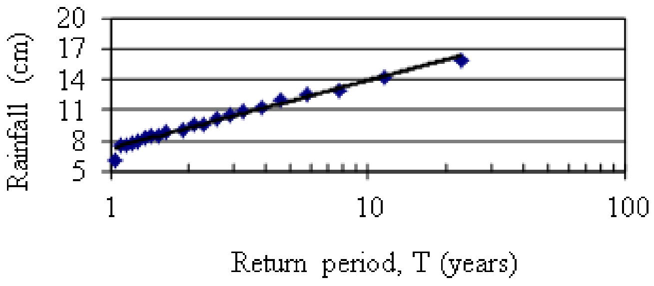
(a) Estimate the 24 h maximum rainfall with return periods of 13 and 50 years.

(b) What would be the probability of a rainfall of magnitude equal to or exceeding 10 cm occurring in 24 h at station A.

Solution

| m | Rainfall, cm | P =m/n+1 | T=1/P | m | Rainfall, cm | P =m/n+1 | T=1/P |
|---|---|---|---|---|---|---|---|
| 1 | 16 | 0.043 | 23.00 | 12 | 9 | 0.522 | 1.92 |
| 2 | 14.3 | 0.087 | 11.50 | 13 | 8.9 | | |
| 3 | 13 | 0.130 | 7.67 | 14 | 8.9 | 0.609 | 1.64 |
| 4 | 12.5 | 0.174 | 5.75 | 15 | 8.5 | 0.652 | 1.53 |
| 5 | 12 | 0.217 | 4.60 | 16 | 8.4 | 0.696 | 1.44 |
| 6 | 11.2 | 0.261 | 3.83 | 17 | 8.3 | 0.739 | 1.35 |
| 7 | 10.8 | 0.304 | 3.29 | 18 | 8 | 0.783 | 1.28 |
| 8 | 10.6 | 0.348 | 2.88 | 19 | 7.8 | 0.826 | 1.21 |
| 9 | 10.2 | 0.391 | 2.56 | 20 | 7.6 | 0.870 | 1.15 |
| 10 | 9.6 | 0.435 | 2.30 | 21 | 7.5 | 0.913 | 1.10 |
| 11 | 9.5 | 0.478 | 2.09 | 22 | 6 | 0.957 | 1.05 |

Rainfall Frequency Curve



(a) After interpolating and extrapolating the above graph, we can determine rainfall magnitude for 13 and 50 year return period respectively.

13 year RI = 14.55 cm

50 year RI = 18.00 cm

(b) For Rainfall = 10 cm, T =2.4 years and P = 0.417.

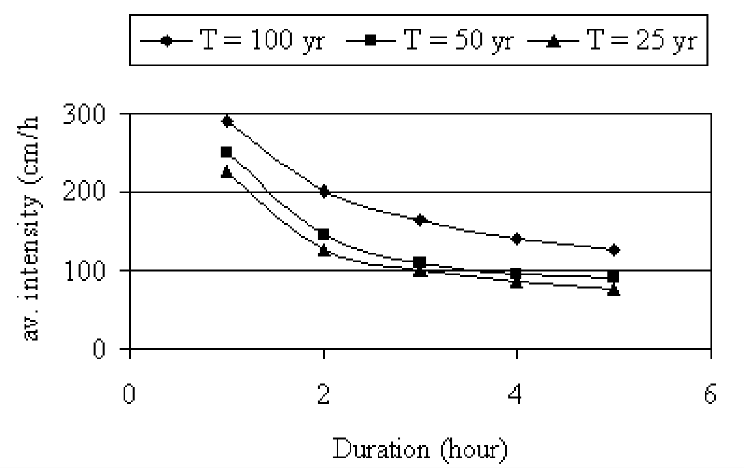
### 10.2 Intensity-duration-frequency (IDF) Relationship

- In many studies related to watershed management, such as runoff disposal and erosion control, it is necessary to know the rainfall intensities of different durations and return periods.

The relationship between intensity (i, cm/hr), duration (d, hour) and return period (T, years) can be expressed as follows:

$$i = \frac{KT^x}{(D+a)^n} \qquad (10.1)$$

Where K, x, a and n are constants for a given catchment.



Intensity –Duration-Frequency Curve

Typical values of constants K,, x, a, and n for a few selected cities are given below:

Table – Values of constants in equation (29)

(Source: CSWCRTI- Dehradun)

| City | K | $x$ | $A$ | n |
|---|---|---|---|---|
| Bhopal | 6.93 | 0.189 | 0.50 | 0.878 |
| Nagpur | 11.45 | 0.156 | 1.25 | 1.032 |
| Chandigarh | 5.82 | 0.160 | 0.40 | 0.750 |
| Bellary | 6.16 | 0.694 | 0.50 | 0.972 |
| Raipur | 4.68 | 0.139 | 0.15 | 0.928 |

Example: Compute 10 year, 1 hour design rainfall intensity for Bhopal and Nagpur.

Solution:

For Bhopal

$$i = \frac{6.93(10)^{0.189}}{(1+0.50)^{0.878}} = \frac{10.708}{1.427} = 7.50$$

cm/hr

For Nagpur

$$i = \frac{11.45(10)^{0.156}}{(1+1.25)^{1.032}} = \frac{16.398}{2.309} = 7.10$$

cm/hr

Example

Perform the frequency analysis using different graphical methods for the following annual rainfall data. Estimate annual rainfall amount with 100 and 50 years return periods.

| Year | Annual rain (mm) | Year | Annual rain (mm) | Year | Annual rain (mm) |
|---|---|---|---|---|---|
| 1950 | 115 | 1959 | 71.4 | 1968 | 68.6 |
| 1951 | 96.5 | 1960 | 83 | 1969 | 67 |

| | | | | | |
|---|---|---|---|---|---|
| 1952 | 78 | 1961 | 96.5 | 1970 | 93 |
| 1953 | 89.5 | 1962 | 88.3 | 1971 | 108.8 |
| 1954 | 94.7 | 1963 | 70.6 | 1972 | 104.2 |
| 1955 | 73.3 | 1964 | 84.5 | 1973 | 89 |
| 1956 | 79.3 | 1965 | 92.7 | 1974 | 86 |
| 1957 | 87.1 | 1966 | 101.8 | | |
| 1958 | 124.8 | 1967 | 76.4 | | |

Solution

| Year | Annual rain (mm) | Data in Descending Order | Rank (m) | $P_{Ca}$= (m/N) | $T_{Ca}$= 1/$P_{ca}$ | $P_H$ = (m-0.5)/N | $T_H$= 1/$P_H$ | $P_W$= m/(N+1) | $T_W$= 1/$P_W$ |
|---|---|---|---|---|---|---|---|---|---|
| | | | | California | | Hazen | | Weibull | |
| 1950 | 115 | 124.8 | 1 | 0.04 | 25.00 | 0.02 | 50.00 | 0.038 | 26.00 |
| 1951 | 96.5 | 115 | 2 | 0.08 | 12.50 | 0.06 | 16.67 | 0.077 | 13.00 |
| 1952 | 78 | 108.8 | 3 | 0.12 | 8.33 | 0.1 | 10.00 | 0.115 | 8.67 |
| 1953 | 89.5 | 104.2 | 4 | 0.16 | 6.25 | 0.14 | 7.14 | 0.154 | 6.50 |
| 1954 | 94.7 | 101.8 | 5 | 0.2 | 5.00 | 0.18 | 5.56 | 0.192 | 5.20 |
| 1955 | 73.3 | 96.5 | 6 | 0.24 | 4.17 | 0.22 | 4.55 | 0.231 | 4.33 |
| 1956 | 79.3 | 96.5 | 7 | 0.28 | 3.57 | 0.26 | 3.85 | 0.269 | 3.71 |
| 1957 | 87.1 | 94.7 | 8 | 0.32 | 3.13 | 0.3 | 3.33 | 0.308 | 3.25 |
| 1958 | 124.8 | 93 | 9 | 0.36 | 2.78 | 0.34 | 2.94 | 0.346 | 2.89 |

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 1959 | 71.4 | 92.7 | 10 | 0.4 | 2.50 | 0.38 | 2.63 | 0.385 | 2.60 |
| 1960 | 83 | 89.5 | 11 | 0.44 | 2.27 | 0.42 | 2.38 | 0.423 | 2.36 |
| 1961 | 96.5 | 89 | 12 | 0.48 | 2.08 | 0.46 | 2.17 | 0.462 | 2.17 |
| 1962 | 88.3 | 88.3 | 13 | 0.52 | 1.92 | 0.5 | 2.00 | 0.500 | 2.00 |
| 1963 | 70.6 | 87.1 | 14 | 0.56 | 1.79 | 0.54 | 1.85 | 0.538 | 1.86 |
| 1964 | 84.5 | 86 | 15 | 0.6 | 1.67 | 0.58 | 1.72 | 0.577 | 1.73 |
| 1965 | 92.7 | 84.5 | 16 | 0.64 | 1.56 | 0.62 | 1.61 | 0.615 | 1.63 |
| 1966 | 101.8 | 83 | 17 | 0.68 | 1.47 | 0.66 | 1.52 | 0.654 | 1.53 |
| 1967 | 76.4 | 79.3 | 18 | 0.72 | 1.39 | 0.7 | 1.43 | 0.692 | 1.44 |
| 1968 | 68.6 | 78 | 19 | 0.76 | 1.32 | 0.74 | 1.35 | 0.731 | 1.37 |
| 1969 | 67 | 76.4 | 20 | 0.8 | 1.25 | 0.78 | 1.28 | 0.769 | 1.30 |
| 1970 | 93 | 73.3 | 21 | 0.84 | 1.19 | 0.82 | 1.22 | 0.808 | 1.24 |
| 1971 | 108.8 | 71.4 | 22 | 0.88 | 1.14 | 0.86 | 1.16 | 0.846 | 1.18 |
| 1972 | 104.2 | 70.6 | 23 | 0.92 | 1.09 | 0.9 | 1.11 | 0.885 | 1.13 |
| 1973 | 89 | 68.6 | 24 | 0.96 | 1.04 | 0.94 | 1.06 | 0.923 | 1.08 |
| 1974 | 86 | 67 | 25 | 1 | 1.00 | 0.98 | 1.02 | 0.962 | 1.04 |

| Year | Annual rain (mm) | Data in Descending Order | Rank (m) | $P_{Ch}$= (m-0.3)/(N+0.4) | $T_{Ch}$ = 1/$P_{Ch}$ | $P_B$= (m-0.44)/(N+0.12) | $T_B$= 1/$P_B$ | $P_G$= (m-3/8)/(N+1/4) | $T_G$= 1/$P_G$ |
|---|---|---|---|---|---|---|---|---|---|
| | | | | Chegodayev | | Blom | | Gringorten | |
| 1950 | 115 | 124.8 | 1 | 0.03 | 36.29 | 0.02 | 44.86 | 0.02 | 40.40 |
| 1951 | 96.5 | 115 | 2 | 0.07 | 14.94 | 0.06 | 16.10 | 0.06 | 15.54 |
| 1952 | 78 | 108.8 | 3 | 0.11 | 9.41 | 0.10 | 9.81 | 0.10 | 9.62 |
| 1953 | 89.5 | 104.2 | 4 | 0.15 | 6.86 | 0.14 | 7.06 | 0.14 | 6.97 |

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 1954 | 94.7 | 101.8 | 5 | 0.19 | 5.40 | 0.18 | 5.51 | 0.18 | 5.46 |
| 1955 | 73.3 | 96.5 | 6 | 0.22 | 4.46 | 0.22 | 4.52 | 0.22 | 4.49 |
| 1956 | 79.3 | 96.5 | 7 | 0.26 | 3.79 | 0.26 | 3.83 | 0.26 | 3.81 |
| 1957 | 87.1 | 94.7 | 8 | 0.30 | 3.30 | 0.30 | 3.32 | 0.30 | 3.31 |
| 1958 | 124.8 | 93 | 9 | 0.34 | 2.92 | 0.34 | 2.93 | 0.34 | 2.93 |
| 1959 | 71.4 | 92.7 | 10 | 0.38 | 2.62 | 0.38 | 2.63 | 0.38 | 2.62 |
| 1960 | 83 | 89.5 | 11 | 0.42 | 2.37 | 0.42 | 2.38 | 0.42 | 2.38 |
| 1961 | 96.5 | 89 | 12 | 0.46 | 2.17 | 0.46 | 2.17 | 0.46 | 2.17 |
| 1962 | 88.3 | 88.3 | 13 | 0.50 | 2.00 | 0.50 | 2.00 | 0.50 | 2.00 |
| 1963 | 70.6 | 87.1 | 14 | 0.54 | 1.85 | 0.54 | 1.85 | 0.54 | 1.85 |
| 1964 | 84.5 | 86 | 15 | 0.58 | 1.73 | 0.58 | 1.73 | 0.58 | 1.73 |
| 1965 | 92.7 | 84.5 | 16 | 0.62 | 1.62 | 0.62 | 1.61 | 0.62 | 1.62 |
| 1966 | 101.8 | 83 | 17 | 0.66 | 1.52 | 0.66 | 1.52 | 0.66 | 1.52 |
| 1967 | 76.4 | 79.3 | 18 | 0.70 | 1.44 | 0.70 | 1.43 | 0.70 | 1.43 |
| 1968 | 68.6 | 78 | 19 | 0.74 | 1.36 | 0.74 | 1.35 | 0.74 | 1.36 |
| 1969 | 67 | 76.4 | 20 | 0.78 | 1.29 | 0.78 | 1.28 | 0.78 | 1.29 |
| 197 | 93 | 73.3 | 21 | 0.81 | 1.23 | 0.82 | 1.22 | 0.82 | 1.22 |

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | | | |
| 1971 | 108.8 | 71.4 | 22 | 0.85 | 1.17 | 0.86 | 1.17 | 0.86 | 1.17 |
| 1972 | 104.2 | 70.6 | 23 | 0.89 | 1.12 | 0.90 | 1.11 | 0.90 | 1.12 |
| 1973 | 89 | 68.6 | 24 | 0.93 | 1.07 | 0.94 | 1.07 | 0.94 | 1.07 |
| 1974 | 86 | 67 | 25 | 0.97 | 1.03 | 0.98 | 1.02 | 0.98 | 1.03 |

Example

Develop intensity-duration-frequency curve for given data

| 5 min | | 15 min | | 60 min | | 120 min | |
|---|---|---|---|---|---|---|---|
| Year | Rainfall | Year | Rainfall | Year | Rainfall | Year | Rainfall |
| | mm | | mm | | mm | | mm |
| 1908 | 0.79 | 1910 | 1.34 | 1916 | 2.09 | 1917 | 2.91 |
| 1910 | 0.71 | 1914 | 1.13 | 1914 | 1.87 | 1914 | 2.58 |
| 1918 | 0.67 | 1916 | 1.05 | 1910 | 1.64 | 1911 | 2.28 |
| 1914 | 0.66 | 1918 | 0.97 | 1915 | 1.39 | 1908 | 2.06 |
| 1916 | 0.6 | 1909 | 0.91 | 1908 | 1.34 | 1916 | 1.77 |
| 1912 | 0.56 | 1913 | 0.86 | 1912 | 1.27 | 1913 | 1.58 |
| 1917 | 0.45 | 1915 | 0.84 | 1918 | 1.19 | 1910 | 1.49 |
| 1915 | 0.39 | 1917 | 0.76 | 1917 | 1.14 | 1909 | 1.45 |
| 1913 | 0.3 | 1911 | 0.61 | 1909 | 1.08 | 1912 | 1.4 |
| 1911 | 0.22 | 1912 | 0.56 | 1911 | 1.05 | 1915 | 1.35 |
| 1909 | 0.15 | 1908 | 0.46 | 1913 | 1.03 | 1918 | 1.28 |

Solution

| 5 min | | 15 min | | 60 min | | 120 min | |
|---|---|---|---|---|---|---|---|
| Year | Rainfall Intensity | Year | Rainfall Intensity | Year | Rainfall Intensity | Year | Rainfall Intensity |
| | cm/h | | cm/h | | cm/h | | cm/h |
| 1908 | 0.948 | 1908 | 0.536 | 1908 | 0.209 | 1915 | 0.1455 |
| 1921 | 0.852 | 1915 | 0.452 | 1904 | 0.187 | 1908 | 0.129 |
| 1915 | 0.804 | 1904 | 0.42 | 1915 | 0.164 | 1904 | 0.114 |
| 1934 | 0.792 | 1921 | 0.388 | 1926 | 0.139 | 1921 | 0.103 |
| 1929 | 0.72 | 1926 | 0.364 | 1921 | 0.134 | 1926 | 0.0885 |
| 1926 | 0.672 | 1934 | 0.344 | 1914 | 0.127 | 1917 | 0.079 |
| 1931 | 0.54 | 1929 | 0.336 | 1931 | 0.119 | 1914 | 0.0745 |
| 1904 | 0.468 | 1931 | 0.304 | 1934 | 0.114 | 1931 | 0.0725 |
| 1917 | 0.36 | 1911 | 0.244 | 1929 | 0.108 | 1934 | 0.07 |
| 1914 | 0.264 | 1917 | 0.224 | 1911 | 0.105 | 1929 | 0.0675 |
| 1911 | 0.18 | 1914 | 0.184 | 1917 | 0.103 | 1911 | 0.064 |

| For 5 min duration | | | | | |
|---|---|---|---|---|---|
| Year | Rainfall | Rainfall Intensity | Rank | P= (m/(n+1)) | T= (1/P) |
| | mm | cm/h | | | |
| 1908 | 0.79 | 0.948 | 1 | 0.0833 | 12.00 |
| 1921 | 0.71 | 0.852 | 2 | 0.1667 | 6.00 |
| 1915 | 0.67 | 0.804 | 3 | 0.2500 | 4.00 |
| 1934 | 0.66 | 0.792 | 4 | 0.3333 | 3.00 |
| 1929 | 0.6 | 0.72 | 5 | 0.4167 | 2.40 |

| | | | | | |
|---|---|---|---|---|---|
| 1926 | 0.56 | 0.672 | 6 | 0.5000 | 2.00 |
| 1931 | 0.45 | 0.54 | 7 | 0.5833 | 1.71 |
| 1904 | 0.39 | 0.468 | 8 | 0.6667 | 1.50 |
| 1917 | 0.3 | 0.36 | 9 | 0.7500 | 1.33 |
| 1914 | 0.22 | 0.264 | 10 | 0.8333 | 1.20 |
| 1911 | 0.15 | 0.18 | 11 | 0.9167 | 1.09 |

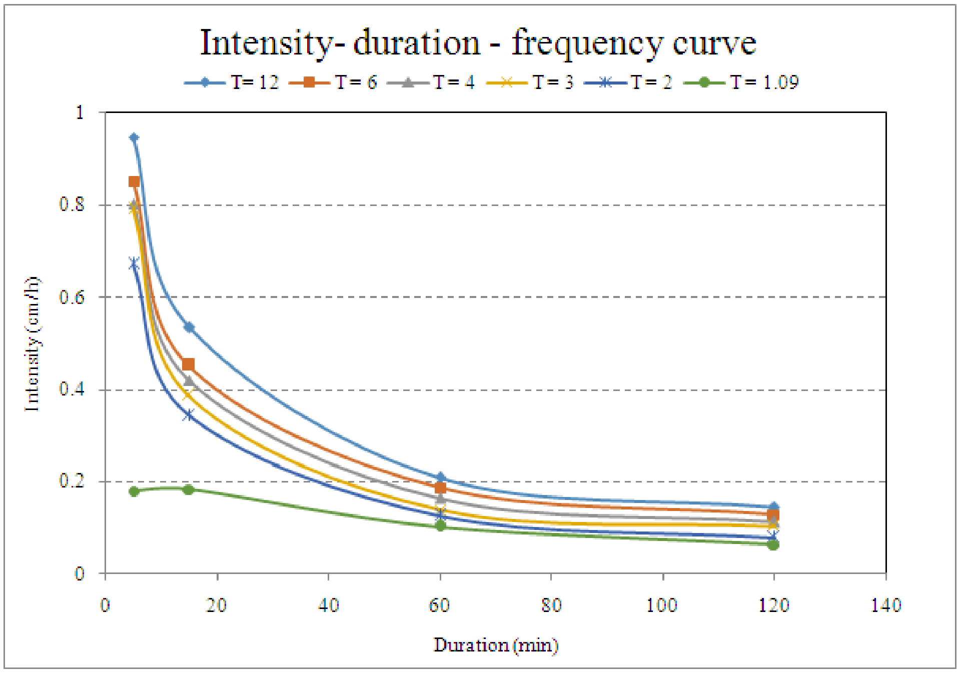
| For 15 min duration | | | | | |
|---|---|---|---|---|---|
| Year | Rainfall | Rainfall Intensity | Rank | P= (m/(n+1)) | T= (1/P) |
| | mm | cm/h | | | |
| 1908 | 1.34 | 0.536 | 1 | 0.0833 | 12.00 |
| 1921 | 1.13 | 0.452 | 2 | 0.1667 | 6.00 |
| 1915 | 1.05 | 0.42 | 3 | 0.2500 | 4.00 |
| 1934 | 0.97 | 0.388 | 4 | 0.3333 | 3.00 |
| 1929 | 0.91 | 0.364 | 5 | 0.4167 | 2.40 |
| 1926 | 0.86 | 0.344 | 6 | 0.5000 | 2.00 |
| 1931 | 0.84 | 0.336 | 7 | 0.5833 | 1.71 |
| 1904 | 0.76 | 0.304 | 8 | 0.6667 | 1.50 |
| 1917 | 0.61 | 0.244 | 9 | 0.7500 | 1.33 |
| 1914 | 0.56 | 0.224 | 10 | 0.8333 | 1.20 |
| 1911 | 0.46 | 0.184 | 11 | 0.9167 | 1.09 |

| For 60 min duration | | | | | |
|---|---|---|---|---|---|
| Year | Rainfall | Rainfall Intensity | Rank | P= (m/(n+1)) | T= (1/P) |
| | mm | cm/h | | | |

| | | | | | |
|---|---|---|---|---|---|
| 1908 | 2.91 | 0.209 | 1 | 0.0833 | 12.00 |
| 1921 | 2.58 | 0.187 | 2 | 0.1667 | 6.00 |
| 1915 | 2.28 | 0.164 | 3 | 0.2500 | 4.00 |
| 1934 | 2.06 | 0.139 | 4 | 0.3333 | 3.00 |
| 1929 | 1.77 | 0.134 | 5 | 0.4167 | 2.40 |
| 1926 | 1.58 | 0.127 | 6 | 0.5000 | 2.00 |
| 1931 | 1.49 | 0.119 | 7 | 0.5833 | 1.71 |
| 1904 | 1.45 | 0.114 | 8 | 0.6667 | 1.50 |
| 1917 | 1.4 | 0.108 | 9 | 0.7500 | 1.33 |
| 1914 | 1.35 | 0.105 | 10 | 0.8333 | 1.20 |
| 1911 | 1.28 | 0.103 | 11 | 0.9167 | 1.09 |

| For 120 min duration | | | | | |
|---|---|---|---|---|---|
| Year | Rainfall | Rainfall Intensity | Rank | P= (m/(n+1)) | T= (1/P) |
| | mm | cm/h | | | |
| 1908 | 3.06 | 0.1455 | 1 | 0.0833 | 12.00 |
| 1921 | 2.73 | 0.129 | 2 | 0.1667 | 6.00 |
| 1915 | 2.43 | 0.114 | 3 | 0.2500 | 4.00 |
| 1934 | 2.21 | 0.103 | 4 | 0.3333 | 3.00 |
| 1929 | 1.92 | 0.0885 | 5 | 0.4167 | 2.40 |
| 1926 | 1.73 | 0.079 | 6 | 0.5000 | 2.00 |
| 1931 | 1.64 | 0.0745 | 7 | 0.5833 | 1.71 |
| 1904 | 1.6 | 0.0725 | 8 | 0.6667 | 1.50 |
| 1917 | 1.55 | 0.07 | 9 | 0.7500 | 1.33 |
| 1914 | 1.5 | 0.0675 | 10 | 0.8333 | 1.20 |
| 1911 | 1.43 | 0.064 | 11 | 0.9167 | 1.09 |

| Recurrence Interval | Rainfall Intensity (cm/h) | | | |
|---|---|---|---|---|
| T= (1/P), Year | 5 | 15 | 60 | 120 |
| | min | min | min | min |
| 1.090909091 | 0.18 | 0.184 | 0.103 | 0.064 |
| 1.2 | 0.264 | 0.224 | 0.105 | 0.0675 |
| 1.333333333 | 0.36 | 0.244 | 0.108 | 0.07 |
| 1.5 | 0.468 | 0.304 | 0.114 | 0.0725 |
| 1.714285714 | 0.54 | 0.336 | 0.119 | 0.0745 |
| 2 | 0.672 | 0.344 | 0.127 | 0.079 |
| 2.4 | 0.72 | 0.364 | 0.134 | 0.0885 |
| 3 | 0.792 | 0.388 | 0.139 | 0.103 |
| 4 | 0.804 | 0.42 | 0.164 | 0.114 |
| 6 | 0.852 | 0.452 | 0.187 | 0.129 |
| 12 | 0.948 | 0.536 | 0.209 | 0.1455 |



## Module.3 Hydrological Abstractions

## Lesson 11 Interception and Depression Storage

### 11.1 Interception

Interception is the removal of water that wets and adheres to plant foliage, buildings, and other objects above ground surface. This water is subsequently removed from the surface through evaporation. Interception can be as high as 2 mm during a single rainfall event, but typically removes about 0.5 mm during a single rainfall/storm event. The quantity of water removed through interception is usually not significant for an isolated storm, but, when added over a period of time, it can be significant. It is thought that as much as 25 percent of the total annual precipitation for certain heavily forested areas of the Pacific Northwest of the United States is lost through interception during the course of a year. Interception is the process by which water is captured on vegetation (leaves, bark, grasses, crops, etc.) during a precipitation event. Intercepted precipitation is not available for runoff or infiltration, but instead is returned to the atmosphere through evaporation. Interception losses generally occur during the first part of a precipitation event and the interception loss rate trends toward zero rather quickly (Fig. 11.1).

Interception losses are described by the following equation

$L_{i = S + KEt}$

Where,

$L_i$ is the total volume of water intercepted.

S is the interception storage.

K is the ratio of the surface area of the leaves to the area of the entire canopy.

E is the rate of evaporation during the precipitation event, and

t is time.

As the Horton equation suggests, the total interception is dependent on the storm duration, as longer duration storms allow more evaporation from the canopy during the storm event. The intensity of the storm also plays a role in canopy interception however, there is debate as to whether intensity increases or decreases interception storage in canopy. There are many other factors that influence interception potential. Interception varies widely by season as deciduous trees lose much of their canopy storage potential during winter months.

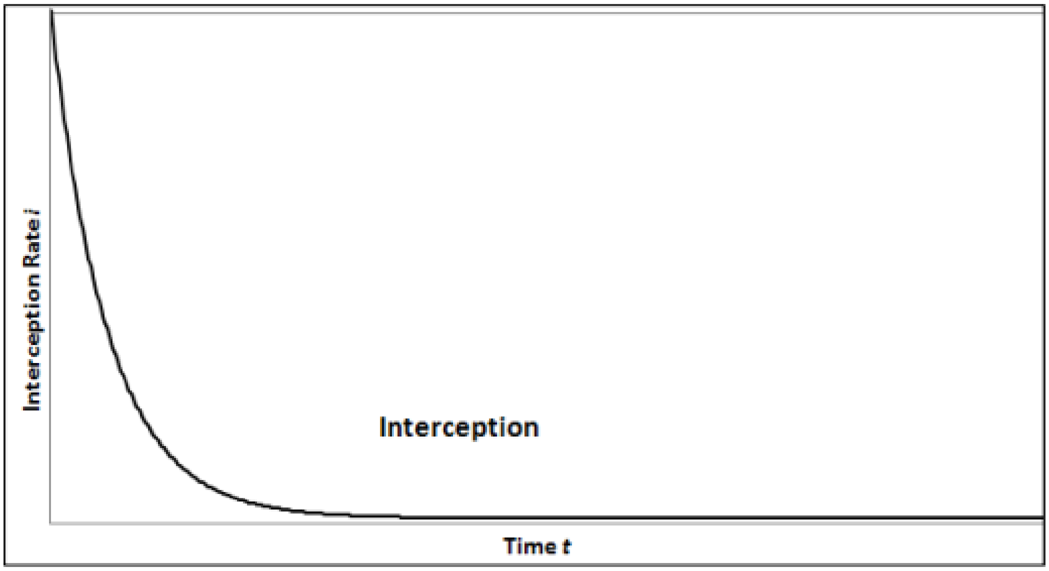


Fig. 11.1.Interception Rate versusTime.(Source: Singh, 1994)

There are three Main Components of Interception:

1. Interception Loss

2. Throughfall

3. Stemflow

1. Interception loss: The water that is retained by vegetation surfaces that is later evaporated into the atmosphere, or absorbed by the plant.Interception loss prevents water from reaching the ground surface and is regarded as a primary water loss.

2. Throughfall: The water which falls through spaces in the vegetation canopy, or which drips from the leaves, twigs and stems and falls to the ground.

3. Stemflow: The water which trickles along the stems and branches and down the main stem or trunk to the ground surface.

### 11.2 Factors Affecting Interception

Interception Storage: The ability of vegetation surfaces to collect and retain Precipitation, capacity will be highest at the onset of rainfall when the vegetation is dry, when water is held by surface tension.

Evaporation: Even when the interception storage capacity is exceeded water may be lost by evaporation off leaf surfaces, which increases in windy conditions, though the interception storage capacity may be reduced with increased windspeed.

Duration of Rainfall: Influences interception by determining the balance between reduced storage of water on vegetation surfaces and increased evaporative loss over time.Total interception losses increase with duration of rainfall (but only gradually), though the relative importance of interception decreases with time.The importance of interception decreases with time, due to duration of rainfall and changes in the interception storage capacity.

Rainfall Frequency: The highest levels of interception loss occur when the leaves are dry and interception storage is large, so the frequency of re-wetting is more significant than the duration and amount of rainfall.

Precipitation Type: The contrast between rain and snow.

Snow clings to leaves and branches more, but interception loss is limited due to low temperatures and evaporation rates.

May be a contrast between coniferous and deciduous trees

Type and Morphology of the Vegetation Cover: Different vegetation types have:

Different interception storage capacities

Different aerodynamic roughness characteristics

Different rates of evaporation from their surfaces.

Interception losses are generally greater from trees than other types of vegetation (grasses and agricultural crops)due to the greater aerodynamic roughness of trees in promoting increased evaporation in wet conditionsor to their higher interception capacities (in some cases) especially when wetted and dried frequently. Interception losses are greater from coniferous forests than from deciduous woodlands.

## 11.3 Depression Storage

Depression storage is the term applied to water that is lost because it becomes trapped in the numerous small depressions that are characteristic of any natural surface. When water temporarily accumulates in a low point with no possibility for escape as runoff, the accumulation is referred to as depression storage. The amount of water that is lost due to depression storage varies greatly with the land use. A paved surface will not detain as much water as a recently furrowed field. The relative importance of depression storage in determining the runoff from a given storm depends on the amount and intensity of precipitation in the storm. Typical values for depression storage range from 1 to 8 mm (0.04 to 0.3 in) with some values as high as 15 mm (0.6 in) per event. As with evaporation and transpiration, depression storage is generally not directly calculated in highway design.

If the soil surface has a low infiltration capacity and low hydraulic conductivity, and if the topography allows for surface storage, then water may be stored at the surface in small pools or depressions. These water-filled depressions, called vernal pools, are often seasonal features that form because of perched water tables. These depression storage areas may become hydrologically connected during high water conditions and develop a flow network to deliver water to streams or other surface water bodies. Depression storage refers to small low points in undulating terrain that can store precipitation that otherwise would become runoff. The precipitation stored in these depressions is then either removed through infiltration into the ground or by evaporation. Depression storage exists on both pervious and impervious surfaces.

The volume of water in depression storage at any time during a precipitation event can be approximated as (Linsley 1982):

$$V = S_d(1 - e^{-kP_e}) \quad (11.2)$$

Where,

V is the volume of water in depression storage.

$S_d$ is the maximum storage capacity of the depression.

$P_e$ is the rainfall excess, and

k is a constant equal to $1/S_d$.

## 11.4 Factors Affecting Depression Storage

(1) Nature of terrain

(2) Slope

(3) Type of soil surface

(4) Land use

(5) Antecedent rainfall

(6) Time

Empirical Estimates of Depression Storage (for storms)

| | |
|---|---|
| Sand | 0.20 inches |
| Loam | 0.15 inches |
| Clay | 0.10 inches |
| Impervious areas | 0.062 inches |
| Pervious urban | 0.25 inches |

## Lesson 12 Evaporation

### 12.1 Evaporation Process

Evaporation includes all processes by which water returns to the atmosphere as water vapour: evaporation of intercepted rain and snow; evaporation from bare soil and water bodies, such as ponds, lakes, and streams; and transpiration from plant leaves. Evaporation (and Transpiration) are small for a runoff event and can be neglected.The bulk of these abstractions take place during the time between runoff events, which is usually long.Hence, these are more important during this time interval.

Evaporation requires the following four conditions:

(1) Available water

(2) Higher humidity at the evaporative surface (i.e., vapour pressure) than in the surrounding air

(3) Energy to evaporate the water and

(4) Movement, or transfer, of water vapour away from the evaporative surface.

Energy required to evaporate water depends on incoming solar radiation, reflectivity of the evaporative surface, and air and surface temperature. Diffusion and convection move the vapour away from the surface. Increasing solar radiation, air temperature, and wind speed and decreasing atmospheric humidity all create an increase in evaporation rate. Evaporation is enhanced by warm air flowing over a cooler surface (e.g., air moving from dry rangeland over an irrigated crop or a small lake), but decreases rapidly with distance from the boundary between dry and wet surfaces.Intercepted rain or snow, and open water are in direct contact with the air. Both boundary-layer and aerodynamic resistance affect water loss from these surfaces. The boundary layer is a thin layer adjacent to a surface through which vapour moves by diffusion. Aerodynamic resistance describes vapour movement in the rest of the atmosphere. Both resistances depend on the size and shape of the evaporative surface, and both decrease as wind speed increases. Tree needles have a lower boundarylayer resistance than large leaves and a much lower resistance than that of a lake. Trees generate more turbulence to airflow than smooth surfaces, such as a lake; consequently, trees have lower aerodynamic resistances at the same wind speed. The combined resistances for a wet surface are relatively low compared to the resistance to movement of water from inside leaves or from below a dry soil surface.

## 12.2 Factors Affecting Evaporation

The factors that affect evaporation are:

1. Wind: When wind speed is high it assists evaporation.

2. Heat:Evaporation is more in summer as compared to winter.

3. Exposed surface area:For instance, a wet cloth spread out dries faster than when folded.

4. Humidity: Dryness assists evaporation; for instance, clothes dry faster in summer than during the monsoon when the air is humid.

5. Nature of the liquid: Rate of evaporation depends upon the type of liquid; for example, petrol evaporates faster than water.

6.Vapour pressure: If pressure is applied on the surface of a liquid, evaporation is hindered; consider, for example, the case of a pressure cooker.

## 12.3Measurement of Evaporation

### 12.3.1 Lysimeter

A lysimeter is a measuring device which can be used to measure the amount of actual evapotranspiration which is released by plants, usually crops or trees. By recording the amount of precipitation that an area receives and the amount lost through the soil, the amount of water lost to evapotranspiration can be calculated.In general, a lysimeter consists of the soil-filled inner container and retaining walls or an outer container, as well as special devices for measuring percolation and changes in the soil-moisture content. There is no universal international standard lysimeter for measuring evapotranspiration. The surface area of lysimeters in use varies from 0.05 to some 100 $m^2$ and their depth varies from 0.1 to 5 m.

According to their method of operation, lysimeters can be classified into non-weighable and weighable instruments. Each of these devices has its special merits and drawbacks, and the choice of any type of lysimeter depends on the problem to be studied.

Monolithic weighable lysimeters are a tool for water balance studies and solute transport determination.

Lysimeters are of two types:

1. Weighing
2. Non-weighing

Non-weighable (percolation-type) lysimeters can be used only for long-term measurements, unless the soil-moisture content can be measured by some independent and reliable technique. Large-area percolation-type lysimeters are used for water budget and evapotranspiration studies of tall, deep rooting vegetation cover, such as mature trees. Small, simple types of lysimeters in areas with bare soil or grass and crop cover could provide useful results for practical purposes under humid conditions. This type of lysimeter can easily be installed and maintained at a low cost and is, therefore, suitable for network operations.

Weighable lysimeters, unless of a simple microlysimeter-type for soil evaporation, are much more expensive, but their advantage is that they secure reliable and precise estimates of short-term values of evapotranspiration, provided that the necessary design, operation and siting precautions have been taken.

Several weighing techniques using mechanical or hydraulic principles have been developed. The simpler, small lysimeters are usually lifted out of their sockets and transferred to mechanical scales by means of mobile cranes. The container of a lysimeter can be mounted on a permanently installed mechanical scale for continuous recording. The design of the weighing and recording system can be considerably simplified by using load cells with strain gauges of variable electrical resistance. The hydraulic weighing systems use theprinciple of fluid displacement resulting from the changing buoyancy of a floating container (socalled floating lysimeter), or the principle of fluid pressure changes in hydraulic load cells.The large weighable and recording lysimeters are recommended for precision measurements in research centres and for standardization and parameterization of other methods of evapotranspiration measurement and the modelling of evapotranspiration. Small weighable types of lysimeters are quite useful and suitable for network operation. Microlysimeters for soil evaporation are a relatively new phenomenon.

### 12.3.2 Pan Evaporation

Pan evaporation is a measurement that combines or integrates the effects of several climate elements: temperature, humidity, rain fall, drought dispersion, solar radiation, and wind. Evaporation is greatest on hot, windy, dry, sunny days; and is greatly reduced when clouds block the sun and when air is cool, calm, and humid.Pan evaporation measurements enable farmers and ranchers to understand how much water their crops will need. There are many types of evaporation pans used by farmers. However, the universal pan is the United States Weather Bureau (USWB) Class A pan evaporimeter (Fig.12.1). It is important to use the same dimensions as this universal pan, mainly because the effect of wind and temperature on evaporation will vary with the surface area and the depth of water in the pan. Evaporation and irrigation replacements cannot be compared between sites if non standard pans are used.

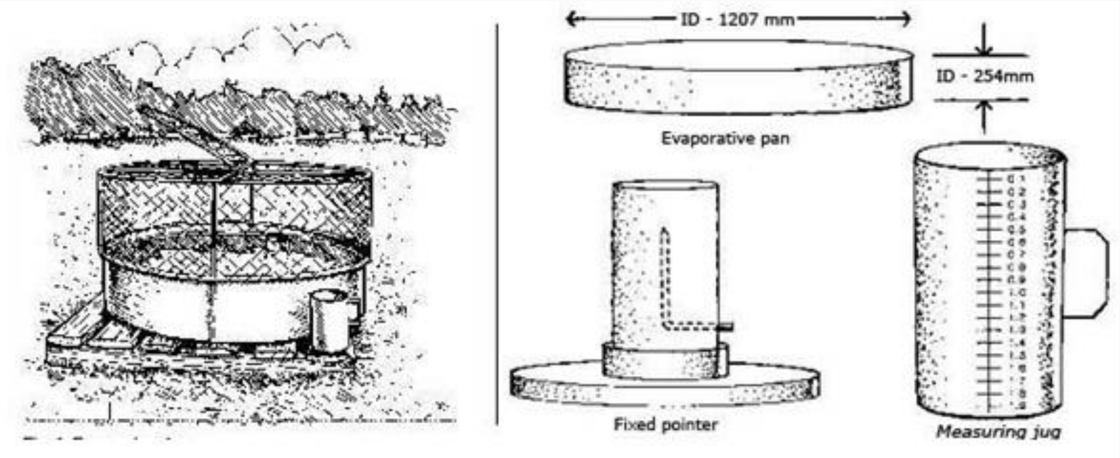


Fig. 12.1. USWB Class A Pan Evaporimeter.

**Construction:**

There are three parts to an evaporimeter (Fig. 12.1). All parts can be made very cheaply with common materials. Alternatively a complete unit can be purchased at considerably greater cost. The following is a description of how to construct the three components of the evaporimeter.

Evaporation Pan:The evaporation pan must be made to the standard specifications of an internal diameter of 1207 mm and height of 254 mm using 20 gauge galvanised iron. The standard material is galvanised iron as alternatives will have different thermal and reflectance properties, therefore altering the evaporation rate. It is best to have the pan made by either a galvanised tank manufacturer or an engineering firm. Before the pan is sited in the field it should be checked for leaks.

**Fixed Pointer:**

The fixed pointer that sits inside the pan can be made from standard irrigation fittings and a piece of stainless steel rod (Fig. 12.2). There are three parts to the fixed pointer:

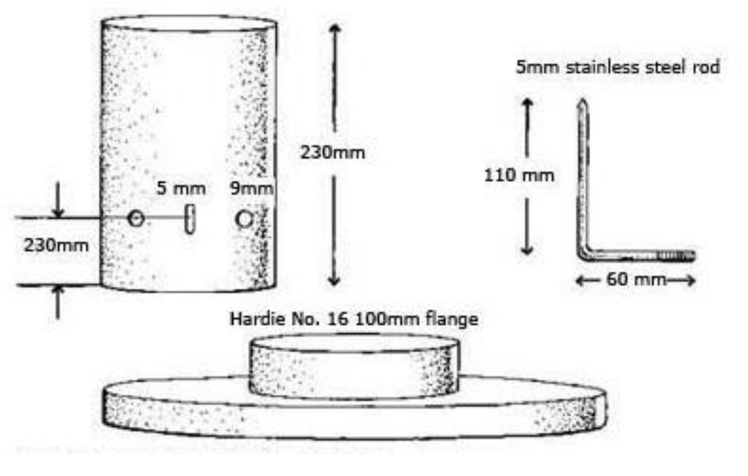


Fig. 12.2.Components of fixed pointer.

(Source:http://www.depi.vic.gov.au/agriculture-and-food/farm-management/soil-and-water/irrigation/construction-of-an-evaporation-pan-for-irrigation-scheduling)

the base, a 100 mm PVC flange

- The pointer support, a 230 mm long piece of 100 mm PVC pipe. Four equally spaced 9 mm holes 70 mm from the base are drilled to allow the water height around the fixed pointer to quickly adjust to the water height in the pan. A single 15 mm long 5 mm wide elongated hole is also drilled 70 mm from the base of the PVC pipe.
- The pointer, a 170 mm long piece of 5 mm stainless steel rod bent at a right angle 60 mm from one end. From the shorter end a thread is tapped for about 15 mm and a point is ground on the other end of the rod.

After fitting the PVC pipe into the flange, the stainless steel rod is inserted into the elongated hole with nuts located on the inside and outside of the PVC pipe. To initially set the stainless steel rod in the correct position, the fixed pointer is placed in the pan and the pan is filled with water to a depth of 190 mm. The rod is then slid up or down in the 5 mm elongated hole so that the point of the rod just breaks the surface of the water.

Measuring Cylinder:

To measure evaporation the pan must be refilled with a known volume of water. The surface area of the pan is 1.14 square metres, so for every mm of evaporation 1.14 litres of water must be added to the pan. A transparent plastic 2 litre measuring jug with vertical sides is an excellent measuring cylinder if it is scaled properly. It is important that the jug actually holds more water than 2 litres so the sides of the jug must extend past the 2 litre mark. The jug is filled with 2.28 litres of water and the water level marked. This can conveniently be done by weighing the jug and adding 2.28 kilograms of water. For most jugs this will just about overflow, which is perfect.

A jug of water filled to the marker will be equivalent to 2 mm of evaporation. To scale the jug when less than 2 mm of water is required to fill the pan, the distance from the top marker to the bottom of the jug is measured and divided by 20. The numbers 0 to 2.0 in increments of 0.1 are then written with a permanent marking pen from the top marker to the bottom of the jug. These numbers are equivalent to the same number of mm of evaporation from the pan.

**Measurement:**

With evaporation the water level in the pan will fall. To measure the amount of evaporation, water is added to the pan with the measuring jug filled to the top mark. Water is added until the pointer just breaks the surface of the water. The PVC pipe supporting the pointer will help by reducing wave motion. It is important to keep track of the number of jugs used to refill the pan and the reading on the last jug when the pan water level is just broken by the pointer. The total amount of water added equals the amount of evaporation.

It is also essential to measure rainfall in conjunction with evaporation. Both measurements enable evaporation to be calculated on rainy days. After heavy rain the pan may have to be emptied to bring the water level down to the pointer. After rainfall on a hot summer day, less water may have to be removed than actually fell as rain. For example, after a 25 mm rainfall there might only be 12 mm of water removed from the pan with the measuring jug to bring the water level back to the pointer. The difference between the rainfall (25 mm) and the water removed from the pan (12 mm) is the evaporation. In this example it is 13 mm.

If the rain does not fill the pan above the pointer, the rainfall must still be added onto the measured evaporation to give the actual evaporation. For example, if there was 7 mm of rainfall and 6 mm of water was added to the pan with the measuring jug then the evaporation would be 13 mm.

Evaporation measurements should be routinely done every day at 9.00 am and clearly recorded. If measurements are not done routinely then the volume of water in the pan will decrease and take less time to heat up during the day and cool at night. This will induce an error which will become greater as the volume of water in the pan decreases. Evaporation measurements are very simple and take less than 5 minutes.

## 12.4 Determination of Evaporation from Water Surfaces

Evaporation from water surfaces can be determined by:

(1) Water budget

(2) Energy budget

(3) Mass transfer methods

(4) Combination methods

(5) Evaporation formulas

### Water Budget

The water-budget equation for estimating evaporation (Horton, 1943) can be written as:

$$E = I + P - O - O_s + \Delta S \quad (12.1)$$

Where,

E =Evaporation

I = Inflow

P = Precipitation

O = Outflow

$O_s$ = Seepage and

$\Delta S$ = Change in storage

Here, Inflow, outflow, precipitation, and change in storage can be measured reasonably accurately .Seepage, $O_s$, cannot be measured or evaluated directly and accurately, and the extent to which this quantity is accurate will affect the true value of evaporation.

The water-budget method of determining long-term evaporation can be used as a standard for comparing other methods. This method is not perfect, but it is satisfactory for practical purposes.

# Lesson 13 Evapotranspiration

## 13.1 Transpiration

Transpiration consists of the vaporization of liquid water contained in plant tissues and the vapour removal to the atmosphere. Crops predominately lose their water through stomata. These are small openings on the plant leaf through which gases and water vapour pass. The water, together with some nutrients, is taken up by the roots and transported through the plant. The vaporization occurs within the leaf, namely in the intercellular spaces, and the vapour exchange with the atmosphere is controlled by the stomatal aperture. Nearly all water taken up is lost by transpiration and only a tiny fraction is used within the plant.

Transpiration, like direct evaporation, depends on the energy supply, vapour pressure gradient and wind. Hence, radiation, air temperature, air humidity and wind terms should be considered when assessing transpiration. The soil water content and the ability of the soil to conduct water to the roots also determine the transpiration rate, as do water logging and soil water salinity. The transpiration rate is also influenced by crop characteristics, environmental aspects and cultivation practices. Different kinds of plants may have different transpiration rates. Not only the type of crop, but also the crop development, environment and management should be considered when assessing transpiration.

## 13.2 Evapotranspiration (ET)

Evaporation and transpiration occur simultaneously and there is no easy way of distinguishing between the two processes. Apart from the water availability in the topsoil, the evaporation from a cropped soil is mainly determined by the fraction of the solar radiation reaching the soil surface. This fraction decreases over the growing period as the crop develops and the crop canopy shades more and more of the ground area. When the crop is small, water is predominately lost by soil evaporation, but once the crop is well developed and completely covers the soil, transpiration becomes the main process. In Figure 1 the partitioning of evapotranspiration into evaporation and transpiration is plotted in correspondence to leaf area per unit surface of soil below it. At sowing nearly 100% of ET comes from evaporation, while at full crop cover more than 90% of ET comes from transpiration.

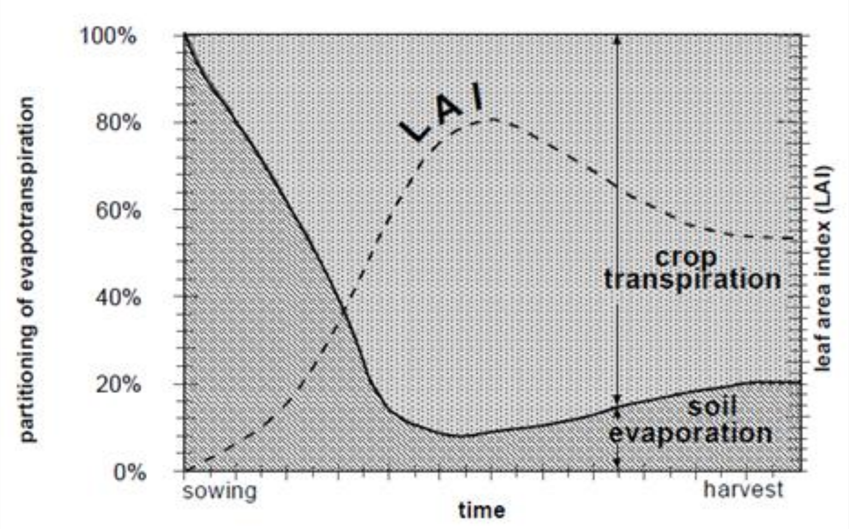


Fig.13.1. The partitioning of evapotranspiration into evaporation and transpiration over the growing period for an annual field crop (Richard,1998 )

The evapotranspiration rate is normally expressed in millimetres (mm) per unit time. The rate expresses the amount of water lost from a cropped surface in units of water depth. The time unit can be an hour, day, decade, month or even an entire growing period or year.

### 13.3 Measurement of Evapotranspiration

The principal methods for direct measurement of evapotranspiration are:

1) Lysimeter experiment

2) Field experimental plots

3) Soil moisture depletion studies

4) Water balance method

#### 13.3.1 Lysimeter

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

#### 13.3.2 Field Experimental Plots

Measurement of water supplies to the field and changes in soil moisture content of the field plots are sometime more dependable for computing seasonal water requirement of crops than measurement with lysimeters which do not simulate field conditions. The seasonal water requirements are computed by adding measured quantities of irrigation water, the effective rainfall received during the season and the contribution of moisture from the soil. Field water balance may be expressed by the following relationship:

$$\mathrm{WR} = \mathrm{IR} + \mathrm{ER} + \sum_{i=1}^{n} \frac{\mathrm{M_{bi}} - \mathrm{M_{ei}}}{100} \mathrm{A_i D_i} \qquad (13.1)$$

Where,WR is seasonal water requirement (mm), IR is total water applied (mm), ER is seasonal effective rainfall (mm), $M_{bi}$ and $M_{ei}$ are the moisture percentage at the beginning and end of the season in the $i_{th}$layer of soil, $A_i$ is the apparent specific gravity of the $i_{th}$layer of soil, $D_i$ is the depth of the $i_{th}$layer of soil within the root zone (mm) and n is the number of soil layer in the root zone D.

### 13.3.3 Soil Moisture Depletion Studies

The soil moisture depletion method is usually employed to determine the consumptive use of irrigated field crops grown on fairly uniform soils when the depth to the ground water is such that it will not influence the soil moisture fluctuation within the root zone.

$$u = \sum_{i=1}^{n} \frac{M_{1i} - M_{2i}}{100} \mathrm{A_i D_i} (13.2)$$

Where, u is the water use from the root zone for successive sampling periods or within one irrigation cycle (mm), n is the number of soil layers sampled in the root zone depth D, $M_{1i}$ and $M_{2i}$ are the soil moisture percentage at the time of the first and second sampling in the $i_{th}$ layer respectively, $A_i$is the apparent specific gravity of the $i_{th}$layer of soil and $D_i$ is the depth of the $i_{th}$layer of soil (mm).

Seasonal consumptive use (Cu = Σu) is calculated by assuming consumptive use values of each sampling interval. A correction is made by adding PET values for accelerated water loss for the intervals(s) just after irrigation and before soil moisture sampling.

### 13.3.4 Water Balance Method

Water balance method is also called the inflow-outflow method, is suitable for large areas (watersheds) over long period. It may be represented by the following hydrological equation;

Precipitation = Evapotranspiration + Surface Runoff + Sub-surface drainage + change in soil water content

This method necessitates adequate measurement of all factors, expect evapotranspiration. The value of evapotranspiration is computed from the measured data.

### 13.4 Determination of Evapotranspiration

Owing to the difficulty in obtaining accurate direct measurement of pan evaporation under field conditions, evaporation is often predicted on the basis of climatological data. The approaches followed are to relate the magnitude and variation of evapotranspiration to one or more climatic factors (temperature, day length, humidity, wind, sunshine etc.). The more commonly used empirical formulae in estimating evapotranspiration are:

a) Blaney-Criddle Method

b) Thornthwaite Method

c) Hargreaves' Method

The Blaney-Criddle method is recommended for periods of one month or longer.

### 13.4.1 Blaney-Criddle Method

This method requires the use of only two factors, temperature and information of day light hours which is a factor based purely on the latitude of the place. Using Blaney-Criddle approach, potential evapotranspiration can be expressed as follows, in metric unit:

$$\mathrm{PET} = 0.46\mathrm{p}(\mathrm{T} + 17.8) \qquad (13.3)$$

Where, PET = potential evapotranspiration, mm of water per day (mean value over the month)

P= monthly percent of total day time hours of the year

T= mean monthly temp. In °C (Average of daily max and minvalues)

The seasonal consumptive use of a crop can be determined from the following relationship,

$$\mathrm{U} = \mathrm{KF} = \sum \mathrm{kf} = \sum \mathrm{u} \qquad (13.4)$$

Where, U = seasonal consumptive use of water by the crop for a given period (mm/inches)

u = monthly consumptive use (mm)

K = empirical seasonal consumptive use consumptive us for the growing season

F = sum of the monthly consumptive use factors (f) for the growing season

k = empirical consumptive use crop coefficient for the month (u/f)

f = value of monthly PET in mm

### 13.4.2 Thornthwaite Method

Thornthwaite method is based on the assumption of an exponential relationship between mean monthly temperature and mean monthly consumptive use.

$$\mathrm{e} = 1.6(10\mathrm{t}/\mathrm{I})^{\mathrm{a}} \qquad (13.5)$$

Where, e = unadjusted PET (cm per month) t = mean air temperature (°C)

I = annual or seasonal heat index, the summation of 12 values of monthly heat indices (i) when, $i = (t / 5)^{1.514}$

$$a = 0.0000006751I^3 - 0.000071I^2 + 0.01792I + 0.49239$$

### 13.4.3 Hargreaves' Method

Hargreaves based on his work on data from grass lysimeter, proposed the following relationship to estimate ET,

$$PET = 0.0135(t + 17.78)R_s$$

Where, PET = reference crop potential consumptive use

t = mean daily temperature (°C)

$R_s$= incident solar radiation in langlay/day, it can be calculated using the following relationship,

$$R_s = 0.10\, R_{so}(S)^{1/2} \quad (13.7)$$

Where, S is the percent possible sunshine hour and $R_{so}$ is the clear daysolar radiation in langlay/day.

### 13.4.4 FAO Penman-Monteith Method

The FAO Penman-Monteith method is used to estimate reference evapotranspiration. The equation is:

$$ET_0 = \frac{0.408\Delta(R_n - G) + \gamma \frac{900}{T+273} u_2(e_s - e_a)}{\Delta + \gamma(1 + 0.34u_{2)}}$$

Where, $ET_0$ = reference evapotranspiration [mm $day^{-1}$]

$R_n$ = net radiation at the crop surface [MJ $m^{-2}$ $day^{-1}$]

T = mean daily air temperature at 2 m height [°C]

$u_2$ = wind speed at 2 m height [m/s]

$e_s$ = saturation vapour pressure [kPa]

$e_a$ = actual vapour pressure [kPa]

$e_s - e_a$ = saturation vapour pressure deficit [kPa]

Δ = slope of vapour pressure curve [kPa°$C^{-1}$]

Y = psychometric constant [$kPa°C^{-1}$]

The reference evapotranspiration, $ET_0$, provides a standard to which:

a) Evapotranspiration at different periods of the year or in other region can be compared.

b) Evapotranspiration of other crops can be related.

## Lesson 14 Infiltration

Water is constantly evaporated from the earth, and is precipitated back on the earth, mainly in the form of rainfall. One part of this rainfall sinks into the ground, forming groundwater reservoir; second major part flows as runoff in the form of rivers; and the rest is lost in evaporation and transpiration. The part of the rainfall which sinks into the ground is discussed in this chapter.

### 14.1 Infiltration Process

It is well-known that when water is applied to the surface of a soil, a part of it seeps into the soil. This movement of water through the soil surface is known as infiltration and plays a very significant role in the runoff process by affecting the timing, distribution and magnitude of the surface runoff. Further, infiltration is the primary step in the natural groundwater recharge.

Infiltration is the flow of water into the ground through the soil surface and the process can be easily understood through a simple analogy. Consider a small container covered with wire gauze, if water is poured over the gauze, a part of it will go to container and a part overflows. Further, the container can hold only a fixed quantity and when it is full no more flow into the container can take place. This analogy, though a highly simplified one, underscores two important aspects, viz., the maximum rate at which the ground can absorb water, the infiltration capacity and the volume of water that it can hold, the field capacity.

### 14.2 Factors Affecting Infiltration Rate

The major factors affecting the infiltration of water into the soil are,

1. Initial moisture content
2. Condition of the soil surface
3. Hydraulic conductivity of the soil profile
4. Texture
5. Porosity
6. Degree of swelling of soil colloids
7. Organic matter
8. Vegetative cover
9. Duration of irrigation or rainfall

10. Viscosity of water

The antecedent soil moisture content has considerable influence on the initial rate and total amount of infiltration, but decreasing as the soil moisture content rises. The infiltration rate of any soil is limited by any restraint to the flow of water into and through the soil profile. The soil layer with the lowest permeability, either at the surface or below it, usually determines the infiltration rate. Infiltration rates are also affected by the porosity of the soil which is changed by cultivation or compaction. Cultivation influences the infiltration rate by increasing the porosity of the surface soil and breaking up the surface seals. The effect of tillage on infiltration usually lasts only until the soil settles back to its former condition of bulk density because of subsequent irrigations. Infiltration rates are generally lower in soils of heavy texture than in soil of light texture. It has been established that in surface irrigation, increased depth increases initial infiltration slightly but the depth of application has negligible effect after prolonged irrigation. Infiltration rates are also influenced by the vegetal cover. Infiltration rates on grassland are subsequently higher than bare uncultivated land. Addition of organic matter increases infiltration rate substantially. The hydraulic conductivity of soil profile often change during infiltration, not only because of increasing moisture content, but also because of the puddling of the surface caused by reorientation of surface particles and washing of finer materials into the soil. Viscosity of water influences infiltration. The high rates of infiltration in the tropics under otherwise comparable soil conditions are due to the low viscosity of warm water.

### 14.3 Measurement of Infiltration

Information about the infiltration characteristics of the soil at a given location can be obtained by conducting controlled experiments on small areas. The experimental set-up is called an infiltrometer. There are two kinds of infiltrometers:

1. Flooding-type infiltrometer
2. Rainfall simulator

### 14.3.1 Flooding-Type lnfiltrometer

This is a simple instrument consisting essentially of a metal cylinder, 30 cm diameter and 60 cm long, open at both ends. This cylinder is driven into the ground to a depth of 50 cm (Fig.14.1). Water is poured into the top part to a depth of 5 cm and a pointer is set to mark the water level. As infiltration proceeds, the volume is made up by adding water from a burette to keep the water level at the tip of the pointer. Knowing the volume of water added at different time intervals, the plot of the infiltration capacity vs lime is obtained. The experiments are continued till a uniform rate of infiltration is obtained and this may take 2-3 h. The surface of the soil is usually protected by a perforated disk to prevent formation capacity vs time is obtained. The experiments are continued till a uniform rate of infiltration is obtained and this may take 2-3 h.

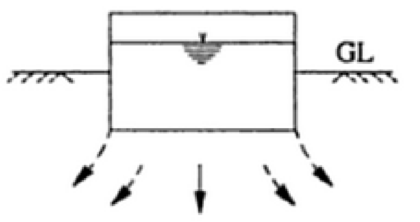


Fig. 14.1.Simple Infiltrometer.(Source:Subramanya, 2006)

The surface of the soil is usually protected by a perforated disk to preventformation of turbidity and it's settling on the soil surface.A major objection to the simple infiltrometer as above is that the infiltered waterspreads at the outlet from the tube (as shown by dotted lines in Fig. 14.1) and assuch the tube area is not representative of the area in which infiltration takesplace.

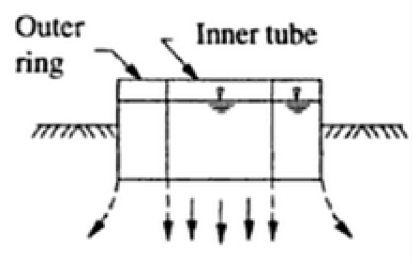


Fig. 14.2.Ring Infiltrometer.(Source:Subramanya, 2006)

To overcome this ring infiltrometer consisting of a set of two concentric rings (Fig. 14.2) is used. In this two rings are inserted into the ground and water is maintained on the soil surface, in both the rings, to a common fixed level. The outer ring provides a water jacket to the infiltering water of the inner ring and hence prevents the spreading out of the infiltering water of the inner tube. The measurement of water volume is done on the inner ring only.

The main disadvantages of flooding-type infiltrometer are:

(1) The raindrop-impact effect is not simulated;

(2) The driving of the tube or rings disturbs the soil structure;

(3) The results of the infiltrometer depend to some extent on their size with thelarger meters giving fewer rates than the smaller ones; this is due to the border effect.

### 14.3.2 Rainfall Simulator

In this a small plot of land, of about 2 m X 4 m size, is provided with a size of nozzles on the longer side with arrangements to collect and measure the surface runoff rate. The specially designed nozzles produce raindrops falling from a height of 2 m and are capable of producing various intensities of rainfall. Experiments are conducted under controlled conditions with various combinations of intensities and durations and the surface runoff is measured in each case. Using the water-budget equation involving the volume of rainfall, infiltration and runoff, the infiltration rate and its variation with time is calculated. If the rainfall intensity is higher than the infiltration rate, infiltration-capacity values are obtained.

Rainfall simulator type infiltrometers give lower values than flooding type infiltrometers. This is due to the effect of the rainfall impact and turbidity of the surface water present in the former.

### 14.4 Infiltration indices

Hydrological calculations involving floods it is found convenient to use a constant value of infiltration rate for the duration of the storm. The average infiltration rate is called infiltration index and two types of indices are in common use.

#### 14.4.1 Φ-index

The Φ index is the average rainfall above which the rainfall volume is equal tothe runoff volume. The Φ index is derived from the rainfall hyetograph with theknowledge of the resulting runoff volume. The initial loss is also considered as infiltration. The Φ value is found by treating it as a constant infiltration capacity. If the rainfall intensity is less than 0, then the infiltration rate is equal to the rainfall intensity; however, if the rainfall infiltration. TheΦ value is found by treating it intensity is larger than Φ the difference between rainfall and infiltration in an interval of time represents the runoff volume (Fig. 14.3).

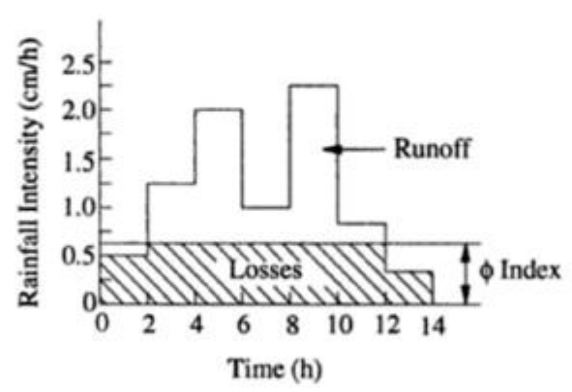


Fig. 14.3.Φ-index.(Source:Subramanya, 2006)

The amount of rainfall in excess of the Φ index is called rainfall excess. The Φ -index thus accounts for the total abstraction and enables runoff magnitudes to be estimated for a givenrainfall hyetograph.

#### 14.4.2 W- Index

In an attempt to refine the Φ-index the initial losses are separated from the total abstraction and an average value of infiltration rate called the W index is defined as

$$W = \frac{P - R - I_a}{t_e} \quad (14.1)$$

Where, P is total precipitation (cm), R is total storm runoff (cm), $I_a$ is initial losses (cm), $t_e$ is the duration of the rainfall excess, i.e. the total time in which the rainfall intensity is greater than W (in hours) and W is the average rate of infiltration (cm/h).

Since $I_a$ values are difficult to obtain, the accurate estimation of the W index is ratger difficult. The minimum value of the W index obtained under very wet soil conditions, representing the constant minimum rate of infiltration of the catchment, is known as $W_{min.}$ Both theW-index and Φ index vary from storm to storm.

## Lesson 15 Types of Watersheds

### 15.1 Definition of Drainage Basin

It is defined as, "any portion of the earth's surface within a physical boundary defined by topographic slopes that divert all runoff to the same drainage outlet."

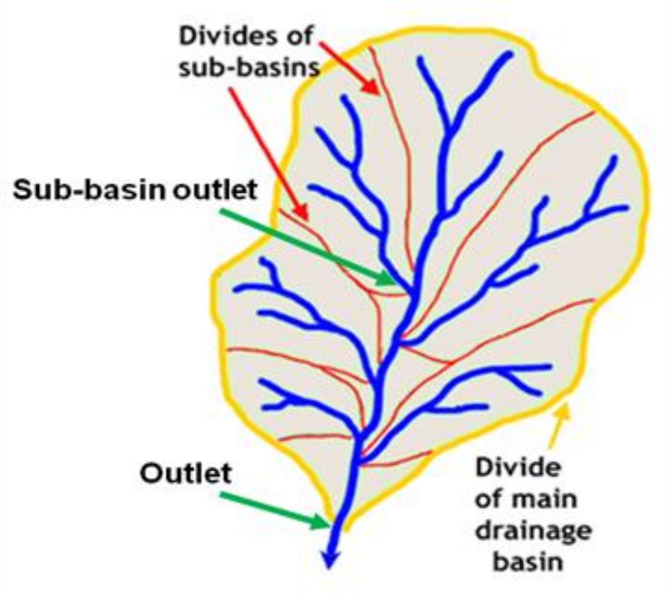


Fig.15.1.Drainage basin with its sub-basins.

(Source:http://nationalatlas.gov/articles/geology/a_continentalDiv.html)

This definition permits the selection of any drainage outlet desired. One can move the drainage outlet up the drainage system or down the drainage system to any location of interest (makes possible the sub-basin studies).

By definition, any point on the main drainage system can be selected as the basin outlet. Thus, a basin is defined with respect to the outlet.

The physical boundary of the drainage basin is called the drainage divide.The watershed area includes all the points that lie above the elevation of the outlet and within the drainage divide that separates adjacent watersheds.

Other terms synonymous with drainage basin are watershed, catchment, basin, river basin, runoff area, and stream basin. Watershed, catchment and basin are most commonly used terms by hydrologists.

Watersheds can be classified based on size, mean slope, length, land use, etc. Two hydrologically meaningful criteria are size and land use.

## 15.2 Classification of Watersheds by Size

Three types of watershed are distinguished according to size:

1. Small size: < 250 $km^2$
2. Medium size: between 250 $km^2$- 2500 $km^2$
3. Large: >250 $km^2$

This classification is vague, but the implication is in terms of spatial heterogeneity and dampening (averaging) of hydrological processes.

Runoff generation on these watersheds can be considered in two phases: i) land phase and ii) channel phase. Each phase has its own storage characteristics.

### 15.2.1 Large Watersheds

1) They have well-developed channel networks and channel phase, and, thus, channel storage is dominant.

2) They are less sensitive to high-intensity rainfalls of short duration.

### 15.2.2 Small Watersheds

1) They have dominant land phase and overland flow, have relatively less conspicuous channel phase.

2) They are highly sensitive to high-intensity, short-duration rainfalls.

Two watersheds of the same size may behave very differently if they do not have similar land and channel phases.

Small watersheds are usually least heterogeneous and large watersheds are most heterogeneous. In other words, spatial variability of watershed characteristics increases with size.

As the watershed size increases, storage increases and averaging of hydrologic processes increases as a result. The effect of averaging is to linearise the watershed behavior. On the average, small watersheds are more nonlinear than large watersheds.

Small watersheds are, within a given drainage system, represented by upland areas where rainfall and runoff depths are usually greater and an extensive, well-developed channel system is lacking. Lee (1980) has reported that flow rates per unit area, Q ($m^3$/s-ha, or depth/time), generally follow the relationship

$$\frac{Q}{A} = kA^{x-1} \quad (15.1)$$

Wherek is empirical constant, and $x < 1$ for peak flows (typically $x = 0.8$); $x > 1$ for low flows (typically $x = 1.2$), and x  1 for average discharge. This equation shows that as A increases, Q/A decreases at high flows; but at low flows, Q/A increases with area due to delayed subsurface flow.

Watersheds are also classified into different categories based on area that the watershed contains:

Fig. 15.1.Classification of watershed.(Source:Singh, 1994)

| Size (ha) | Classification |
|---|---|
| 50,000-2,00,000 | Watershed |
| 10,000-50,000 | Sub-watershed |
| 1,000-10,000 | Mili-watershed |
| 100-1,000 | Micro-watershed |
| 10-100 | Mini-watershed |

### 15.3 Classification of Watersheds by Land Use

Land use defines exploitation of watershed. Accordingly, watersheds can be classified as agricultural, urban, mountainous, forest, desert, coastal or marsh, or mixed - a combination of two or more of the previous classifications. These watersheds behave hydrologically so differently that different branches of hydrology have arisen:-

1) Urban watersheds: urban hydrology

2) Agricultural watersheds: agricultural hydrology

3) Forest watersheds: forest hydrology

4) Mountainous watersheds: mountain hydrology

5) Desert watersheds: desert hydrology

6) Coastal watersheds: coastal hydrology

7) Wetland/marsh watersheds: wetland hydrology

### 15.3.1 Urban Watersheds

An urban watershed is dominated by buildings, roads, streets, pavements, and parking lots. These features reduce the infiltrating land area and increase imperviousness. Because drainage systems are artificially built, the natural pattern of water flow is substantially altered. For a given rainfall event, interception and depression storage can be significant but infiltration is considerably reduced and so is the case with evaporation. As a result, there is pronounced increase in runoff and pronounced decrease in soil erosion. Thus, an urban watershed is more vulnerable to flooding if the drainage system is inadequate.

Depending upon the degree of urbanization, topography, and drainage facility, the production of runoff varies for different parts of watershed. If lakes, ponds and parks are numerous in the watershed, evaporation will be significant and may compensate for reduction in evaporation elsewhere due to impervious land surfaces. Once a watershed is urbanized, its land use is almost fixed and its hydrologic behaviour changes due to changes in precipitation.

If small urbanizing watershed is considered by itself, then runoff peak increases and its time of occurrence decreases with urbanization. This is because as development proceeds, there are more pavements, sidewalks, houses, parking lots, storm sewers, channels, etc., which all decrease infiltration and increase runoff. When an entire complex watershed considered, then all runoff peak may actually be reduced because its roads, bridges, tunnels, etc., can cause impoundments that dampen the runoff hydrograph.

### 15.3.2 Agricultural Watersheds

An agricultural watershed experiences perhaps the most dynamically significant land-use change. Changing land use and the treatment (cultivated land, fallow, row crop, small grain crop, rotation meadow, rotation, straight row, contoured, grass land, meadows, woods and forests, and gardens) usually lead to increased infiltration, increased erosion, and/or decreased runoff. Depression storage also is increased by agricultural operations. When the fields are barren, falling raindrops tend to compact the soil and infiltration is reduced. There is lesser development of streams in agricultural watersheds, for small channels formed by erosion and runoffs are obliterated by tillage operations. The soil texture is altered by regular application of organic and/or inorganic manure. This, in turn, leads to changed infiltration characteristics. Evapotranspiration constitutes the principal loss of water from agricultural point of view.

### 15.3.3 Forest Watersheds

The hydrological behavior of forest watersheds is quite different from that of agricultural or urban watersheds. Interception is significant, and evapotranspiration is a dominant component of the hydrologic cycle. In forest watersheds, the ground is usually littered with leaves, stems, branches, wood, etc. Consequently, when it rains, the water is held by the trees and the ground cover and has greater opportunity to infiltrate. The subsurface flow becomes dominant and there are times when there is little to no surface runoff. There is greater recharge of groundwater. Because forests resist flow of water, the peak discharge is reduced, although inundation of the ground may be increased. This reduces flooding and flood damage downstream. Due to reduced surface-potential, stream development is much less. Plants and trees provide good protective cover to soil from erosion.

### 15.3.4 Mountainous Watersheds

The landscape of these watersheds is predominantly mountainous. Because of higher altitudes, such watersheds receive considerable snowfall. By and large, such watersheds have substantial vegetation, such that in some cases, these could be considered as forest watersheds also. Interception is significant. Due to steep gradient and relatively less porous soil, infiltration is less and surface runoff is dominantly high for a given rainfall event. Flash floods are a common occurrence. The areas downstream of the mountains are vulnerable to flooding whenever there is a heavy rainfall in the mountains. Flooding in valleys downstream may be even more severe when there is rain in mountainous on the top of snow. There is little to virtually no change in land use. Erosion is minimal if the mountainous are rocky. Sliding and collapsing of slopes are not uncommon occurrences during periods of heavy precipitation. Due to snow melt, water yield is significant even during spring and summer, which can be used for water supply. Recharge of groundwater is small and evapotranspiration is considerable.

### 15.3.5 Desert Watersheds

There is little to virtually no vegetation in desert, watersheds. The soil is mostly sandy and little annual rainfall occurs. Sand dunes and sand mounds are formed by blowing winds. Stream development is minimal. Whenever there is little rainfall, most of it is absorbed by the porous soil, some of it evaporates, and the remaining runs off only to be soaked in during its journey. There is limited opportunity for groundwater recharge due to limited rainfall.

### 15.3.6 Costal Watersheds

The watersheds in coastal areas may partly be urban and are in dynamic contact with the sea. Their hydrology is considerably influenced by backwater from wave and tidal action. Usually, these watersheds receive high rainfall, mostly of cyclonic type, do not have channel control in flow, and are vulnerable to severe local flooding. Coastal erosion is a continuing problem due to tidal action, and land-use change is common. The water table is high, and saltwater intrusion threatens the health of coastal aquifers, which usually are a source of water supply. The land gradient is small, drainage is slow, and the soil along the coast has a considerable sand component.

### 15.3.7 Marsh, Old Wetland, Watersheds

Such lands are almost flat and are comprised of swamps, marshes, water courses, etc. They have rich wildlife and plenty of vegetation. Evaporation is dominant, for water is no limiting factor to satisfy evaporative demand. Rainfall is normally high and infiltration is minimal. Most of the rainfall becomes runoff, which discharges slowly for minimal land declivity. Erosion is also minimal, except along the coast. The flood hydrograph peaks gradually and lasts for a long time.

# Lesson 16 Physical Characteristics of Watersheds

## 16.1 Watershed Area

Basin area is defined as the area contained within the vertical projection of the drainage divide on a horizontal plane.

Watershed area is comprised of two subcomponents: 1) Stream areas 2) Inter-basin areas.

The inter-basin areas are the surface elements contributing flow directly to streams of order higher than 1.

Stream areas are those areas that would constitute the area draining to a predetermined point in the stream or outlet.

For example, the stream area for first-order streams would be delineated by measuring the drainage area for each first-order channel. This classification of areas is useful in geomorphologic modeling of watershed runoff.

Horton (1945) inferred that mean drainage areas of progressively higher orders might form a geometric sequence. This characteristic was formulated as a law of drainage areas by Schumm (1954), who stated that the mean drainage areas of streams of each order tend to approximate a geometric progression as

$$\overline{A}_w = \overline{A}_1 R_a^{w-1}$$

$$\log \overline{A}_w = \log \overline{A}_1 + (w-1)\log R_a$$

$$\log \overline{A}_w = \log\left(\frac{\overline{A}_1}{R_a}\right) + w \log R_a = a + wb$$

where $\overline{A}_w$ = mean area of basins of order w; $\overline{A}_1$ = mean area of first-order basins; and $R_a$= Stream Area ratio defined as $A_w/A_{w-1}$and normally varies from 3 to 6.Drainage area is highly correlated with several hydrologic parameters, e.g., watershed discharge Q (Leopold and Miller, 1956; Hack, 1957). One of the simplest form is

Q

$=KA^x$

Wherek and x = parameters

Equations (1) and (2) can be combined to obtain an average flow for a watershed of given order as

$$Q_w = Q_1 R_a^{x(w-1)}$$

Where$Q_w$ = average flow for $A_w$; and $Q_1$ = average flow for A1

### 16.2 Watershed Shape

Numerous symmetrical and irregular forms of drainage areas are encountered in practice. A frequently occurring shape is a pear shape in plan view, as shown in Fig.16.1a

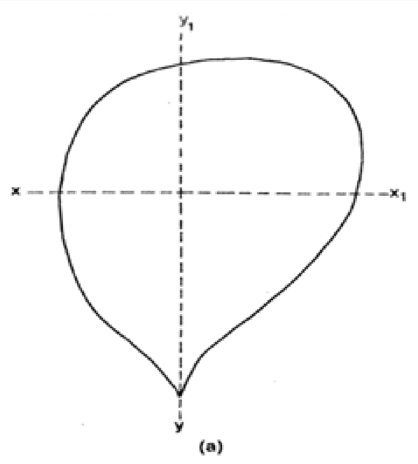


Fig. 16.1a.Typical pear shaped watershed.(Source: Singh, 1994)

The watershed surface, however, is always a tilted concavity that determines the general direction of flow. Depending upon the interaction of climate and geologic processes, the lateral section of a watershed may approximate a U shaped valley, Fig. 16.1b, or a V-shaped one, Fig. 16.1c.

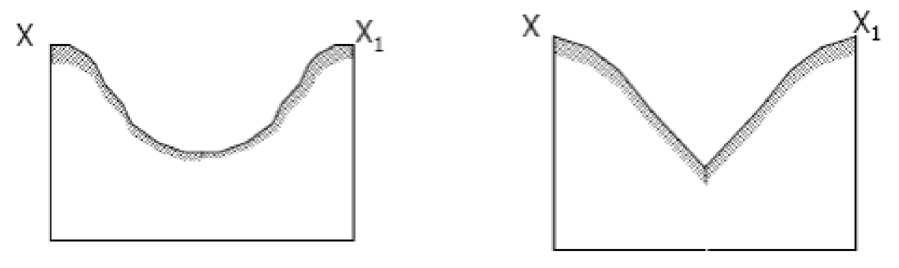


Fig. 16.1b.U-shaped valley.Fig. 16.1c.U-shapedvalley.

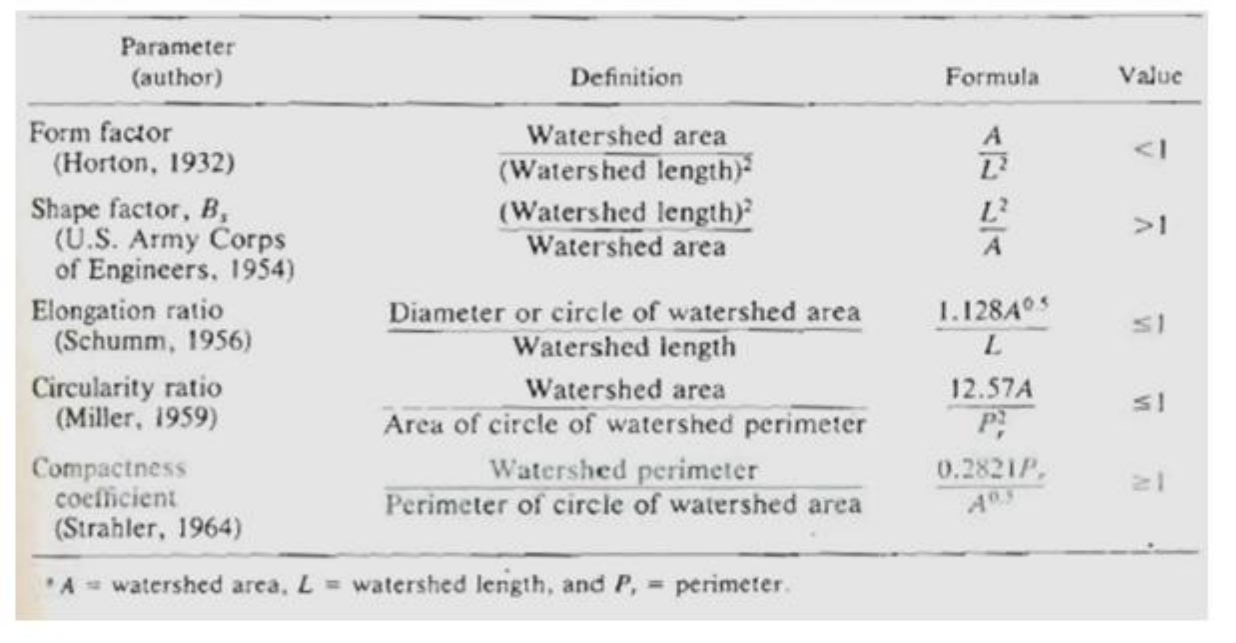
(Source: Singh, 1994)

The Transverse Section, Fig. 16.2d, displays increase in steepness towards the upstream area.

Fig. 16.1d.Cross-section along Y-$Y_1$.(Source: Singh, 1994)

A multitude of dimensionless parameters have been proposed to quantitatively define watershed shape. Some of the commonly used parameters are given in Table 16.1.

Table 16.1. Watershed shape parameters (Source: Singh, 1994)



| Parameter (author) | Definition | Formula | Value |
|---|---|---|---|
| Form factor (Horton, 1932) | $\frac{\text{Watershed area}}{(\text{Watershed length})^2}$ | $\frac{A}{L^2}$ | <1 |
| Shape factor, $B_s$ (U.S. Army Corps of Engineers, 1954) | $\frac{(\text{Watershed length})^2}{\text{Watershed area}}$ | $\frac{L^2}{A}$ | >1 |
| Elongation ratio (Schumm, 1956) | $\frac{\text{Diameter or circle of watershed area}}{\text{Watershed length}}$ | $\frac{1.128A^{0.5}}{L}$ | ≤1 |
| Circularity ratio (Miller, 1959) | $\frac{\text{Watershed area}}{\text{Area of circle of watershed perimeter}}$ | $\frac{12.57A}{P_r^2}$ | ≤1 |
| Compactness coefficient (Strahler, 1964) | $\frac{\text{Watershed perimeter}}{\text{Perimeter of circle of watershed area}}$ | $\frac{0.2821P_r}{A^{0.5}}$ | ≥1 |

* $A$ = watershed area, $L$ = watershed length, and $P_r$ = perimeter.

These factors involve watershed length L, area, and/or perimeter $P_r$.

Length is defined in more than one way: (1) The greatest straight-line distance between any two points on the perimeter, (2) The greatest distance between the outlet and any point on the perimeter, (3) The length of the main stream from its source (projected to the perimeter) to the outlet.

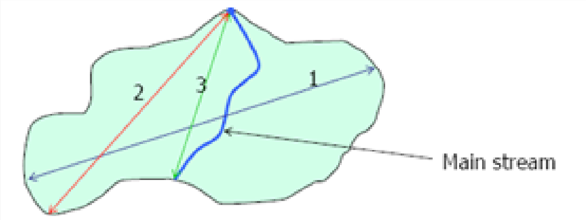


Fig. 16.2.Watershed Length Measurement.(Source: Singh, 1994)

Watershed shape influences the hydrograph shape, especially for small watersheds. If a watershed is long and narrow, then it will take longer for water to travel from watershed extremities to the outlet and the resulting runoff hydrograph will be flatter (Fig. 16.3a).

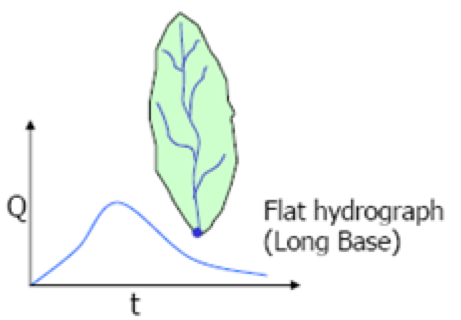


Fig. 16.3a.(Source: Singh, 1994)

For more compact watersheds, the runoff hydrograph is expected to be sharper with a greater peak and shorter duration (Fig. 16.3b).

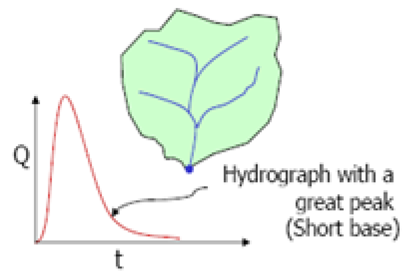


Fig. 16.3b.(Source: Singh, 1994)

For a watershed that is partly long and narrow, and partly compact, the runoff hydrograph is expected to be a complex composite of the aforementioned hydrographs (Fig. 16.3c).

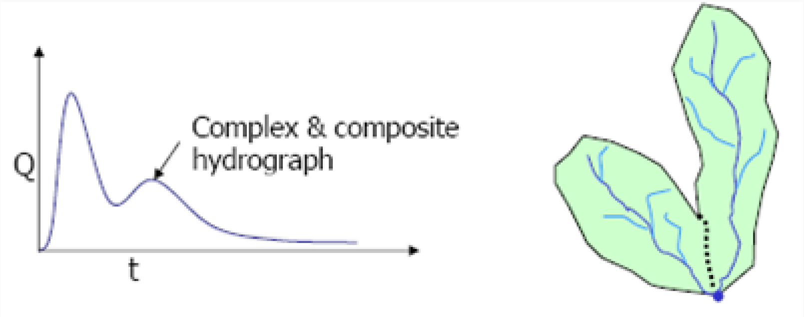


Fig. 16.3c.(Source: Singh, 1994)

**16.3 Watershed slope**

Basin slope has a profound effect on the velocity of overland flow, watershederosion potential, and local windsystems.

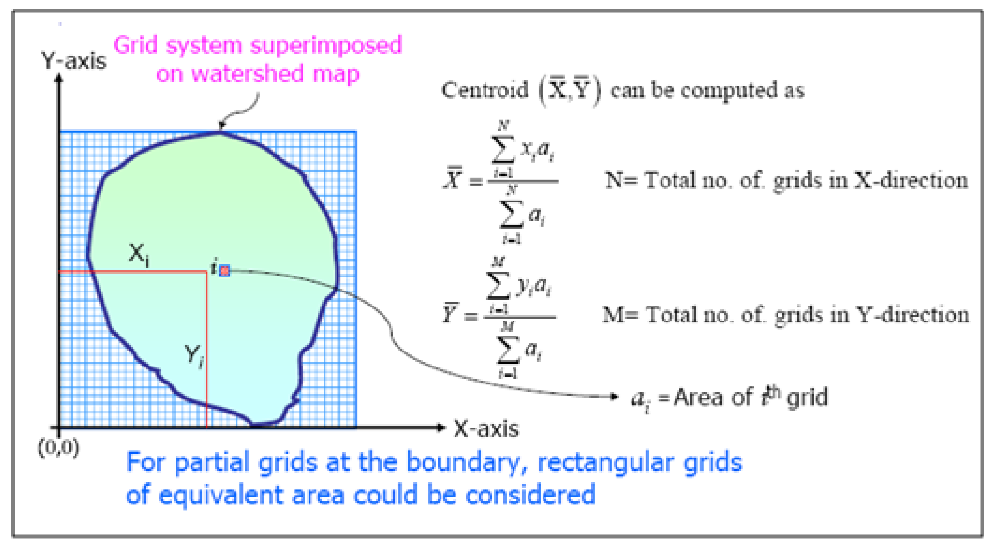
Basin slope S is defined as,

$$S = \frac{h}{L} \quad (16.4)$$

Whereh = fall in meters, and L = horizontal distance (length) over which the fall occurs.

## 16.4 Watershed Centroid

The centroid of a drainage basin is simply the location of the point within the drainage basin that represents the weighted centre of the basin. It is the first moment of the area about the origin. One method of locating the centroid, in the absence of necessary mathematical equations, is to use a cutout tracing of the drainage basin. Trial-and-error balancing of this tracing on the point of a pencil will locate the centroid sufficiently well for hydrologic uses. Another method that is used is to cut the basin map into equal grid sizes and equalize the weights of the cutout grids on a sensitive chemical balance. The centroid of a drainage basin can be determined more accurately by using the method of moments and representing the basin by a grid system.



## 16.5 Watershed Length

Basin length, Lb, is the longest dimension of a basin parallel to its principal drainage channel. Basin width can be measured in a direction approximately perpendicular to the length measurement. The relation between mainstream length and drainage-basin area for small watershed is given as (Gray, 1961).

Where$L_b$ is in km and A in km$^2$

# Lesson 17 Channel Characteristics of Watersheds

## 17.1 Number of Channels and their Order

Identification of first-order channels is the first step in applying channel order in hydrology. Application of stream ordering requires the use of a topographic map. Fig 16.5 shows the Strahler method of channel ordering. The number of stream channels of each order is expressed by a mathematical relation, known as Horton's law of channel numbers, in which the number of stream channels of each order forms an inverse geometric sequence with order number,

$$N_w = R_b^{W-w} \qquad (17.1)$$

Or

$$\log N_w = W \log R_b - w \log R_b = a - bw \qquad (17.2)$$

$$a = W \log R_b, \quad b = \log R_b \qquad (17.3)$$

Where $N_w$ = number of streams of order w; W = order of the watershed; and $R_b$ = Bifurcation Ratio, defined as $R_b = N_w/N_{w+1}$, and varies between 3 and 5. This law is an expression of topological phenomenon, and is a measure of drainage efficiency.

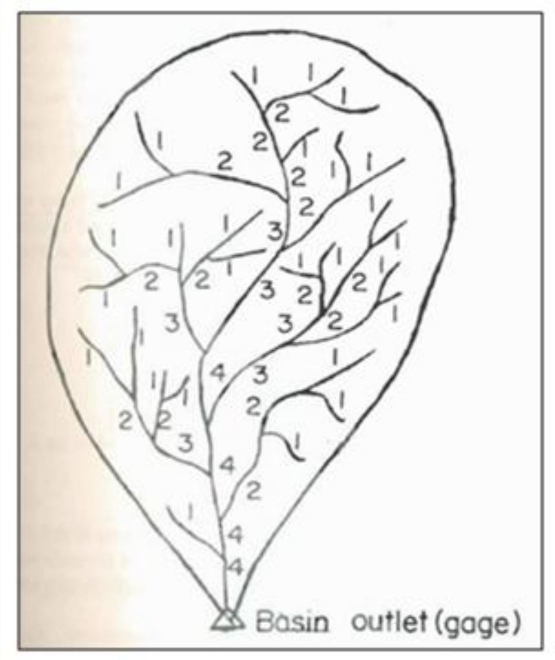


Fig. 17.1. Channel ordering by the Strahler method.(Source: Singh, 1994)

The number of channels of a given order in a drainage basin is a function of the nature of the surface of that drainage basin. In general, the greater the infiltration of the soil material covering the basin, the fewer will be the number of channels required to carry the remaining runoff water. Moreover, the large number of channels of a given order, the smaller is the area drained by each channel order.

## 17.2 Channel Length

This refers to the length of channels of each order. The average length of channels of each higher order increases as a geometric sequence. Thus, the first-order channels are the shortest of all the channels and the length increases geometrically as the order increases. This relation is called Horton's law of channel lengths and can be formulated as

$$\overline{L}_w = \overline{L}_1 R_L^{w-1} \quad (17.4)$$

$$\overline{L}_w = \frac{L_w}{\overline{N}_w} \quad (17.5)$$

whereL$_w$= total length of all channels of order w; N$_w$ = number of channels of order w; = mean channel length of order w; = mean length of the first-order streams; and R$_L$ = Stream-Length Ratio, defined as R$_L$ = L$_w$/L$_{w-1}$, and is generally varies between 1.5 and 3.5.

## 17.3 Channel Area

The channel area of order w, A$_w$, is the area of the watershed that contributes to the channel segment of order w and all lower-order channels. Hack (1957) derived an expression for A$_w$ based on  , R$_b$, and r, which can be written as

$$A_w = \overline{A}_1 R_a^{w-1} \frac{r^w - 1}{r - 1}, \qquad r = \frac{R_L}{R_b} \quad (17.6)$$

The ratios R$_b$,R$_L$, and R$_a$, are of fundamental significance not only for describing drainage-basin composition, but also for hydrologic synthesis.

## 17.4 Channel Profile

The longitudinal channel profile represents the relationship between altitude and horizontal distance, and can be determined from a watershed topographic map. Data on channel profile are used to compute the slope of the channel of a given order. Standard curve-fitting techniques can be used to represent the channel profile mathematically (Strahler, 1964).

The channel profile is normally found to be concave upward. The nature of concavity is a function of the basin geology and precipitation (Hack, 1957). Generally, the upper part of the drainage basin is fairly uniform. Where non-uniform geology occurs in a drainage basin, rocks more resistant to erosion distort the uniform nature of the concave profile. The result of changes in the stream profile caused by bedrock geology can cause large variations in the flow velocity of a stream channel.

## 17.5 Channel Slope

Channel slope has a profound effect on the velocity of flow in a channel, and, consequently, on the flow characteristics of runoff from a drainage basin. The importance of channel slope lies in two areas: (1) determining discharge and velocity using the Manning or Chezy equation, and (2) as a variable in multivariate analysis to determine the amount of influence accounted for by channel slope.

### Law of Stream Slopes

Horton (1945) introduced the law of stream slopes, which states that the average slope of streams of each order tends to approximate an inverse geometric series.

$$\bar{S}_w = \bar{S}_1 R_s^{W-w} \quad (17.7)$$

Where $\bar{S}_w$ is the average slope of streams of order w; $\bar{S}_1$ is the average slope of first-order streams; $R_s$ is the slope ratio defined as $R_s = \bar{S}_w / \bar{S}_{w-1}$; and W is the order of the basin. The value of $R_s$ is approximately 0.55.

## 17.6 Drainage Density

It is defined as the length of drainage per unit area. This term was first introduced by Horton (1932) and is expressed as

$$D_d = \frac{L}{A}$$

Or

$$D_d = \frac{\sum_{w=1}^{W} \sum_{i=1}^{N_w} L_{wi}}{A} \quad (17.8)$$

Where $D_d$ is the drainage density of a watershed of order W in kilometers per square kilometer, L is the total length of all channels of all orders in the drainage basin, and A is the area of the drainage basin in square kilometers. $D_d$ is a measure of the closeness (density) of channel spacing. It is an indication of the drainage efficiency of overland flow and the length of overland flow as well as the index of relative proportions.

### Constant of Channel Maintenance

Schumm (1956) defined constant of channel maintenance as the inverse of the drainage density. Its value increases with the size of watershed. It provides an estimate of the area in square meters of watershed required to maintain a meter of channel.

### Stream Frequency

The number of stream channels per unit area is called the stream frequency.

### 17.7 Length of Overland Flow

As rain falls on a drainage-basin surface, it flows down slope toward a channel. The maximum length of this surface flow is called the length of overland flow. Horton (1945) recommended using one-half the reciprocal of the drainage density to determine the average of overland flow for the entire drainage basin. Because the drainage density is a function of infiltration characteristics of the drainage basin, so, too, must be length of overland flow.

Horton (1945) expressed the length of overland flow $L_o$ as

$$L_o = \frac{1}{2D_d} \quad (17.9)$$

## Lesson 18 Components of Runoff and Factors Affecting Runoff

### 18.1 Components of Runoff

Runoff means the draining or flowing off of precipitation from a catchment area through a surface channel enters into a stream channel. It represents the output from catchment in a given unit of time. Fig. 18.1 shows components of runoff.

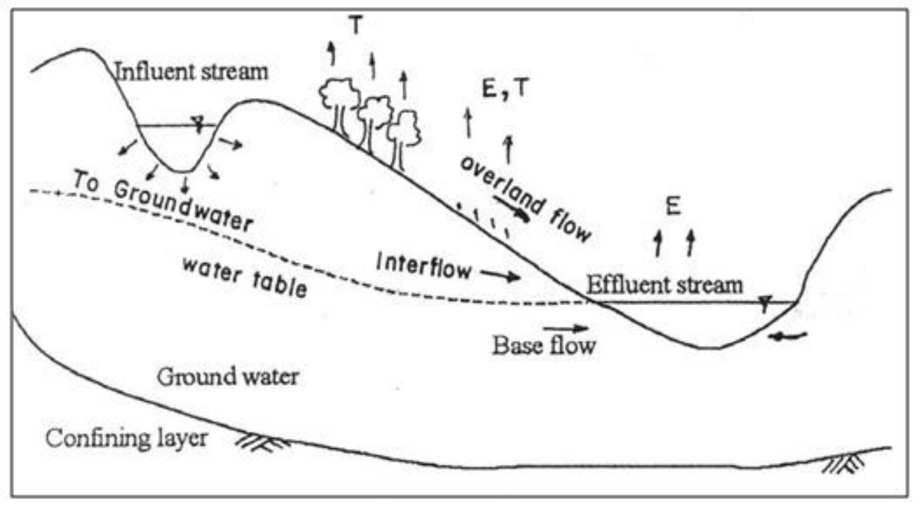


Fig. 18.1.Components of runoff.(Source: Subramanya, 2008)

Consider a catchment area receiving precipitation. For a given precipitation, when the evapotranspiration, initial loss, infiltration and detention storage requirements are satisfied, the excess precipitation moves over the land surfaces to reach smaller channels. This portion of runoff is called overland flow and involves building up of storage over the surface and draining off the same.Flows from several small channels join bigger channels and flows from these in turn combine to form a larger stream, and so on, till the flow reaches the catchment outlet. The flow in this mode, where it travels all the time over the surface as overland flow and through the channels as open-channel flow and reaches the catchment outlet is called surface runoff.

A part of the precipitation that infilters moves laterally through upper crusts of the soil and returns to the surface at some locations away from the point of entry into the soil. This component of runoff is known variously as interflow, through flow, storm seepage, subsurface flow or quick return flow.

Depending upon the time delay between the infiltration and the outflow, the interflow is sometimes classified into prompt interflow, i.e. the interflow with the least time lag and delayed interflow.

Another route for the infiltered water is to undergo deep percolation and reach the groundwater storage. The time lag, i.e. the difference in time between the entry into the soil and outflows from it is very large, being of the order of months and years. This part of runoff is called groundwater runoff or groundwater flow.

Based on the time delay between the precipitation and the runoff, the runoff is classified into two categories; as (a) Direct runoff (b) Base flow.

a) Direct runoff

It is the part of runoff which enters the stream immediately after the rainfall. It includes surface runoff, prompt interflow and rainfall on the surface of the stream. In the case of snow-melt, the resulting flow entering the stream is also a direct runoff. Direct storm runoff and storm runoff are also used to designate direct runoff.

b) Base flow

The delayed flow that reaches a stream essentially as groundwater flow is called base flow.

### 18.2 Factors Affecting Runoff

The main factors affecting the runoff from a catchment area are:

a) Precipitation characteristics

b) Shape and size of catchment

c) Topography

d) Geologic characteristics

e) Meteorological characteristics

f) Storage characteristics of a catchment

#### 18.2.1Precipitation Characteristics

Precipitation is the most important factor, which affects runoff. The important characteristics of precipitation are duration, intensity and areal distribution.

**Duration** Total runoff depends on the duration of rainstorm. For a given rainfall intensity and other conditions, a longer duration rainfall event will result in more runoff.

**Intensity** Rainfall intensity influences both rate and volume of runoff. The runoff volume and also runoff rate will be greater for an intense rainfall event than for less intense event.

**Areal Distribution** It also influences both the rate and volume of runoff. Generally, the maximum rate and volume of runoff occurs when the entire watershed contributes.

#### 18.2.2 Shape and Size of Catchment

The runoff from a catchment depends upon the size, shape and location of the catchment. The following are the general observations:

a) More intense rainfall events are generally distributed over a relatively smaller area, i.e., larger the area lower will be the intensity of rainfall.

b) The peak normally decreases as the area of the basin increase. (peak flow per unit area)

c) Larger basins give a more constant minimum flow than the smaller ones. (effect of local rains and greater capacity of the ground-water reservoir)

d) Fan shaped catchments give greater runoff because tributaries are nearly of same size and hence time of concentration of runoff is nearly same. On the contrary, discharges over fern leaf arrangement of tributaries are distributed over long period because of the different lengths of tributaries.

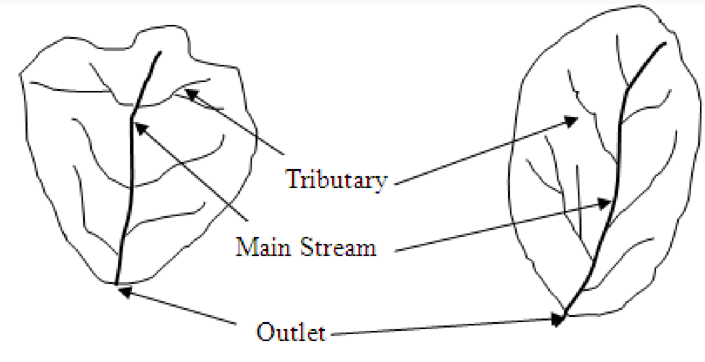


Fig. 18.2a.Fan shaped catchment. Fig. 18.2b.Leaf shaped catchment.(Source: Subramanya, 2008)

### 18.2.3 Topography

The runoff depends upon surface condition, slope and land features. Runoff will be more from a smooth surface than from rugged surface. Also, if the surface slope is steep, water will flow quickly and adsorption and evaporation losses will be less, resulting in greater runoff. On the other hand if the catchment is mountainous, the rainfall intensity will be high and hence runoff will be more.

### 18.2.4 Geologic Characteristics

Geologic characteristics include surface and sub-surface soil type, rocks and their permeability. Geologic characteristics influence infiltration and percolation rates. The runoff will be more for low infiltration capacity soil (clay) than for high infiltration capacity soil (sand).

### 18.2.5 Meteorological Characteristics

Temperature, wind speed, and humidity are the major meteorological factors, which affect runoff. Temperature, wind speed and humidity affect evaporation and transpiration rates, thus soil moisture regime and infiltration rate, and finally runoff volume.

### 18.2.6 Storage Characteristics of a Catchment

Presence of artificial storage such as dams, weirs etc. and natural storage such as lakes and ponds etc. tend to reduce the peak flow. These structures also give rise to greater evaporation.

# Lesson 19 Estimation of Runoff-I

## 19.1 Empirical Formulae

In the past, many empirical formulae have been developed, but these are applicable only to the region where they were derived. Furthermore, caution must be taken in their application if the characteristics of the region have been subjected to manmade disturbance (settlement, land use pattern change). These are essentially rainfall-runoff relationships with additional third or fourth parameters to account for climatic or catchment characteristics. Some of the important empirical runoff estimation formulae used in various parts of India are given below:

### 19.1.1 Binnie's Percentages

Sir Alexander Binnie measured the runoff from a small catchment near Nagpur (Area of 16 $km^2$) during 1869 to 1872 and developed curves of cumulative runoff against cumulative rainfall. The two curves were found to be similar. From these, he established percentage of runoff from rainfall. These percentages have been used in Madhya Pradesh and Vidarbha region of Maharashtra for the estimation of yield.

### 19.1.2 Barlow's Tables

Barlow, the first Chief Engineer of the Hydro-Electric Survey of India (1915) on the basis of his study in small catchments (area ~ 130 $km^2$) in Uttar Pradesh expressed runoff R as

$$R = K_b P \quad (19.1)$$

Where $K_b$ is the runoff coefficient which depends upon the type of catchment and nature of monsoon and P is the rainfall.

Table 19.1. –Barlow's Runoff Coefficients, $K_b$ in percentage

(Developed for use in UP) (Source: Subramanya, 2008)

| Class | Description of catchment | $K_b$ (percentage) | | |
|---|---|---|---|---|
| | | Season I | Season II | Season III |
| A | Flat, cultivated, and absorbent soil | 7 | 10 | 15 |
| B | Flat, partly cultivated, and stiff soil | 12 | 15 | 18 |

| | | | | |
|---|---|---|---|---|
| C | Average catchment | 16 | 20 | 32 |
| D | Hills and plains with little cultivation | 28 | 35 | 60 |
| E | Very hilly, steep and no cultivation | 36 | 45 | 81 |
| Season I: Light rain, no heavy downpour<br>Season II: Average or varying rainfall, no continuous downpour<br>Season III: Continuous downpours | | | | |

### 19.1.3 Strange's Tables

Strange (1892) studied the available rainfall and runoff in the border areas of present-day Maharashtra and Karnataka and has obtained yield ratios as functions of indicators representing catchment characteristics. Catchments lie classified as good, average and bad according to the relative magnitudes of yield they give. Two methods using tables for estimating the runoff volume in a season are given.

(I) Runoff Volume from Total Monsoon Season Rainfall

A table giving the runoff volumes for the monsoon period (i.e. yield during monsoon season) for different total monsoon rainfall values and for the three classes of catchments (viz. good, average and bad) is given in Table 19.2a. The correlation equations of best fitt.ing lines relating percentage yield ratio (Y)to precipitation (P)could be expressed as

Table 19.2a.Strange's Table of Total Monsoon Rainfall and estimated Runoff(Source: Subramanya, 2008)

| Total Monsoon rainfall (inches) | Total Monsoon rainfall (mm) | Percentage of Runoff to rainfall | | | Total Monsoon rainfall (inches) | Total Monsoon rainfall (mm) | Percentage of Runoff to rainfall | | |
|---|---|---|---|---|---|---|---|---|---|
| | | Good catchment | Average catchment | Bad catchment | | | Good catchment | Average catchment | Bad catchment |
| 1.0 | 25.4 | 0.1 | 0.1 | 0.1 | 31.0 | 787.4 | 27.4 | 20.5 | 13.7 |
| 2.0 | 50.8 | 0.2 | 0.2 | 0.1 | 32.0 | 812.8 | 28.5 | 21.3 | 14.2 |
| 3.0 | 76.2 | 0.4 | 0.3 | 0.2 | 33.0 | 838.2 | 29.6 | 22.2 | 14.8 |
| 4.0 | 101.6 | 0.7 | 0.5 | 0.3 | 34.0 | 863.6 | 30.8 | 23.1 | 15.4 |
| 5.0 | 127.0 | 1.0 | 0.7 | 0.5 | 35.0 | 889.0 | 31.9 | 23.9 | 15.9 |
| 6.0 | 152.4 | 1.5 | 1.1 | 0.7 | 36.0 | 914.4 | 33.0 | 24.7 | 16.5 |
| 7.0 | 177.8 | 2.1 | 1.5 | 1.0 | 37.0 | 939.8 | 34.1 | 25.5 | 17.0 |
| 8.0 | 203.2 | 2.8 | 2.1 | 1.4 | 38.0 | 965.2 | 35.3 | 26.4 | 17.6 |
| 9.0 | 228.6 | 3.5 | 2.6 | 1.7 | 39.0 | 990.6 | 36.4 | 27.3 | 18.2 |
| 10.0 | 254.0 | 4.3 | 3.2 | 2.1 | 40.0 | 1016.0 | 37.5 | 28.1 | 18.7 |
| 11.0 | 279.4 | 5.2 | 3.9 | 2.6 | 41.0 | 1 041.4 | 38.6 | 28.9 | 19.3 |
| 12.0 | 304.8 | 6.2 | 4.6 | 3.1 | 42.0 | 1066.8 | 39.8 | 29.8 | 19.9 |
| 13.0 | 330.2 | 7.2 | 5.4 | 3.6 | 43.0 | 1092.2 | 40.9 | 30.6 | 20.4 |
| 14.0 | 355.6 | 8.3 | 6.2 | 4.1 | 44.0 | 1117.6 | 42.0 | 31.5 | 21.0 |
| 15.0 | 381.0 | 9.4 | 7.0 | 4.7 | 45.0 | 1143.0 | 43.1 | 32.3 | 21.5 |
| 16.0 | 406.4 | 10.5 | 7.8 | 5.2 | 46.0 | 1168.4 | 44.3 | 33.2 | 22.1 |
| 17.0 | 431.8 | 11.6 | 8.7 | 5.8 | 47.0 | 1193.8 | 45.4 | 34.0 | 22.7 |
| 18.0 | 457.2 | 12.8 | 9.6 | 6.4 | 48.0 | 1219.2 | 46.5 | 34.8 | 23.2 |
| 19.0 | 482.6 | 13.9 | 10.4 | 6.9 | 49.0 | 1244.6 | 47.6 | 35.7 | 23.8 |
| 20.0 | 508.0 | 15.0 | 11.3 | 7.5 | 50.0 | 1270.0 | 48.8 | 36.6 | 24.4 |
| 21.0 | 533.4 | 16.1 | 12.0 | 8.0 | 51.0 | 1295.4 | 49.9 | 37.4 | 24.9 |
| 22.0 | 558.8 | 17.3 | 12.9 | 8.6 | 52.0 | 1320.8 | 51.0 | 38.2 | 25.5 |
| 23.0 | 584.2 | 18.4 | 13.8 | 9.2 | 53.0 | 1346.2 | 52.1 | 39.0 | 26.0 |
| 24.0 | 609.6 | 19.5 | 14.6 | 9.7 | 54.0 | 1371.6 | 53.3 | 39.9 | 26.6 |
| 25.0 | 635.0 | 20.6 | 15.4 | 10.3 | 55.0 | 1397.0 | 54.4 | 40.8 | 27.2 |
| 26.0 | 660.4 | 21.8 | 16.3 | 10.9 | 56.0 | 1422.4 | 55.5 | 41.6 | 27.7 |
| 27.0 | 685.8 | 22.9 | 17.1 | 11.4 | 57.0 | 1447.8 | 56.6 | 42.4 | 28.3 |
| 28.0 | 711.2 | 24.0 | 18.0 | 12.0 | 58.0 | 1473.2 | 57.8 | 43.3 | 28.9 |
| 29.0 | 736.6 | 25.1 | 18.8 | 12.5 | 59.0 | 1498.6 | 58.9 | 44.4 | 29.41 |
| 30.0 | 762.0 | 26.3 | 19.7 | 13.1 | 60.0 | 1524.0 | 60.0 | 45.0 | 30.0 |

Since there is no appreciable runoff due to the rains in the dry (non-monsoon) period, the monsoon season runoff volume is recommended to be taken as annual yield of the catchment. This table could be used to estimate the monthly yields also in the monsoon season. However, it is to be used with the understanding that the table indicates relationship between cumulative monthly rainfall starting at the beginning of the season and cumulative runoff, i.e. a double mass curve relationship.

(II) Estimating the Runoff Volume from Daily Rainfall

In this method Strange in a most intuitive way recognizes the role of antecedent moisture in modifying the runoff volume due to a rainfall event in a given catchment. Daily rainfall event are considered and three states of antecedent moisture conditions prior to the rainfall event as dry, damp and wet are recognized. The classification of these three states is as follows:

(1) Wetting Process

(a) Transition from Dry to Damp

(i) 6 mm rainfall in the last 1 day (iii) 25 mm in the last 7 days

(ii) 12 mm in the last 3 days (iv) 38 mm in the last 10 days

(b) Transition from Damp to Wet

(i) 8 mm rainfall in the last I day (iii) 25 mm in the last 3 days

(ii) 12 mm in the last 2 days (iv) 38 mm in the last 5 days

(c) Direct Transition from Dry to Wet

Whenever 64 mm rain falls on the previous day or on the same day.

(2) Drying Process

(d) Transition from Wet to Damp

(i) 4 mm rainfall in the last 1 day (iii) 12 mm in the last 4 days

(ii) 6 mm in the last 2 days (iv) 20 mm in the last 5 days

(e) Transition from Damp to Dry

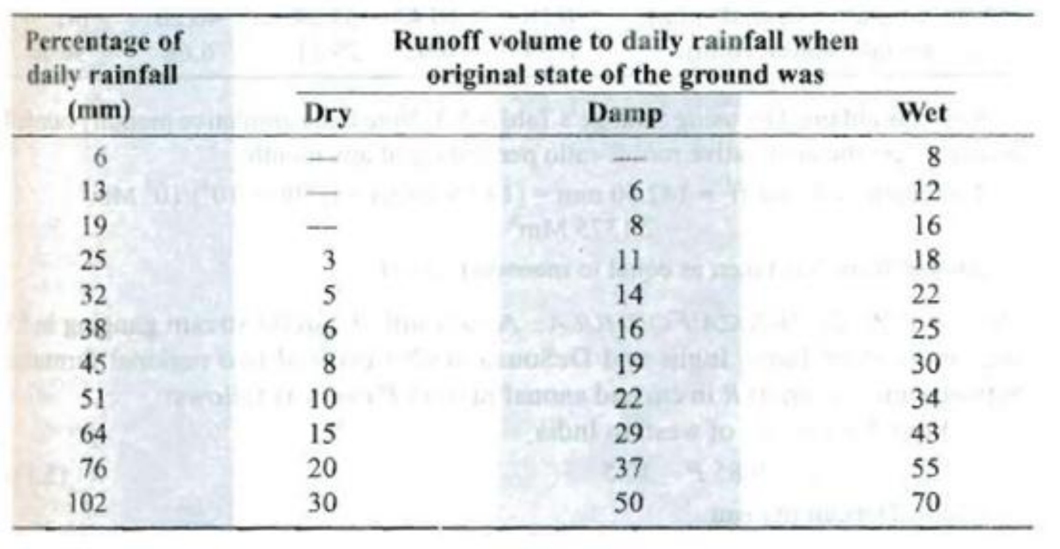
(i) 3 mm rainfall in the last I day (iii) 12 mm in the last 3 days

(ii) 6 mm in the last 3 days (iv) 15 mm in the last 10 days

The percentage daily rainfall that will result in runoff for average (yield producing) catchment is given in Table 19.2(b). For good (yield producing) and bad (yield producing) catchments add or deduct 25% of the yield corresponding to the average catchment.

Table 19.2b.Strange's Table of Runoff Volume from Daily Rainfall

For an Average Catchment(Source: Subramanya, 2008)



| Percentage of daily rainfall (mm) | Runoff volume to daily rainfall when original state of the ground was | | |
|---|---|---|---|
| | Dry | Damp | Wet |
| 6 | — | — | 8 |
| 13 | — | 6 | 12 |
| 19 | — | 8 | 16 |
| 25 | 3 | 11 | 18 |
| 32 | 5 | 14 | 22 |
| 38 | 6 | 16 | 25 |
| 45 | 8 | 19 | 30 |
| 51 | 10 | 22 | 34 |
| 64 | 15 | 29 | 43 |
| 76 | 20 | 37 | 55 |
| 102 | 30 | 50 | 70 |

Use of Strange's Tables

Strange's monsoon rainfall-runoff table (Table 19.2a) and (Table 19.2b) for estimating daily runoff corresponding to a daily rainfall event are in use in parts of Karnataka, Andhra Pradesh and Tamil Nadu.

### 19.1.4 Inglis and Desouza Formula

As a result of careful stream gauging in 53 sites in Western India, Inglis and DeSouza (1929) evolved two regional formulae, between annual runoff R in cm and annual rainfall P in cm as follows:

1.For Ghat regions of western India

$$R = 0.85P - 30.5 \quad (19.2)$$

2.For Deccan plateau

$$R = \frac{1}{254} P\,(P - 17.8) \quad (19.3)$$

### 19.1.5 Khosla’s Formula

Khosla (1960) analyzed the rainfall, runoff and temperature data for various catchments in India and USA to arrive at an empirical relationship between runoff and rainfall. The time period is taken as a month. His relationship for monthly runoff is

$$R_m = P_m - L_m \quad (19.4)$$

and $L_m = 0.48\, T_m \; for \; T_m > 4.5°C$ (19.5)

where $R_m$= monthly runoff in cm and $R_m \geq 0$; $P_m$ =monthly rainfall in cm; $L_m$= monthly losses in cm; $T_m$= mean monthly temperature of the catchment in °C

For, $T_m \leq 4.5°C$ the loss may provisionally be assumed as

| T°C | 4.5 | −1 | −6.5 |
|---|---|---|---|
| $L_m$ (cm) | 2.17 | 1.78 | 1.52 |

Annual runoff = $\sum R_m$

Khosla's formula is indirectly based on the water-balance concept and the mean monthly catchment temperature is used to reflect the losses due to evapotranspiration. The formula has been tested on a number of catchments in India and is found to give fairly good results for the annual yield for use in preliminary studies.

## 19.2 Rational Method

Consider a rainfall of uniform intensity and very long duration occurring over a basin. The runoff rate gradually increases from zero to a constant value as indicated in Fig. 19.1.

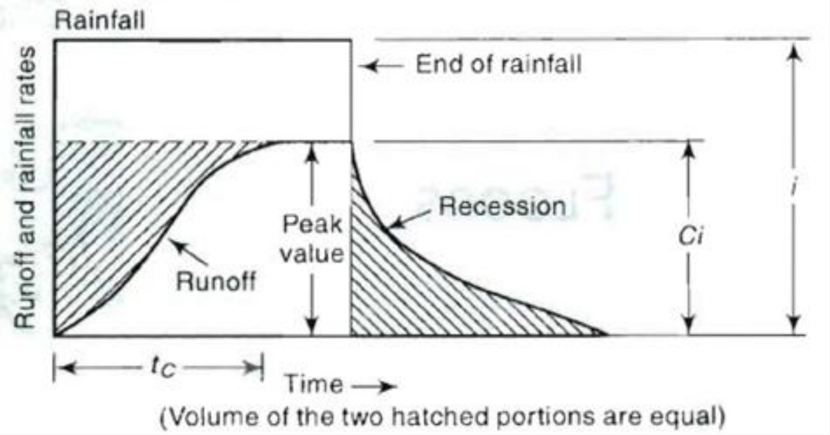


Fig. 19.1.Runoff Hydrograph due to Uniform Rainfall.(Source: Subramanya, 2008)

The runoff increases as more and more flow from remote areas of the catchment reach the outlet. Designating the time taken for a drop of water from the farthest part of the catchment to reach the outlet as $t_c$= time of concentration, it is obvious that if the rainfall continues beyond $t_c$, the runoff will be constant and at the peak value. The peak value of the runoff is given by

$$Q_P = C\,A\,i; for\ t \geq t_c \qquad (19.6)$$

WhereC is coefficient of runoff = (runoff/rainfall), A is area of the catchment andi is intensity of rainfall. This is the basic equation of the rational method.

Using the commonly used units, Eq. (19.6) is written for field application as

$$Q_p = \frac{1}{3.6}\,C\left(i_{tc,p}\right)A \qquad (19.7)$$

Where$Q_p$= peak discharge ($m^3/s$); C = coefficient of runoff; ($i_{tc,p}$) = the mean intensity of precipitation (mm/h) for a duration equal to $t_c$and an exceedence probability P; A= drainage area in $km^2$. The use of this method to compute $Q_p$requires three parameters: $t_c$,($i_{tc,p}$) and C.

### 19.2.1 Time of Concentration($t_c$)

KirpichEquation (1940)

This is the popularly used formula relating the time of concentration of the length of travel and slope of the catchment as

$$t_c = 0.01947 L^{0.77} S^{-0.385} \qquad (19.8)$$

Where$t_c$ is time of concentration (minutes); L is maximum length of travel of water (m) and S is slope of the catchment = ΔH/Lin which ΔH is difference in elevation between the most remote point on the catchment and the outlet.

Rainfall Intensity ($i_{tc,p}$)

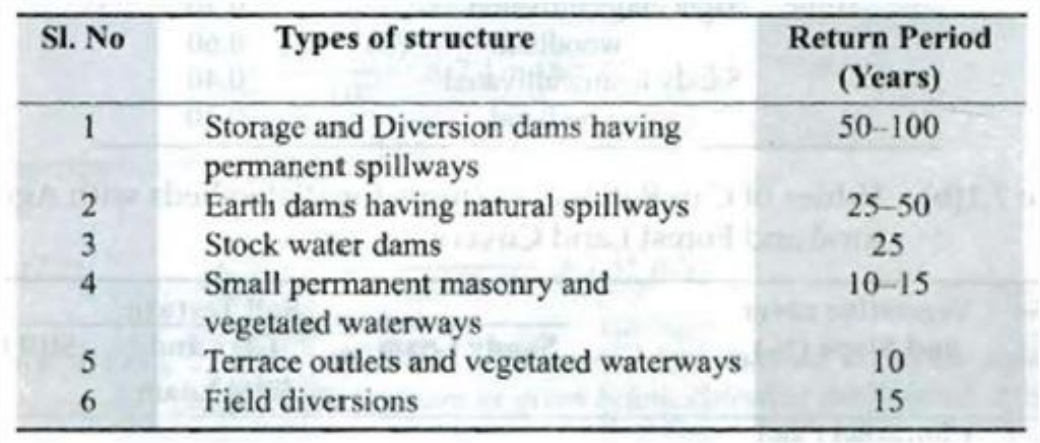
The rainfall intensity corresponding to a duration and the desired probability of exceedenceP, (i.e. return period T = 1/P) is found from the rainfall-frequency-duration relationship for the given catchment area. This relationship is given as

$$i_{tc,p} = \frac{KT^{x}}{(t_c + a)^{n}} \quad (19.9)$$

WhereK, a, x and n are coefficients, specific to a given area.

The recommended frequencies for various types of structures used in watershed development projects in India are as below:

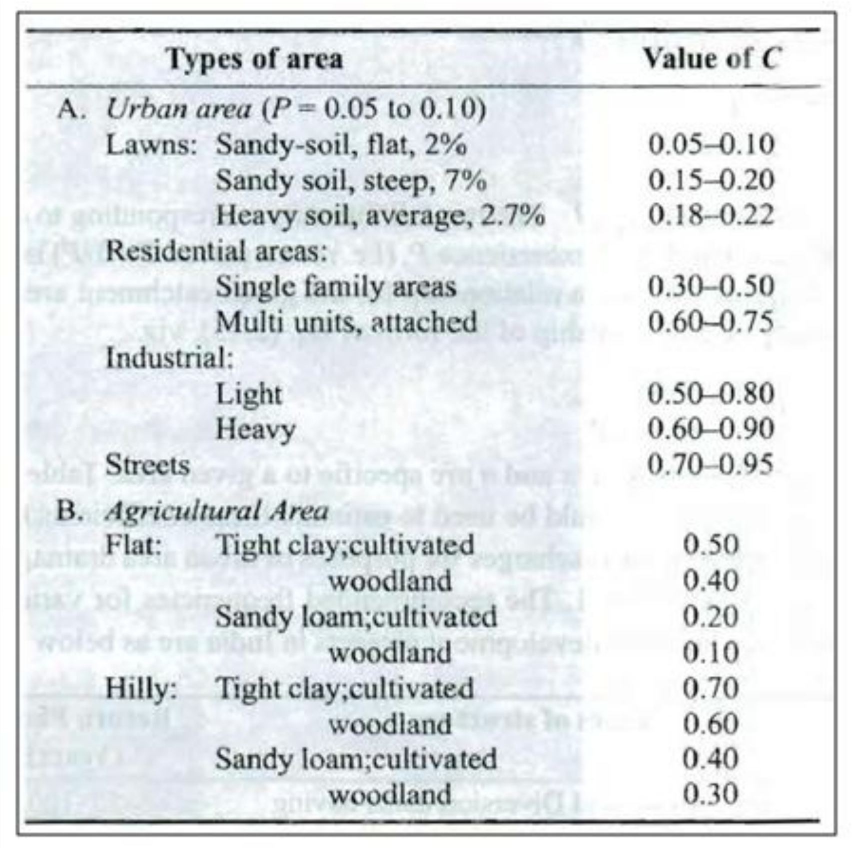
Table 19.3. Frequencies for various types of structures in India(Source: Subramanya, 2008)



| Sl. No | Types of structure | Return Period (Years) |
|---|---|---|
| 1 | Storage and Diversion dams having permanent spillways | 50–100 |
| 2 | Earth dams having natural spillways | 25–50 |
| 3 | Stock water dams | 25 |
| 4 | Small permanent masonry and vegetated waterways | 10–15 |
| 5 | Terrace outlets and vegetated waterways | 10 |
| 6 | Field diversions | 15 |

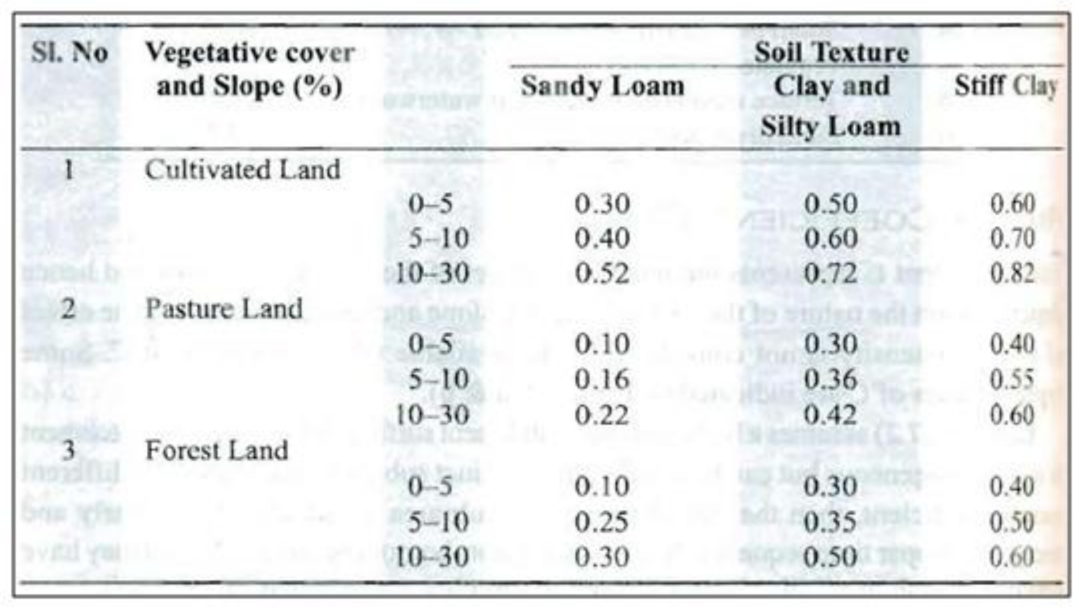
Runoff coefficient (C)

The coefficient C represents the integrated effect of the catchment losses and hence depends upon the nature of the surface, surface slope and rainfall intensity. Some typical values of C are indicated in Table 19.4 (a and b).



| Types of area | | Value of C |
|---|---|---|
| A. *Urban area* ($P = 0.05$ to $0.10$) | | |
| Lawns: | Sandy-soil, flat, 2% | 0.05–0.10 |
| | Sandy soil, steep, 7% | 0.15–0.20 |
| | Heavy soil, average, 2.7% | 0.18–0.22 |
| Residential areas: | | |
| | Single family areas | 0.30–0.50 |
| | Multi units, attached | 0.60–0.75 |
| Industrial: | | |
| | Light | 0.50–0.80 |
| | Heavy | 0.60–0.90 |
| Streets | | 0.70–0.95 |
| B. *Agricultural Area* | | |
| Flat: | Tight clay;cultivated | 0.50 |
| | woodland | 0.40 |
| | Sandy loam;cultivated | 0.20 |
| | woodland | 0.10 |
| Hilly: | Tight clay;cultivated | 0.70 |
| | woodland | 0.60 |
| | Sandy loam;cultivated | 0.40 |
| | woodland | 0.30 |

Table 19.4a. Value of the Coefficient C in Eq. (19.7)(Source: Subramanya, 2008)



| Sl. No | Vegetative cover and Slope (%) | | Soil Texture | | |
|---|---|---|---|---|---|
| | | | Sandy Loam | Clay and Silty Loam | Stiff Clay |
| 1 | Cultivated Land | | | | |
| | | 0–5 | 0.30 | 0.50 | 0.60 |
| | | 5–10 | 0.40 | 0.60 | 0.70 |
| | | 10–30 | 0.52 | 0.72 | 0.82 |
| 2 | Pasture Land | | | | |
| | | 0–5 | 0.10 | 0.30 | 0.40 |
| | | 5–10 | 0.16 | 0.36 | 0.55 |
| | | 10–30 | 0.22 | 0.42 | 0.60 |
| 3 | Forest Land | | | | |
| | | 0–5 | 0.10 | 0.30 | 0.40 |
| | | 5–10 | 0.25 | 0.35 | 0.50 |
| | | 10–30 | 0.30 | 0.50 | 0.60 |

Table 19.4b. Values of C in Rational Formula for Watersheds with Agricultural and Forest Land Covers(Source: Subramanya, 2008)

Equation (19.7) assumes a homogeneous catchment surface. If however, the catchment is non-homogeneous but can be divided into distinct sub areas each having a different runoff coefficient, then the runoff from each sub area is calculated separately and merged in proper time sequence. In such cases a weighted equivalent runoff coefficient $C_e$as below is used.

$$C_e = \frac{\sum_1^N C_i A_i}{A} \quad (19.10)$$

Whereis the areal extent of the sub area ihaving a runoff coefficient C, and N is number of sub areas in the catchment.

The rational formula is found to be suitable for peak-flow prediction in small catchment up to 50 km$^2$ in area. It finds considerable application in urban drainage designs and in the design of small culverts and bridges.

### 19.3 Cook's Method

Cook used a different approach to estimate the runoff from small agricultural areas. In this method, the runoff characteristics of a watershed are examined under four categories:

- Relief
- Infiltration
- Vegetative cover
- Surface storage

Based on observations of peak floods from agricultural areas, numerical values have been assigned to the various conditions of above categories by various investigators.

Steps

1. The sum,Σ W, of the numerical values assigned to the watershed characteristics is obtained.
2. Runoff curves (relating runoff with drainage area for Σ W values) are then entered with the drainage area and Σ W and the value of peak runoff for a 10 year recurrence interval is obtained.
3. This peak runoff value is then modified for recurrence interval and geomorphic characteristics by the formula:

$$Q = P \cdot R \cdot F \quad (19.11)$$

WhereQ is peak runoff for a specified RI and geomorphic location, P is peak runoff obtained in step 2, R is geomorphic rainfall factor (based on longitude and latitude), F is recurrence interval factor.

| Return Period, years | Factor, F |
|---|---|
| 10 | 1.0 |
| 25 | 1.2 |
| 50 | 1.4 |

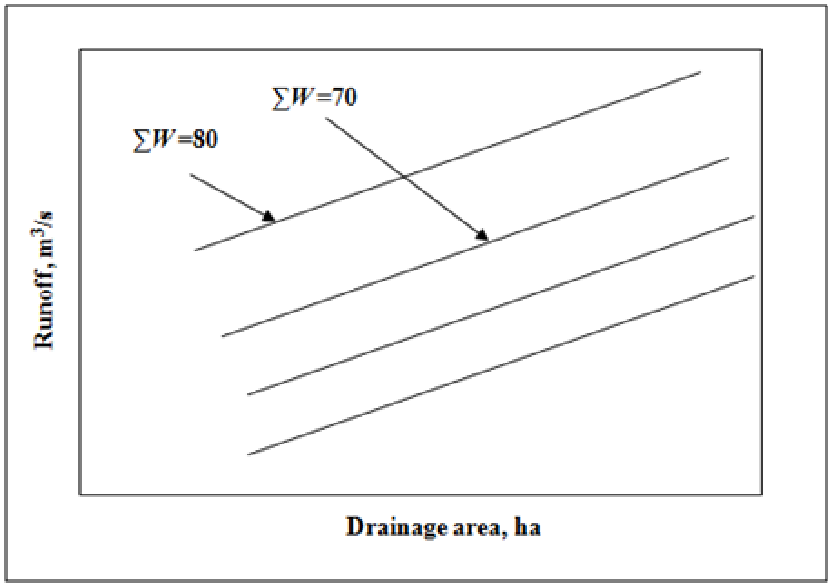


Fig. 19.2.Relationship between drainage area and runoff for a given RI. (Source: Suresh, 2007)

## Lesson 20 Estimation of Runoff-II

### 20.1 SCS Curve Number Method

This method is developed by the Soil Conservation Service (SCS) in 1969. It is now known as Natural Resources Conservation Service (NRCS).

The fundamental hypotheses of the SCS-CN method are as follows:

a) Runoff starts after an initial abstraction $I_a$has been satisfied. This abstraction consists principally of interception, surface storage, and infiltration.

b) The ratio of actual retention of rainfall to the potential maximum retention S is equal to the ratio of direct runoff to rainfall minus initial abstraction.

Mathematically,

$$\frac{P - I_a - V_Q}{S} = \frac{V_Q}{P - I_a}$$

$$V_Q = \frac{(P - I_a)^2}{(P - I_a) + S} \quad (20.1b)$$

Where is the runoff volume uniformly distributed over the drainage basin, P is the mean precipitation over the drainage basin, and S is the retention of water by the drainage basin. The quantity $I_a$can be expressed as a function of S. For Indian Conditions,$I_a$ = 0.3S. Physically, this means that for a given storm, 20% of the potential maximum retention is the initial abstraction before runoff begins. Presumably, 0.8S represents other retention losses, including interception, infiltration, evapotranspiration, and depression storage. Therefore,

$$V_Q = \frac{(P - 0.2S)^2}{(P + 0.8S)} \quad (20.2A)$$

Or

$$V_Q = P - S\left(1.2 - \frac{S}{P + 0.8S}\right) \quad (20.2B)$$

Evidently this is a one-parameter model containing S as the parameter and is illustrated graphically in Fig. 20.1.

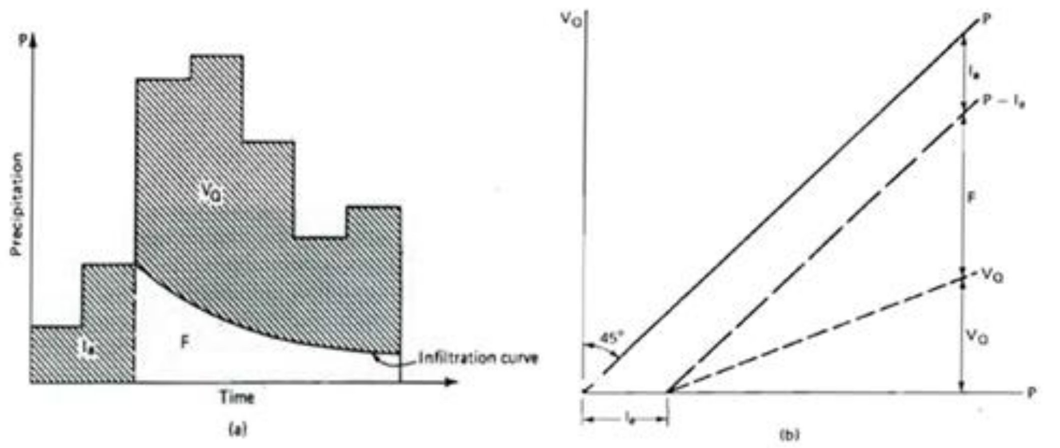


Fig. 20.1 (a) SCS relation between precipitation, runoff, and retention. (b) A mass-curve representation of the SCS relation between precipitation, runoff, and retention.

(Source: Singh, 1994)

Equation ( ) is a form of the hydrologic budget,

$$V_Q = P - L \quad (20.3a)$$

in which L accounts for losses expressed as

$$L = S\left(1.2 - \frac{S}{P + 0.8S}\right) \quad (20.3b)$$

In a limiting case, as P → ∞, L → 1.2 S and not S.

### 20.1.1 Estimation of S

The parameter S depends upon characteristics of the soil-vegetation-land use (SVL) complex and antecedent soil-moisture conditions in a watershed. For each SVL complex, there is a lower limit and an upper limit of S. The Soil Conservation Service expressed S as a function of curve number as

$$\mathrm{CN} = \frac{25400}{S + 254} \quad (20.4)$$

Or

$$S = \frac{25400}{\mathrm{CN}} - 254$$ (20.5)

Where CN is the curve number; it is a relative measure of retention of water by a given SVL complex and takes on values from 0 to 100. This number is derived from the character of the soil; vegetation, including crops; and the land use of that soil, as well as intensity of use. The unit of S is mm. obviously, when CN equals 100, S becomes zero. This leads to $V_Q$ = P. When S , CN 0.

This yields $V_Q$= 0 for all P when S = and CN = 0. Substitution of Eq. (20.5) into Eq. (20.2a) yields

$$V_Q = \frac{\left(P - \frac{200}{\mathrm{CN}} + 2\right)^2}{\left(P + \frac{800}{\mathrm{CN}} - 2\right)}$$ (20.6)

In this equation, CN is the only parameter to be determined.

### 20.1.2 Estimation of CN

The CN value is determined from (a) soil type and (b) antecedent moisture conditions. Soils have been classified in four groups. A short description is given here.

a) Group A: Soils in this group have a low-runoff potential (high-infiltration rates) even when thoroughly wetted. They consist of deep, well to excessively well-drained sands or gravels. These soils have a high rate of water transmission.

b) Group B:Soils in this group have moderate infiltration rates when thoroughly wetted and consist chiefly of moderately deep to deep, well-drained to moderately well-drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.

c) Group C: Soils have slow infiltration rates when thoroughly wetted and consist chiefly of soils with a layer that impedes the downward movement of water, or soils with moderately fine to fine texture. These soils have a slow rate of water transmission.

d) Group D:Soils have a high-runoff potential (very slow infiltration rates) when thoroughly wetted. These soils consist chiefly of clay soils with high swelling potential, soils with a permanent high-water table, soils with a clay pan or clay layer near the surface, and shallow soils over nearly impervious material. These soils have a very slow rate of water transmission.

### Antecedent Moisture Conditions

Antecedent moisture condition (AMC) refers to the water content present in the soil at a given time. The AMC value is intended to reflect the effect of infiltration on both the volume and rate of runoff, according to the infiltration curve of Fig.20.1. The Soil Conservation Service developed three antecedent soil-moisture conditions and labeled them as I, II, and III. These AMCs correspond to the following soil conditions:

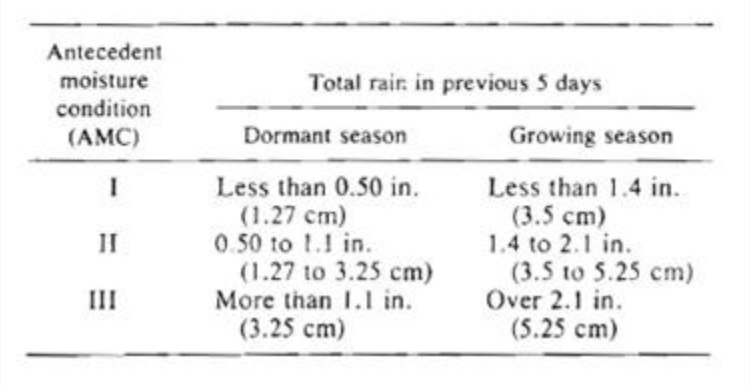
a) AMC I: Soils are dry but not to the wilting point; satisfactory cultivation has taken place.

b) AMC II: Average conditions.

c) AMC III: Heavy rainfall, or light rainfall and low temperatures have occurred within the last 5 days; saturated soil.

Table 20.1.Provides seasonal rainfall limits for the three antecedent soil-moisture infiltration conditions.

Table 20.1. Antecedent moisture conditions for determining the value of CN(Source: Singh, 1994)



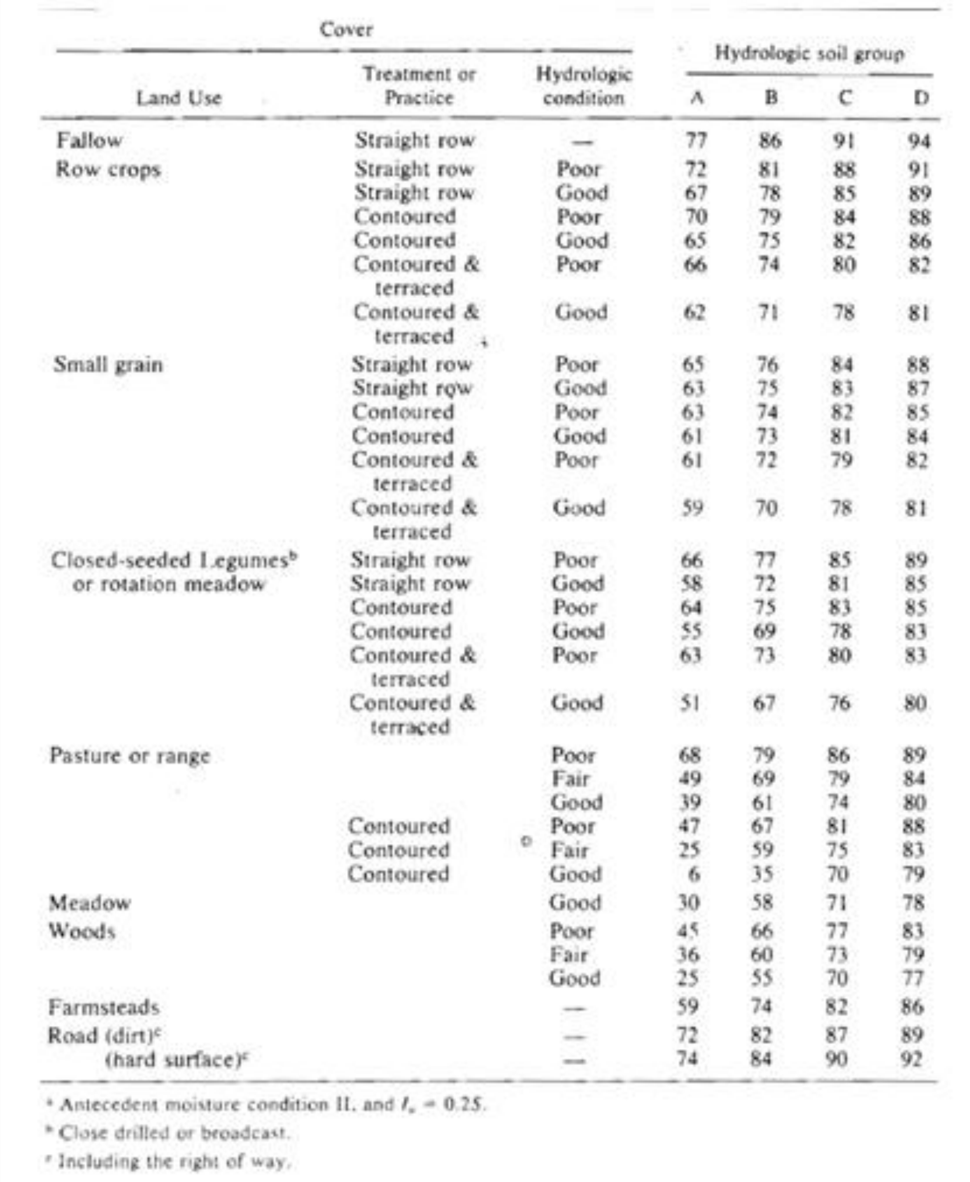
| Antecedent moisture condition (AMC) | Total rain in previous 5 days | |
|---|---|---|
| | Dormant season | Growing season |
| I | Less than 0.50 in.<br>(1.27 cm) | Less than 1.4 in.<br>(3.5 cm) |
| II | 0.50 to 1.1 in.<br>(1.27 to 3.25 cm) | 1.4 to 2.1 in.<br>(3.5 to 5.25 cm) |
| III | More than 1.1 in.<br>(3.25 cm) | Over 2.1 in.<br>(5.25 cm) |

Selection of CN

The procedure for selecting the actual value of CN for any given application begins in Table 20.2. The value of CN is shown for AMC condition II and for a variety of land uses, soil treatment, or farming practices. The hydrologic condition refers to the state of the vegetation growth. Upon selecting the applicable crop cover, treatment, and hydrologic condition, the value of CN is found under the appropriate soil group for AMC II. The value of CN is lower for soils with high infiltration than for soils with low infiltration.

Table 20.2. Runoff curve numbers for hydrologic soil-cover complex

(Source: Singh, 1994)



| Cover | | | Hydrologic soil group | | | |
|---|---|---|---|---|---|---|
| Land Use | Treatment or Practice | Hydrologic condition | A | B | C | D |
| Fallow | Straight row | — | 77 | 86 | 91 | 94 |
| Row crops | Straight row | Poor | 72 | 81 | 88 | 91 |
| | Straight row | Good | 67 | 78 | 85 | 89 |
| | Contoured | Poor | 70 | 79 | 84 | 88 |
| | Contoured | Good | 65 | 75 | 82 | 86 |
| | Contoured & terraced | Poor | 66 | 74 | 80 | 82 |
| | Contoured & terraced | Good | 62 | 71 | 78 | 81 |
| Small grain | Straight row | Poor | 65 | 76 | 84 | 88 |
| | Straight row | Good | 63 | 75 | 83 | 87 |
| | Contoured | Poor | 63 | 74 | 82 | 85 |
| | Contoured | Good | 61 | 73 | 81 | 84 |
| | Contoured & terraced | Poor | 61 | 72 | 79 | 82 |
| | Contoured & terraced | Good | 59 | 70 | 78 | 81 |
| Closed-seeded Legumes[b] or rotation meadow | Straight row | Poor | 66 | 77 | 85 | 89 |
| | Straight row | Good | 58 | 72 | 81 | 85 |
| | Contoured | Poor | 64 | 75 | 83 | 85 |
| | Contoured | Good | 55 | 69 | 78 | 83 |
| | Contoured & terraced | Poor | 63 | 73 | 80 | 83 |
| | Contoured & terraced | Good | 51 | 67 | 76 | 80 |
| Pasture or range | | Poor | 68 | 79 | 86 | 89 |
| | | Fair | 49 | 69 | 79 | 84 |
| | | Good | 39 | 61 | 74 | 80 |
| | Contoured | Poor | 47 | 67 | 81 | 88 |
| | Contoured | Fair | 25 | 59 | 75 | 83 |
| | Contoured | Good | 6 | 35 | 70 | 79 |
| Meadow | | Good | 30 | 58 | 71 | 78 |
| Woods | | Poor | 45 | 66 | 77 | 83 |
| | | Fair | 36 | 60 | 73 | 79 |
| | | Good | 25 | 55 | 70 | 77 |
| Farmsteads | | — | 59 | 74 | 82 | 86 |
| Road (dirt)[c] | | — | 72 | 82 | 87 | 89 |
| (hard surface)[c] | | — | 74 | 84 | 90 | 92 |

[a] Antecedent moisture condition II, and $I_a = 0.2S$.

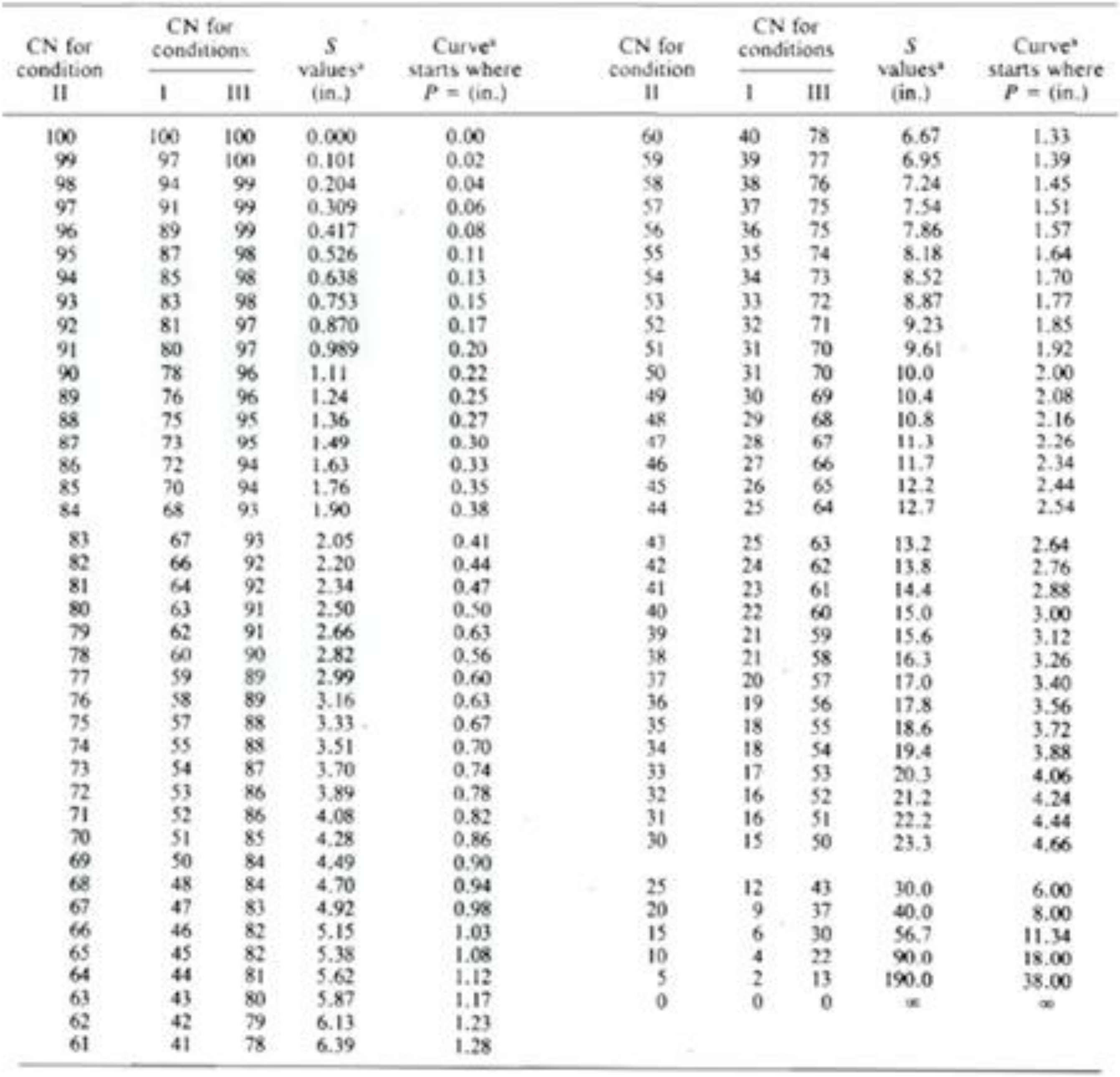
[b] Close drilled or broadcast.

[c] Including the right of way.

Table 20.3 provides conversion from AMC II to AMC I and AMC III. After determining the CN value for AMC II, Table 20.3 is entered at this value in column I. Values for AMC I, AMC III, S, and P are found in adjoining columns. These data can then be applied to determine runoff volume $V_Q$.

Table 20.3. Curve numbers (CN) and constants for the case $I_a = 0.2S$

(Source: Singh, 1994)



| CN for condition II | CN for conditions | | $S$ values$^a$ (in.) | Curve$^a$ starts where $P$ = (in.) | CN for condition II | CN for conditions | | $S$ values$^a$ (in.) | Curve$^a$ starts where $P$ = (in.) |
|---|---|---|---|---|---|---|---|---|---|
| | I | III | | | | I | III | | |
| 100 | 100 | 100 | 0.000 | 0.00 | 60 | 40 | 78 | 6.67 | 1.33 |
| 99 | 97 | 100 | 0.101 | 0.02 | 59 | 39 | 77 | 6.95 | 1.39 |
| 98 | 94 | 99 | 0.204 | 0.04 | 58 | 38 | 76 | 7.24 | 1.45 |
| 97 | 91 | 99 | 0.309 | 0.06 | 57 | 37 | 75 | 7.54 | 1.51 |
| 96 | 89 | 99 | 0.417 | 0.08 | 56 | 36 | 75 | 7.86 | 1.57 |
| 95 | 87 | 98 | 0.526 | 0.11 | 55 | 35 | 74 | 8.18 | 1.64 |
| 94 | 85 | 98 | 0.638 | 0.13 | 54 | 34 | 73 | 8.52 | 1.70 |
| 93 | 83 | 98 | 0.753 | 0.15 | 53 | 33 | 72 | 8.87 | 1.77 |
| 92 | 81 | 97 | 0.870 | 0.17 | 52 | 32 | 71 | 9.23 | 1.85 |
| 91 | 80 | 97 | 0.989 | 0.20 | 51 | 31 | 70 | 9.61 | 1.92 |
| 90 | 78 | 96 | 1.11 | 0.22 | 50 | 31 | 70 | 10.0 | 2.00 |
| 89 | 76 | 96 | 1.24 | 0.25 | 49 | 30 | 69 | 10.4 | 2.08 |
| 88 | 75 | 95 | 1.36 | 0.27 | 48 | 29 | 68 | 10.8 | 2.16 |
| 87 | 73 | 95 | 1.49 | 0.30 | 47 | 28 | 67 | 11.3 | 2.26 |
| 86 | 72 | 94 | 1.63 | 0.33 | 46 | 27 | 66 | 11.7 | 2.34 |
| 85 | 70 | 94 | 1.76 | 0.35 | 45 | 26 | 65 | 12.2 | 2.44 |
| 84 | 68 | 93 | 1.90 | 0.38 | 44 | 25 | 64 | 12.7 | 2.54 |
| 83 | 67 | 93 | 2.05 | 0.41 | 43 | 25 | 63 | 13.2 | 2.64 |
| 82 | 66 | 92 | 2.20 | 0.44 | 42 | 24 | 62 | 13.8 | 2.76 |
| 81 | 64 | 92 | 2.34 | 0.47 | 41 | 23 | 61 | 14.4 | 2.88 |
| 80 | 63 | 91 | 2.50 | 0.50 | 40 | 22 | 60 | 15.0 | 3.00 |
| 79 | 62 | 91 | 2.66 | 0.63 | 39 | 21 | 59 | 15.6 | 3.12 |
| 78 | 60 | 90 | 2.82 | 0.56 | 38 | 21 | 58 | 16.3 | 3.26 |
| 77 | 59 | 89 | 2.99 | 0.60 | 37 | 20 | 57 | 17.0 | 3.40 |
| 76 | 58 | 89 | 3.16 | 0.63 | 36 | 19 | 56 | 17.8 | 3.56 |
| 75 | 57 | 88 | 3.33 | 0.67 | 35 | 18 | 55 | 18.6 | 3.72 |
| 74 | 55 | 88 | 3.51 | 0.70 | 34 | 18 | 54 | 19.4 | 3.88 |
| 73 | 54 | 87 | 3.70 | 0.74 | 33 | 17 | 53 | 20.3 | 4.06 |
| 72 | 53 | 86 | 3.89 | 0.78 | 32 | 16 | 52 | 21.2 | 4.24 |
| 71 | 52 | 86 | 4.08 | 0.82 | 31 | 16 | 51 | 22.2 | 4.44 |
| 70 | 51 | 85 | 4.28 | 0.86 | 30 | 15 | 50 | 23.3 | 4.66 |
| 69 | 50 | 84 | 4.49 | 0.90 | | | | | |
| 68 | 48 | 84 | 4.70 | 0.94 | 25 | 12 | 43 | 30.0 | 6.00 |
| 67 | 47 | 83 | 4.92 | 0.98 | 20 | 9 | 37 | 40.0 | 8.00 |
| 66 | 46 | 82 | 5.15 | 1.03 | 15 | 6 | 30 | 56.7 | 11.34 |
| 65 | 45 | 82 | 5.38 | 1.08 | 10 | 4 | 22 | 90.0 | 18.00 |
| 64 | 44 | 81 | 5.62 | 1.12 | 5 | 2 | 13 | 190.0 | 38.00 |
| 63 | 43 | 80 | 5.87 | 1.17 | 0 | 0 | 0 | ∞ | ∞ |
| 62 | 42 | 79 | 6.13 | 1.23 | | | | | |
| 61 | 41 | 78 | 6.39 | 1.28 | | | | | |

## Lesson 21 Measurement of Runoff

Most hydrologic analyses involve runoff from a drainage area, and hence its measurement is of vital importance. If precipitation and runoff 'can be accurately measured, it is then possible to estimate the total loss on a drainage basin. This information can help predict runoff from similar drainage basins that have no gages. Streamflow measurements are used to develop physical or statistical relations between other variables and runoff volume or peak discharge. These relations form the basis for many calculations to predict streamflow characteristics of ungauged basins.

Streamflow is measured in units of discharge ($m^3/s$ or cfs). Direct measurement of discharge is an expensive, time-consuming procedure. Two steps are employed, therefore, to obtain streamflow measurements. First, the discharge of a specified stream is related to the water-surface elevation or stage using a series of careful measurements, and a stage-discharge relationship, popularly called a rating curve, is established for that stream. Second, the water stage is observed routinely, and the corresponding discharge is obtained from the rating curve. The measurement of stage is easy and relatively inexpensive.

### 21.1 Measurement of Stage

### 21.1.1 Non-recording Stream Gauges

Commonly used non recording stream gages are staff gages and wire gages which are manually operated. A staff gauge, as shown in Fig. 21.1.is the simplest way to measure the river stage. It may be mounted vertically or at an angle from the vertical. The staff is rigidly attached to a permanent structure such as a bridge pier wall abutment, etc. The gage indicates water-surface elevation on a staff that is graduated with clear and accurate markings in tenths of a foot or in centimeters. A portion of the scale is immersed in the water at all times.

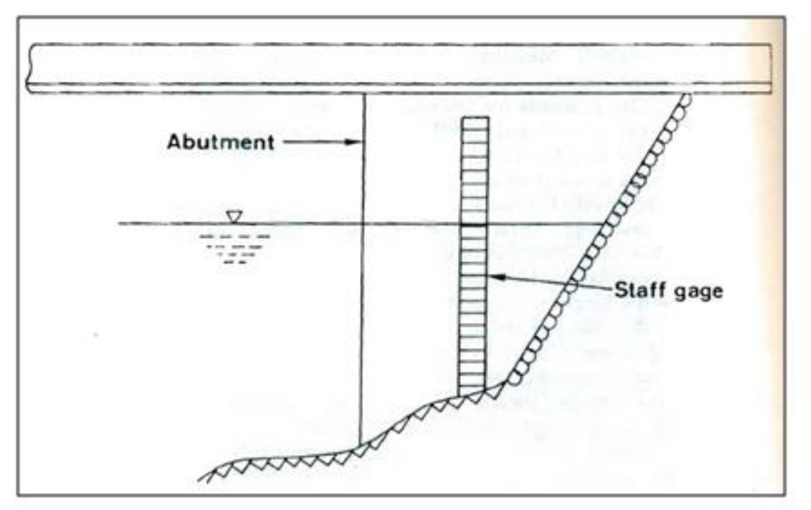


Fig. 21.1a. Staff Gauges.(Source: Singh, 1994)

Sometimes, a single gage is not adequate for all stages; a sectional staff gauge as shown in Figure 21.2, is used. Sectional gages are installed to provide overlap between various gages with their readings corresponding to the same datum.

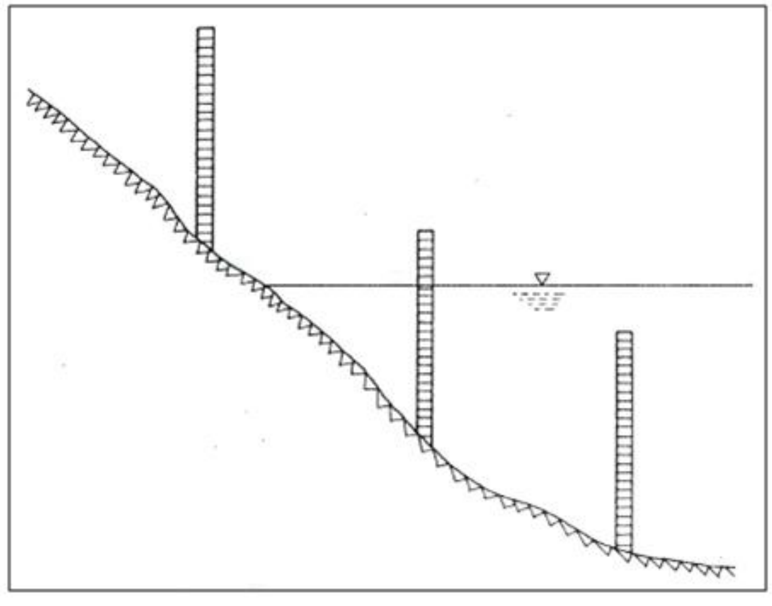


Fig. 21.1b.Sectional gauge.(Source: Singh, 1994)

A wire gauge measures the water-surface elevation from above such as from a bridge or other overhead structure. A weight is lowered from the structure until it reaches the water surface. The gauge has a drum with a circumference equal to 1 ft or 1 m of wire. The number of revolutions of the drum is measured by a mechanical counter, which, in turn, measures the length of the wire transmitted to reach the water surface. The operating range of a wire-weight gage is about 25 m.

### 21.1.2 Crest-stage Gauges

A special application of a staff-gage installation is the crest gauge. The crest gage is designed to obtain a measurement of the peak discharge in a channel reach during a flood event. A crest gauge consists of an ordinary staff gauge of sufficient width hand selected length to fit into a 2-inch galvanized pipe, as shown in Fig. 21.3. This galvanized pipe is fitted with threaded pipe caps on either end in which are drilled several holes of approximately 0.25 inch in diameter. The pipe is mounted vertically in the stream channel with it s bottom at the stream datum. The staff gage is inserted at the top of the pipe along with about a capful of ground cork and the top ventilated pipe cap is replaced.

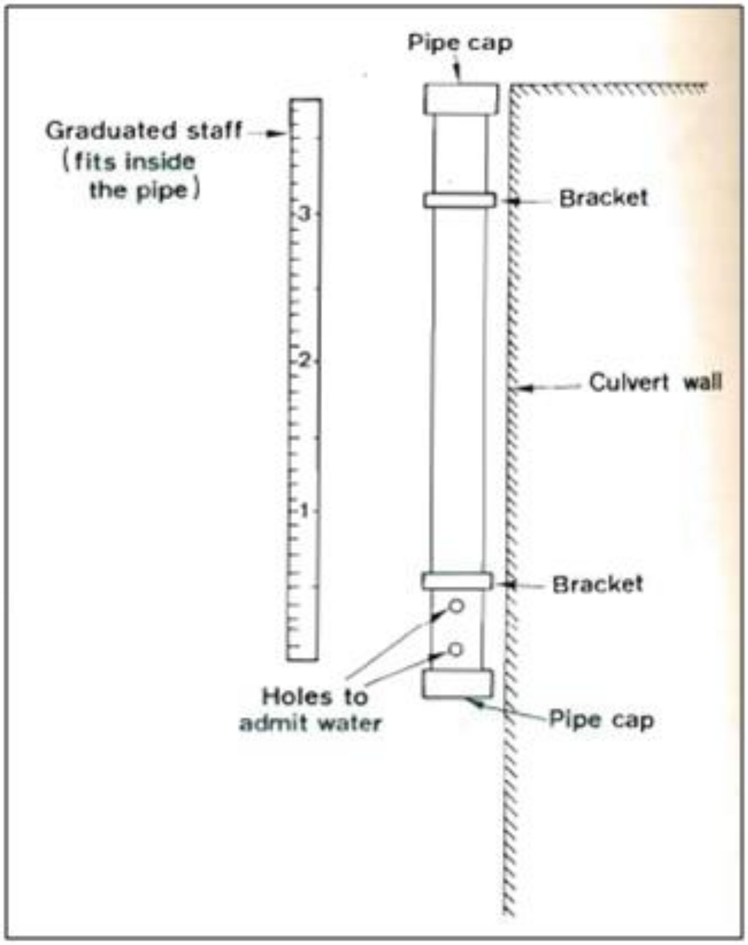


Fig. 21.3.Crest-staff gauge.(Source: Singh, 1994)

During a flood event, water enters the lower cap perforations and rises in the pipe, carrying the ground cork with it. At the highest stage, the cork adheres to the wetter staff, and as the water recedes, a visual record of the highest stage is indicated by the cork adhering to the staff. The operator only needs to remove the stall read the high stage and wipe the staff clean for additional use. Additional ground cork may be added, as necessary to replace that which is lost.

### 21.1.3 Recording Stream Gauges

Recording stream gages are instruments that continuously record the water stage at a given location along the stream. Two types are in general use: (i) the float type and (ii) the bubble gauge or manometer-servo water-level sensor.

a) Float type gauge

The float-type recorder is shown in Fig. 21.4.

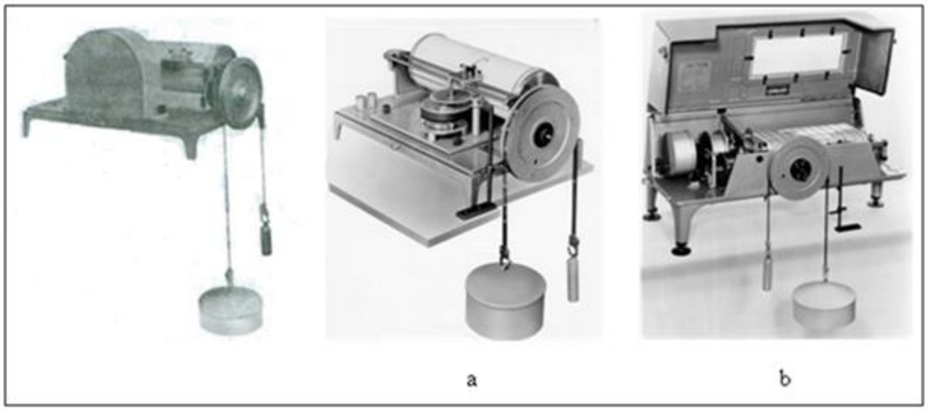


Fig. 21.4.Float-type gauge recorder: (Stevens's type) (a) pen and chart type (b) digital.(Source: http://www.usbr.gov/pmts/hydraulics_lab/pubs/wmm/chap06_05.html)

The recorder is located in a stilling well that consists of a vertically mounted culvert or other similar structure that may be from 1 to 3 feet in diameter, as shown in Fig. 21.5. The culvert is installed on the .bank of the channel to a depth at least equal to the lowest level of the channel bottom. A 2-inch open-ended galvanized pipe is run horizontally from the bottom or near the bottom of the deepest part of the channel and fastened to the culvert. The top of this pipe is usually used as the stream-gage datum because it marks the lowest stage of streamflow or nearly so.

The recorder is mounted in a weatherproof housing at the top of the culvert and stilling well. Water rises in the stilling well through the 2-inch galvanized pipe to a level equal to the water elevation in the channel. The purpose of the stilling well is to dampen the water-surface fluctuations so that the float records changes in water elevation, but does not reflect wave action or other interference. Water-level changes are recorded on punched tape at selected time intervals, often every 15 minutes. This instrument can be adapted to remote monitoring by telephone or radio.

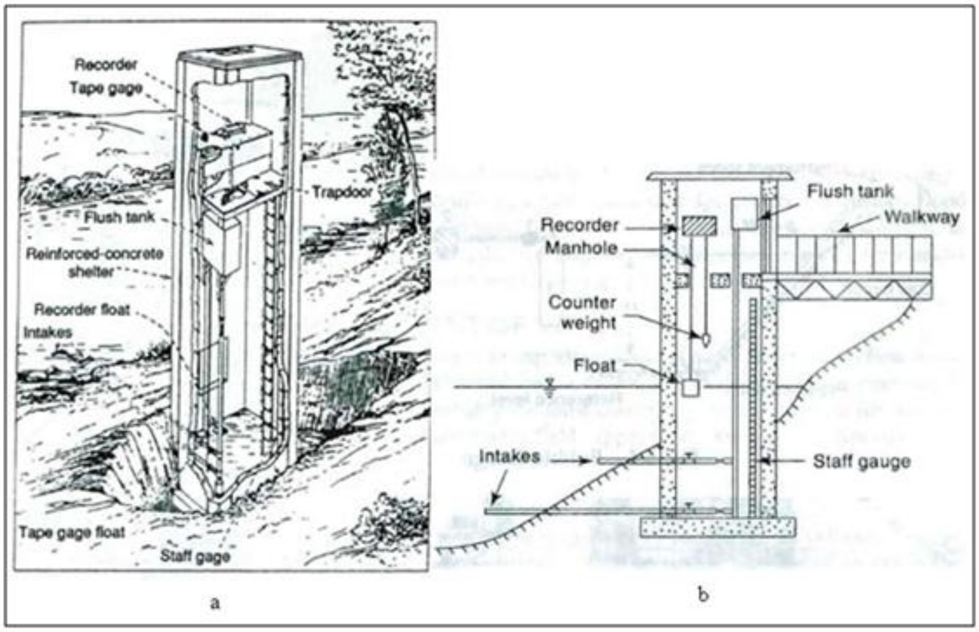


Fig. 21.5.Stilling well for float-type recorder (a) field installation (b) schematic.(Source: Singh, 1994)

Older recorders, from which many records exist, used a pen that marked stage elevations on a chart mounted on a clock-driven drum. These instruments required the chart to be changed periodically.

a) Bubble gauge

The bubble gage, or manometer-servo water-level sensor, is shown in Fig. 21.6. This instrument eliminates the need for a stilling well, but requires battery power and a 116-ft$^3$ dry nitrogen cylinder for operating up to 6 months.Pressure corresponding to the water head in the channel is imparted to the recorder through a tube in the bottom of the channel. This tube is supplied with nitrogen gas pressure equal to the water head by a servo motor that automatically adjusts for changes in water head. This instrument is attached to a digital recorder, similar to the float-type recorder, and can be remotely monitored by telephone or radio.

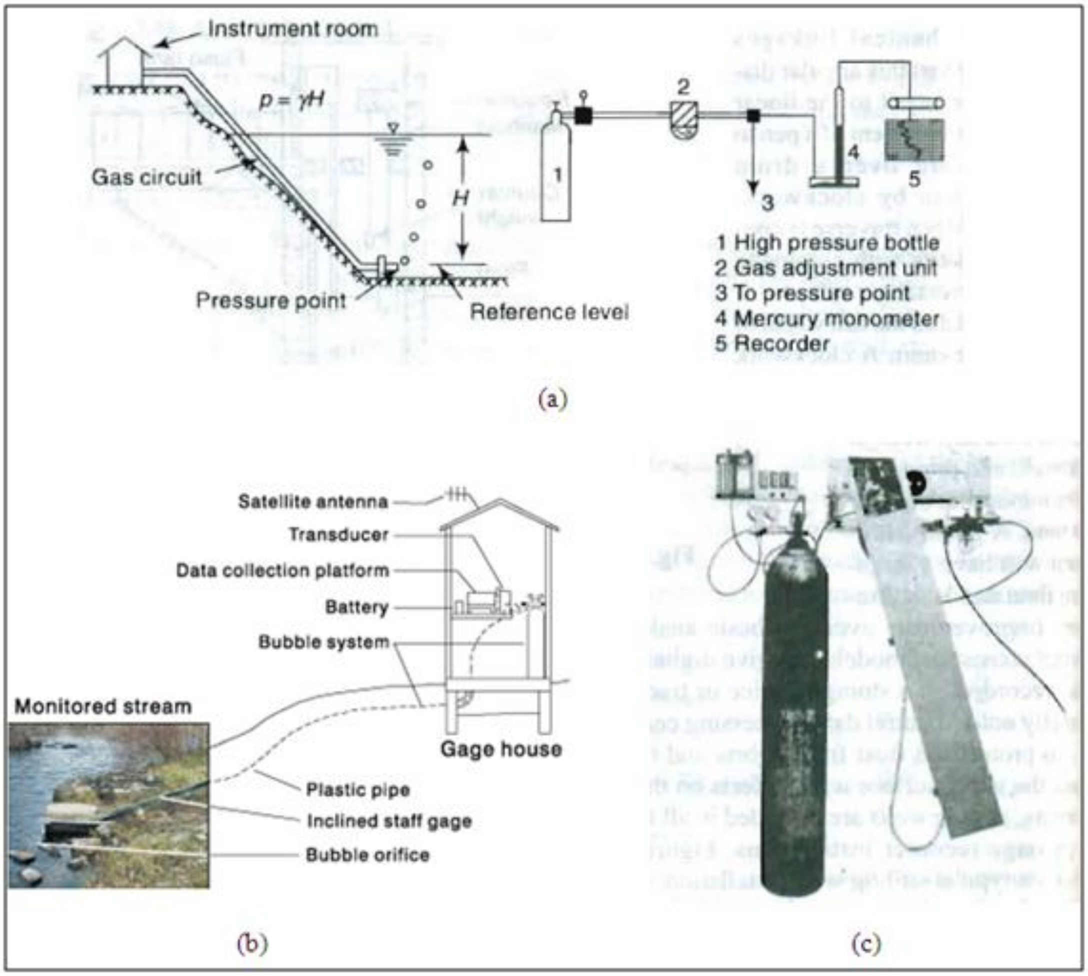


Fig. 21.6. Bubble gauge recorder: (a) Schematic of bubble gauge installation (b) Field installation of bubble gauge (c) Bubble gauge: Stevens's manometer servo.

(Source: http://nh.water.usgs.gov/gauge_station/3_howusgs.htm)(Singh, 1994)

The bubble gage may be preferable to the float-type recording gage for the following reasons:

1. The stilling well, which is expensive, is not needed.
2. Large changes (up to 30 m) in water-surface elevation can be measured.
3. The inlet is less likely to be blocked because of the gas pressure.
4. The recorder assembly can be located far away from the sensing point.

### 21.1.4 River-stage Data

The stage data are presented chronologically in time as a time series. Their plot, as shown in Fig. 21.7, is called a stage hydrograph. The primary use of this data is the determination of discharge. Other uses of this data are in flood-insurance studies, design of flood-protection works, flood warning and evacuation, urban development, flood-damage assessment, water diversion, navigation, etc. Long-term stage data are needed to estimate peak river stages for application in the design of hydraulic structures such as bridges, culverts, weirs, etc.

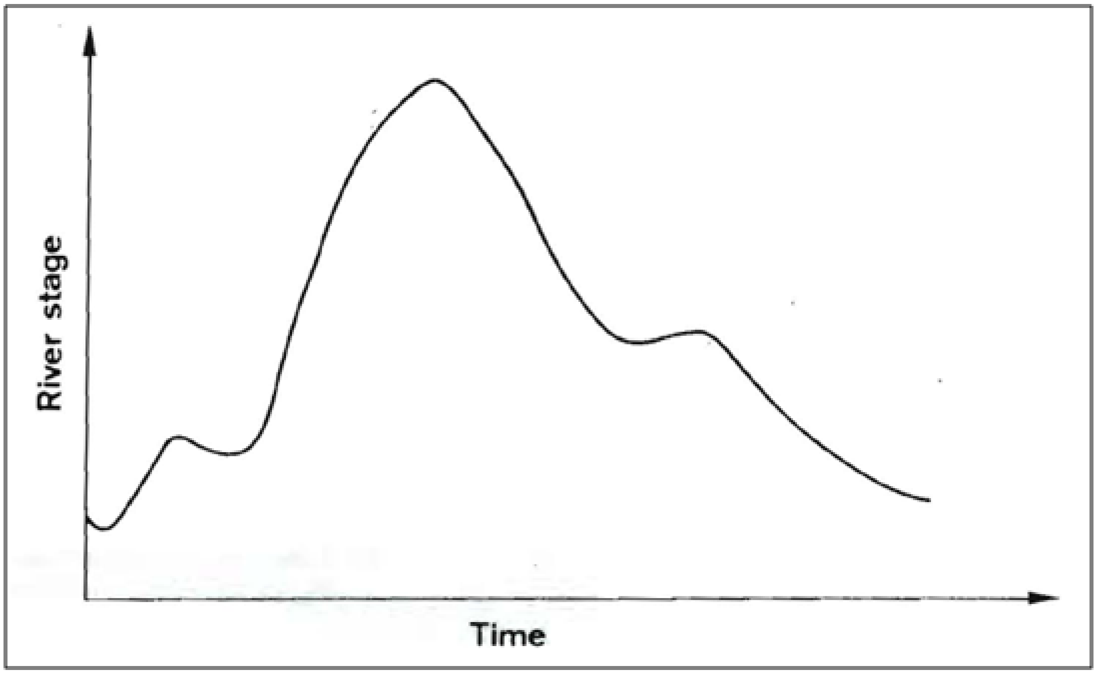


Fig. 21.7.Stage hydrograph.(Source: Singh, 1994)

## 21.2 Measurement of Velocity(Singh, 1994)

### 21.2.1 Velocity Distribution

Velocity distribution in a channel is not uniform over the width and depth of the channel. Velocity is greatest in the deepest part of the channel and is zero along the boundary of flow as shown in Fig. 21.8.

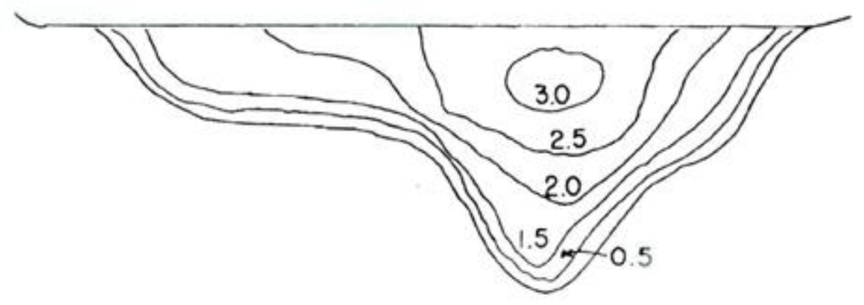


Fig. 21.8.Velocity distribution in a non-uniform channel.

(Source: Singh, 1994)

The greatest velocity occurs just under the water surface in the deepest part of the channel of a straight reach.

For computing the average vertical velocity of streamflow, following criterion is considered:

1.The average velocity in shallow streams with depths of flow not exceeding 3 m is taken as the velocity measured at 0.6 times the depth of flow, ,below the water surface,

$$\bar{v} = v_{0.6} \quad (21.1)$$

This procedure needs a single-point measurement.

2.The average velocity in moderately deep streams is computed as

$$\bar{v} = \frac{v_{0.8} + v_{0.2}}{2} \quad (21.2)$$

Where $v_{0.2}$ is the velocity measured at 0.2 times the depth of flow below the water surface andis the velocity measured at 0.8 times the depth of flow below the water surface.

3. The average velocity in rivers having flood flows is obtained from a single measurement as

$$\bar{v} = cv_{0.5} \quad (21.3)$$

Where $v_{0.5}$ is the surface velocity measured within a depth of 0.5 m below the water surface; and c is a reduction factor, which is usually between 0.85 and 0.95, and is obtained from measurements taken at lower stages.

### 21.2.2 Estimating the Mean Channel Velocity of Floats

It is possible to estimate the mean velocity of flow in a channel, when necessary, by timing the movement of a float along a measured channel reach.

$$\bar{v} = \frac{L}{T} \quad (21.4)$$

WhereL is the distance traveled in meters, and T is the time of travel in seconds.

This method must be used in a straight reach of channel on a windless day so that the float can be maintained in the center of the channel. Because velocity varies across the width and depth of flow in the stream, coefficients must be applied to convert surface-float (Fig. 21.9) velocity to mean channel velocity. These coefficients, shown in Table 21.1., have been developed from empirical relations.

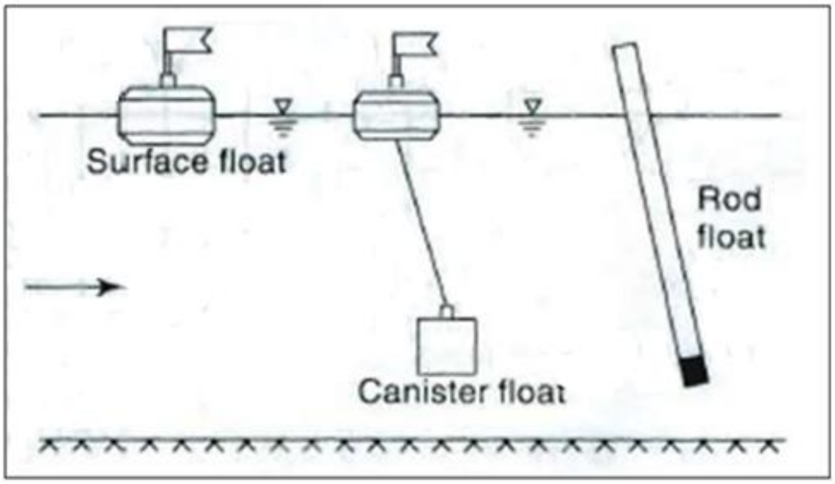
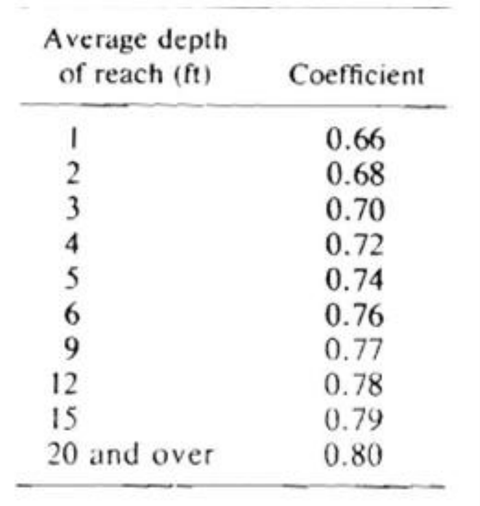


Fig. 21.9.Floats.(Source: Singh, 1994)

Table 21.1a. Coefficients for surface-float velocity measurements

(Source: Singh, 1994)

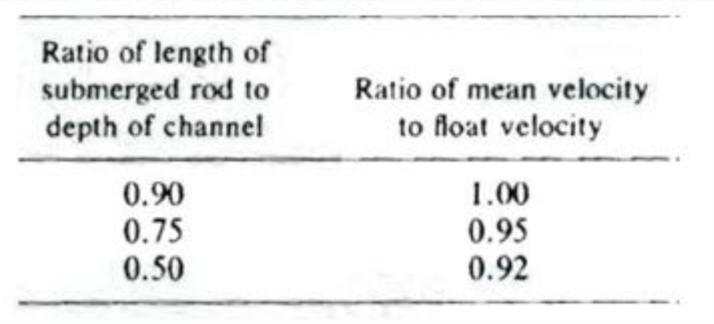


| Average depth of reach (ft) | Coefficient |
|---|---|
| 1 | 0.66 |
| 2 | 0.68 |
| 3 | 0.70 |
| 4 | 0.72 |
| 5 | 0.74 |
| 6 | 0.76 |
| 9 | 0.77 |
| 12 | 0.78 |
| 15 | 0.79 |
| 20 and over | 0.80 |

Somewhat better results are obtained when using a rod float(Fig. 21.9) 1 to 2 inches in diameter. The rod must be weighted on one end such that it floats vertically in the water and should be of such a length that the immersed length is 0.9 times the depth of water. Under these conditions, the rod tends to integrate the variations in vertical velocity distribution and tends to move at approximately the mean velocity. Table 21.2 gives coefficients for conversion of rod-float velocity to mean channel velocity.

Table 21.1b. Coefficients for rod-float velocity measurements

(Source: Singh, 1994)



| Ratio of length of submerged rod to depth of channel | Ratio of mean velocity to float velocity |
|---|---|
| 0.90 | 1.00 |
| 0.75 | 0.95 |
| 0.50 | 0.92 |

### 21.2.3 Current Meters

The current meter is the most commonly used instrument to measure velocity of flow in streams.

It consists essentially of a rotating element which rotates due to the reaction of the stream current with an angular velocity proportional to the stream velocity.

There are two main types of current meters: (i) Vertical-axis meters, and (ii) Horizontal-axis meters.

1) Vertical-Axis Meters

These instruments consist of a series of conical cups mounted around a vertical axis (Fig. 21.10. and 21.11.).

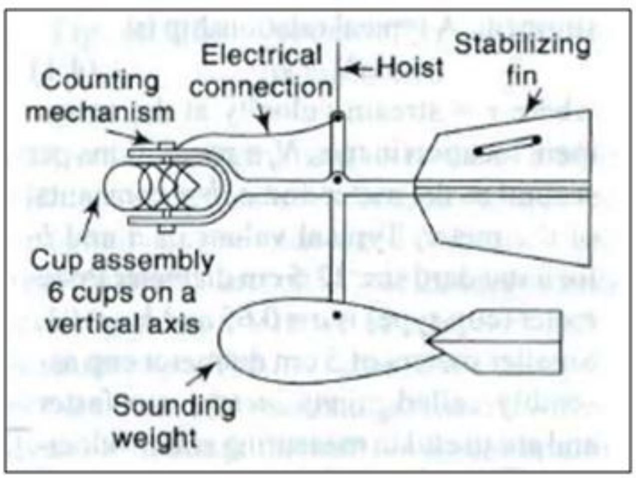


Fig. 21.10.Vertical-axis current meter. (Source: Subramanya, 2008)

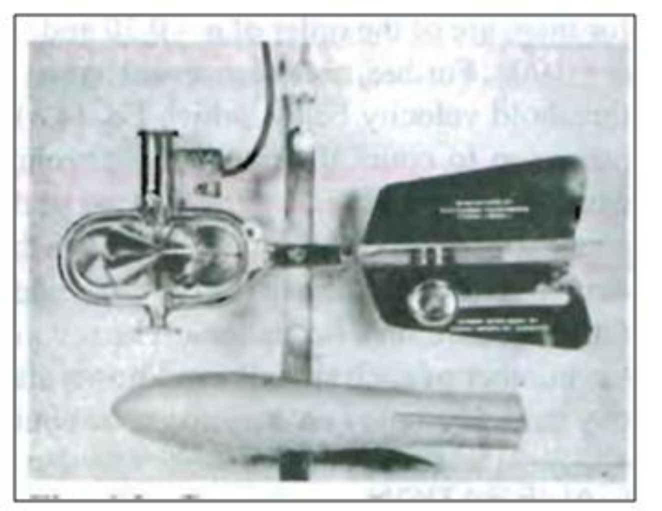


Fig. 21.11.Cup type current meter with sounding weight.

(Source: Subramanya, 2008)

The cups rotate in a horizontal plane and a cam attached to the vertical axial spindle records generated signals proportional to the revolutions of the cup assembly. The Price current meter and Gurley current meter are typical instruments under this category. The normal range of velocities is from 0.15 to 4.0 m/s.

The accuracy of these instruments is about 1.50% at the threshold value and improves to about 0.30% at speeds in excess of 1.0 m/s. Vertical-axis instruments have the disadvantage that they cannot be used in situations where there are appreciable vertical components of velocities.

**(2) Horizontal-axis Meters**

These meters consist of a propeller mounted at the end of horizontal shaft as shown in Fig. 21.12. These come in a wide variety of size with propeller diameters in the range 6 to 12 cm, and can register velocities in the range of 0.15 to 4.0m/s. Ott, Neyrtec and Watt type meters are typical instruments under this kind. These meters are fairly rugged and are not affected by oblique flows of as much as 15°. The accuracy of tile instruments is about 1% at the threshold value and about 0.25% at a velocity of 0.3 m/sand above.

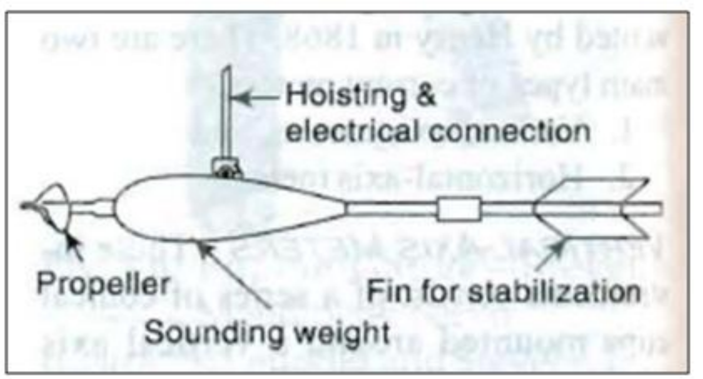


Fig. 21.12a.Horizontal-axis current meter. (Source: Subramanya, 2008)

A current meter is so designed that its rotation speed varies linearly with the stream velocity v at the location of the instrument. A typical relationship is

$$v = aN_s + b \quad (21.5)$$

Wherev is the stream velocity at the instrument location in m/s; N$_s$. = revolutions per second of the meter and a, b =constants of the meter.Typical values of a andb for a standard size 12.5 cm diameter Price meter (cup-type) (Fig. 21.13) is a = 0.65 and b=0.03. Smaller meters of 5 cm diameter cup assembly called pigmy meters run faster are useful in measuring small velocities. The value of the meter constants for them are of the order of a = 0.30 and b= 0.003.

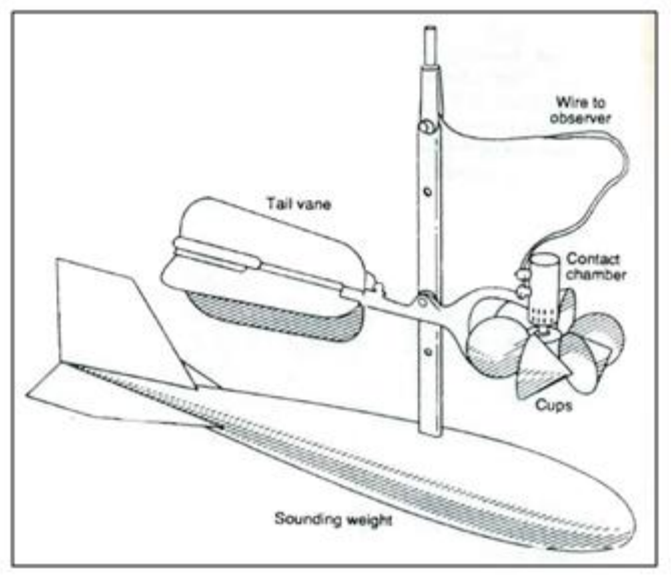


Fig. 21.12b.Price current meter.(Source: Singh, 1994)

## 21.3 Measurement of Discharge

### 21.3.1 Velocity-area Station Method

Velocity-area stations have stream gauges that measure velocities and cross-sectional areas of flow to obtain discharges. The basic equation used to obtain the discharge from velocity-area stations is

$$Q = VA \quad (21.6)$$

WhereQ is the discharge in cubic meters per second (m$^3$/s), V is the mean velocity of flow through the cross-sectional area of the channel in meters per second (m/s), and A is the cross-sectional area of flow in square meters (m$^2$). Equation 21.6 can be used as a continuity equation because the mass flow into a point must equal the mass flow out from that point. To determine the discharge both the mean velocity and cross-sectional area of flow must be measured.

In order to conduct stream-discharge measurements, the stream channel must be subdivided into several smaller widths. The vertical dashed lines in Fig. 21.13 represent subdivisions of the stream width.

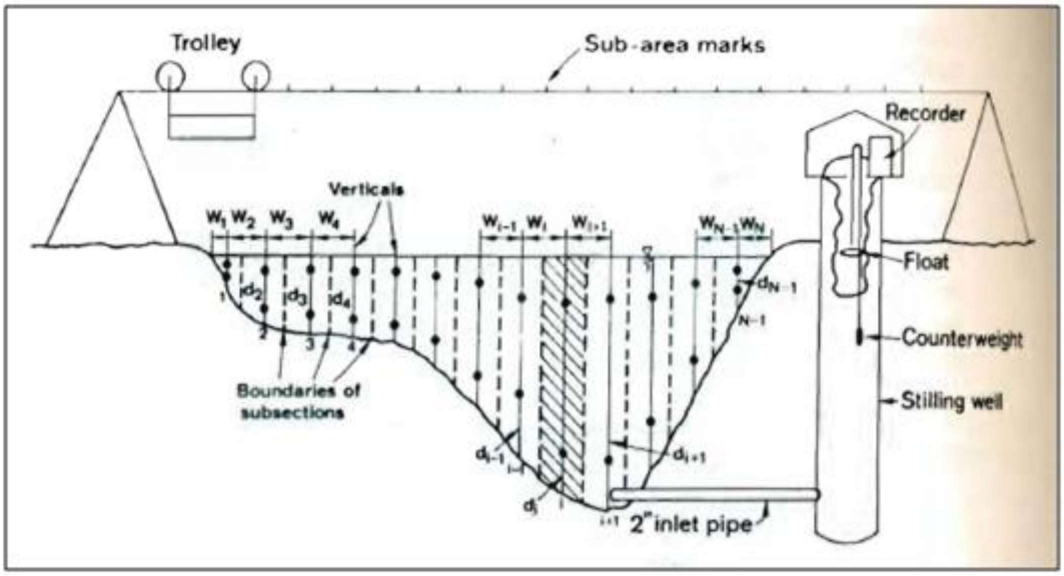


Fig. 21.13.Schematic sketch of a velocity-area station.(Source: Singh, 1994)

The average velocity in each of the sub-width divisions is measured. The mean depth of flow of this subdivision is measured using sounding rods, sounding weights or an echo-depth recorder (an electroacoustic instrument). A current meter is used to measure velocity for this subdivision. Velocity is measured from observations at points 0.2 and 0.8 of the total depth. These two velocity measurements are averaged to obtain the mean velocity in that subwidth. The discharge for the subdivision is computed by taking the product of the mean depth, subarea width and mean velocity. Equation 21.6 is applied to the subdivision. Total discharge for the stream is determined by summing the discharges for all subwidths. The mean stream velocity can be computed by dividing the total discharge by the total cross-sections area. The velocity-area method when using the current-meter method is also referred to as the standard current-meter method.

**Calculation of Discharge**

Fig. 21.13 shows the cross section of a river in which N -1 verticals are drawn. The velocity averaged over the vertical at each section is known. Considering the total area to be divided into N -1 segments, the total discharge is calculated by the method of mid-sections as follows.

$$Q = \sum_{i=1}^{N-1} \Delta Q_i \quad (21.7)$$

Where ΔQ in the i th segment

$$\Delta Q_i = y_i \times \left(\frac{W_i}{2} + \frac{W_{i+1}}{2}\right) \times v_i \text{for } i = 2\text{to}(N-2) \quad (21.8)$$

For the first and last sections, the segments are taken to have triangular areas and area calculated as

$$\Delta A_1 = \bar{W}_1 \cdot y_1$$

Where

$$\bar{W}_i = \frac{\left(W_1 + \frac{W_2}{2}\right)^2}{2W_1} \text{ and } \Delta A_N = \bar{W}_{N-1} \cdot y_{N-1} \quad (21.10)$$

$$\bar{W}_{N-1} = \frac{\left(W_N + \frac{W_{N-1}}{2}\right)^2}{2W_N} \quad (21.11)$$

$$\Delta Q_1 = \bar{v}_1 \cdot \Delta A_1 \text{and} \Delta Q_{N-1} = \bar{v}_{N-1} \cdot \Delta A_{N-1} \quad (21.12)$$

### 21.3.2 Moving Boat Method

Discharge measurement of large alluvial rivers, such as the Ganga, by the standard current meter is very time consuming even when the flow is low or moderate. When the river is in spate, it is almost impossible to use the standard current meter technique due to the difficulty of keeping the boat stationary on the fast-moving surface of the stream for observation purposes. It is in such circumstance that the moving boat techniques prove very helpful.

In this method a special propeller-type current meter which is free to move about a vertical axis is towed in a boat at a velocity $v_b$at right angles to the stream flow. If the flow velocity is $v_f$the meter will align itself in the direction f resultant velocity $v_R$making an angle θwith the direction the boat (Fig. 21.14).

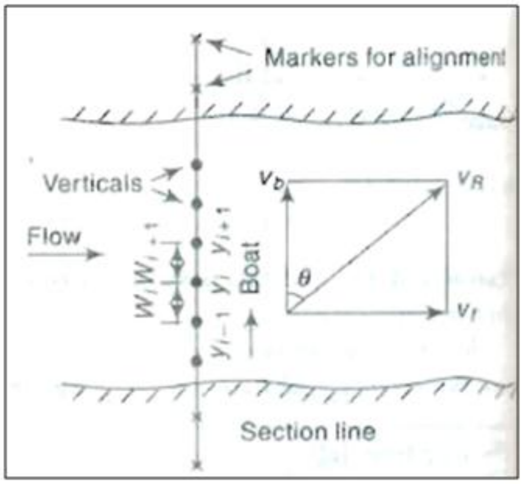


Fig. 21.14.Moving-boat method. (Source: Subramanya, 2008)

Further, the meter will register the velocity $v_R$. If $v_b$ is normal to $v_f$,

If the time of transit between two verticals is Δt, then the width between the two verticals (Fig. 21.14) is

$$W = v_b \Delta t$$

The flow in the sub-area between two verticals i and i + 1 where the depths are $y_i$ and $y_{i+1}$respectively, by assuming the current meter to measure the average velocity in the vertical, is

$$\Delta Q_i = \left(\frac{y_i + y_{i+1}}{2}\right) W_{i+1} v_f$$

i.e.

$$\Delta Q_i = \left(\frac{y_i + y_{i+1}}{2}\right) v_R^2 \cos\theta \sin\theta \, \Delta t \qquad (21.13)$$

Thus by measuring the depths $y_i$, velocity $v_R$and θ in a reach and the time taken to cross the reach Δt, the discharge in the sub-area can be determined. The summation of the partial discharges $\Delta Q_i$, over the whole width of the stream gives the stream discharges

$$Q = \sum \Delta Q_i \, \Delta t (21.14)$$

### 21.3.3 Chemical Gauging

Chemical gauging is also referred to as the dilution method and is especially useful in very small streams mountain streams strewn with boulders, etc. This method employes the conservation of mass of the tracer to be used. Common salt, fluorescent dyes and radioactive materials are the main tracers used. A tracer should be able to mix freely with flow; should not react with sediment, channel boundaries, or vegetation: and should not evaporate. The tracer of specified concentration $C_1$,is injected into the stream at a constant rate $Q_t$at a defined location. Samples are taken at a downstream point where the concentration gradually rises to a constant value $C_2$ as shown in Fig. 21.15.

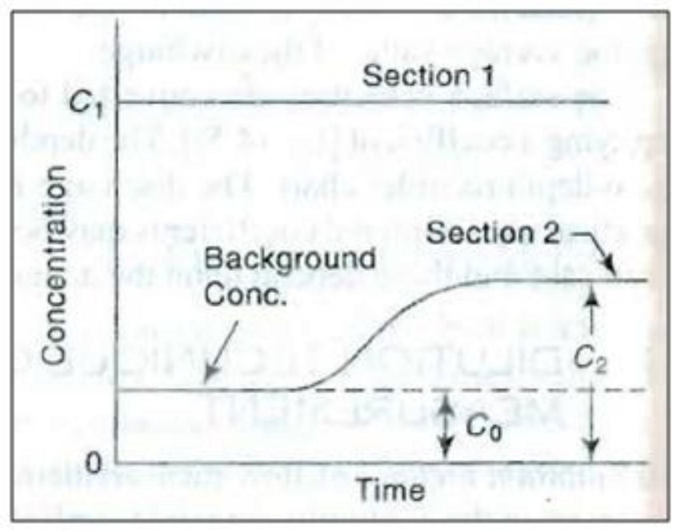


Fig. 21.15.Constant rate injection method. (Source: Subramanya, 2008)

Let $C_0$be the initial concentration of tracer in the stream. At steady state, the mass conservation for the tracer between the two locations can be expressed as

$$Q_t C_1 + Q C_0 = (Q + Q_t) C_2$$

i.e.

$$Q = \frac{Q_t (C_1 - C_2)}{(C_2 - C_0)} \quad (21.15)$$

This technique in which Q is estimated by knowing $C_1C_2$, $C_0$ and $Q_t$, is known as constant rate injection method or plateau gauging.

### 21.3.4 Ultrasonic Method

In this method (Fig. 21.16), ultrasonic signals are used to measure flow velocity. Two transducers are installed at the same elevation from the bed on both sides of the stream. The path connecting the transducers makes an .angle with the direction of flow. The transducers receive as well as transmit ultrasonic signals. The time taken by a signal from one transducer to the other is recorded. The average velocity of flow along the path is then obtained by knowing the path length and its angle with the flow direction. This method of gauging is suitable for automatic recording of data. It is accurate, efficient, and can accommodate rapid changes in the magnitude and direction of flow. However, unstable cross-sections, large loads of sediment in suspension, steep temperature changes, changing weed growth, air entrapment, changes in salinity, etc., can reduce accuracy of this method.

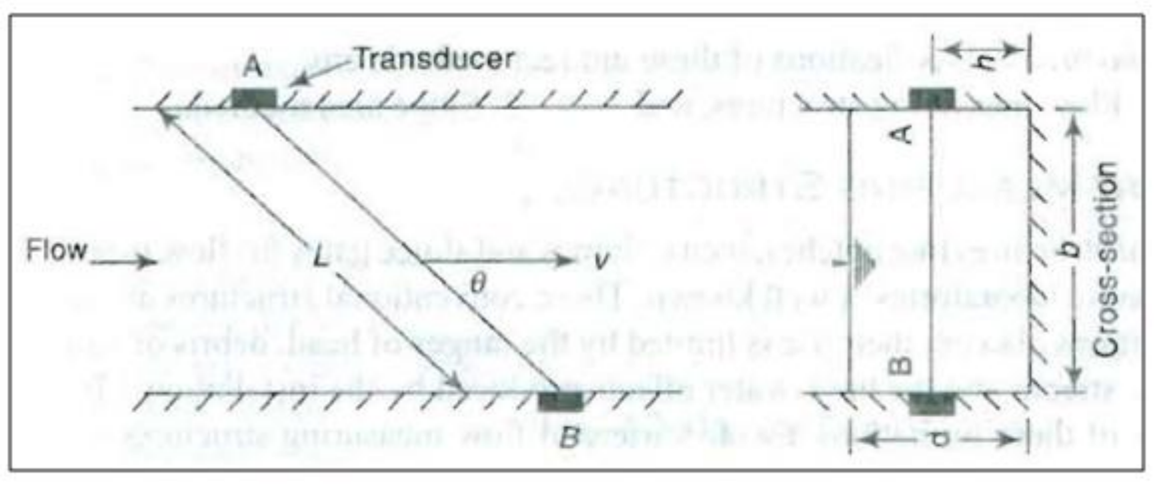


Fig. 21.16.Ultrasonic method. (Source: Subramanya, 2008)

### 21.3.5 Electromagnetic Method

The electromagnetic method is based on the Faraday's principle that an emf is induced in the conductor (water in the present case) when it cuts a normal magnetic field. Large coils buried at the bottom of the channel carry a current 1 to produce a controlled vertical magnetic field (Fig. 21.17)

Electrodes provided at the sides of the channel section measure tbe small voltage produced due to flow of water in the channd.1t has been found that the signal output E will be of the order of millivolts and is related to the discharge Q as

$$Q = K_1\left(\frac{Ed}{I} + K_2\right)^n \quad (21.16)$$

Whered is depth of flow, Iis current in the soil, and n, $K_1$and $K_2$ are system constants.

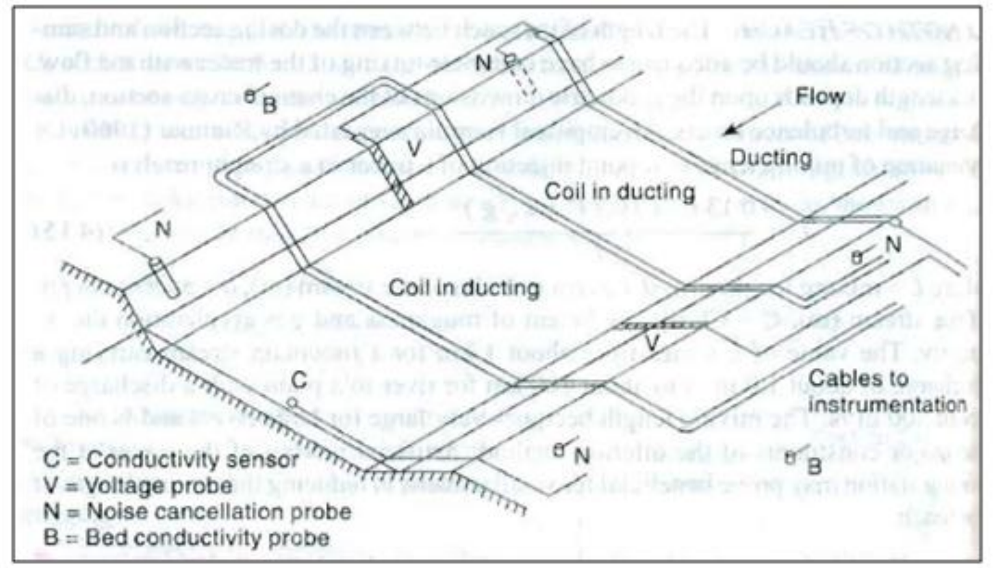


Fig. 21.17.Electromagnetic method. (Source: Subramanya, 2008)

Present day commercially available electromagnetic flowmeters can measure the discharge to an accuracy of ±3%, the maximum channel width that can be accommodated being 100 m. The minimum detectable velocity is 0.005 m/s.

### 21.3.6 Indirect Method

These methods measure flow depths at specified locations, and translate these depths into discharges using depth-discharge relations applicable to those locations. Flow-measuring structures and slope-area methods exemplify indirect methods.

Flow Measuring Structures

Different types of flow-measuring structures are in use. Broad-crested weirs, flumes made of concrete, masonry or metal sheets, and V-notches are common examples of such structures. When a flow-measuring structure is installed in a stream, it produces a unique control section in the flow. The discharge Q through the structure is related to the water-surface elevation H adjacent to or within the structure as

$Q = f(H)$ (21.17)

Where f is some function

For example, for a weir

$$Q = CBH^{x} (21.18)$$

in which B is the width of the weir crest, C is the discharge coefficient, and x is an exponent. Both C and x are specific for a weir, and are obtained by calibration.

The value of C takes into account the channel geometry, the friction loss due to the weir, horizontal and vertical contractions of flow, the form of the weir, etc.

Slope-area Method

This method is based on the principle of energy conservation. A stream reach is selected, as shown in Fig. 21.18. From Bernoulli’s equation applied to the ends of the reach (sections t and 2),

$$Z_1 + y_1 + \frac{V_1^2}{2g} = Z_2 + y_2 + \frac{V_2^2}{2g} + h_L \quad (21.19)$$

Where is head loss in the river reach.The head loss $h_L$can be considered to be made up of two parts (i) frictional loss $h_f$and (ii) eddy loss $h_e$.Denoting Z + Y = h = water surface elevation above the datum,

$$h_1 + \frac{V_1^2}{2g} = h_2 + \frac{V_2^2}{2g} + h_e + h_f$$

$$h_f = (h_1 - h_2) + \left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right) - h_e$$ (21.20)

If L is length of the reach, by Manning's formula for uniform flow,

$$\frac{h_f}{L} = S_f = \text{energy slope} = \frac{Q^2}{K^2}$$

In non-uniform flow an average conveyance is used to estimate the energy slope and

where K = conveyance of channel = $\frac{1}{n} AR^{1/3}$

n = Manning's coefficient

The eddy loss $h_e$ is estimated as

$$\frac{h_f}{L} = \bar{S}_f = \frac{Q^2}{K^2}$$ (21.21)

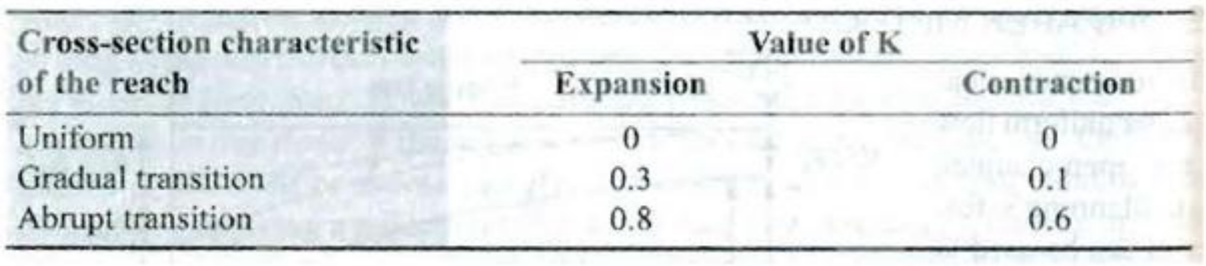
Where $K = \sqrt{K_1 K_2}\ ;\ K_1 = \frac{1}{n_1} A_1 R_1^{1/3} \text{and} K_2 = \frac{1}{n_2} A_2 R_2^{1/3}$

n= Manning's coefficient

The eddy loss $h_e$ is estimated as

$$h_e = K_e \left|\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right|$$

where $K_e$= eddy-loss coefficient having values as below.



| Cross-section characteristic of the reach | Value of K | |
|---|---|---|
| | Expansion | Contraction |
| Uniform | 0 | 0 |
| Gradual transition | 0.3 | 0.1 |
| Abrupt transition | 0.8 | 0.6 |

Equation (21.20), (21.21) and (21.22) together with the continuity equation $Q = A_1V_1 = A_2V_2$ enable the discharge Q to be estimated for known values of h,channel cross-sectional properties and n.

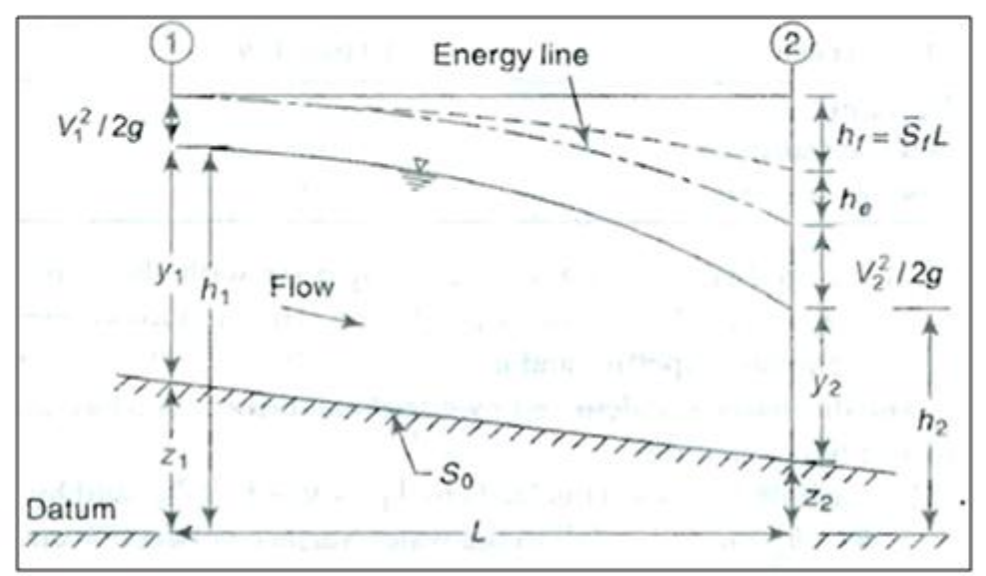


Fig. 21.18.Slope-area method. (Source: Subramanya, 2008)

### 21.4 Rating Curve

A stage-discharge relation for a velocity-area station or gauging section is obtained by plotting measured stage on the ordinate and the measured discharge on the abscissa as shown in

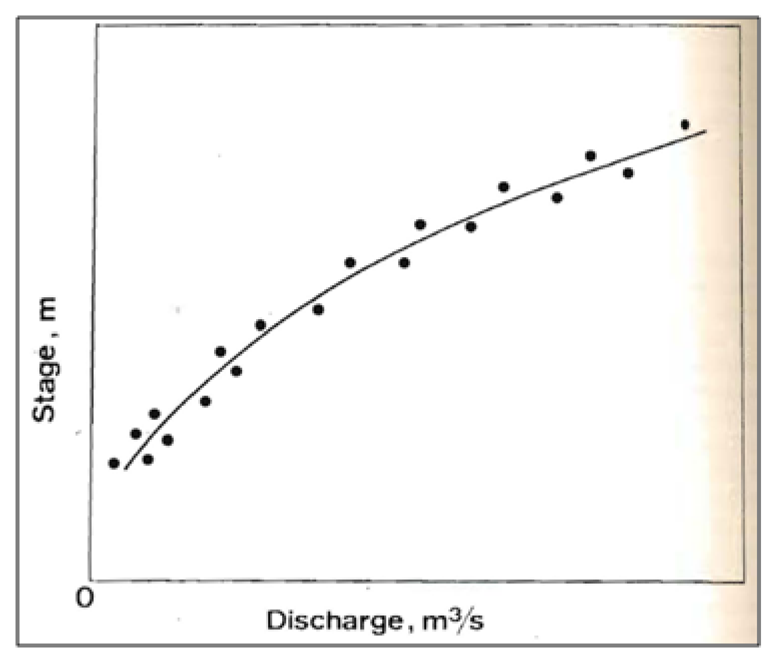


Fig. 21.19. This relation is also called the rating curve.

Fig. 21.19.Stage-discharge relationship.(Source: Singh, 1994)

It represents the integrated effect of a wide range of channel and flow parameters. The combined effect of these parameters is designated as control. If the rating curve for a gauging station does not change with time the control is called permanent; otherwise, 'it is called shifting control. The stage-discharge relation is of fundamental importance in the acquisition of discharge measurements. All direct discharge-measuring methods require construction of this relationship before measured stages can he translated into' discharges.

### 21.4.1 Simple Rating Curve

A majority of streams and rivers, especially non-alluvial rivers exhibit permanent control. For such a case, the relationship between the stage and the discharge is a single valued relation which is expressed as

$$Q = C_r (G - a)^{\beta} \quad (21.23)$$

WhereQ is stream discharge, G is gauge height (stage), ais a constant representing the gage reading for zero discharge, and and  are rating curve constants.

Equation 21.20 is called the rating equation of stream and can be used for estimating the discharge Q of the stream for a given gauge reading G within range of data used in its derivation.

When the data are plotted on logarithmic paper the plot is a straight line, as shown in Fig. 21.20.Equation 21.23becomes

$$\log Q = \log C_r + \beta \log(G - a) \quad (21.24)$$

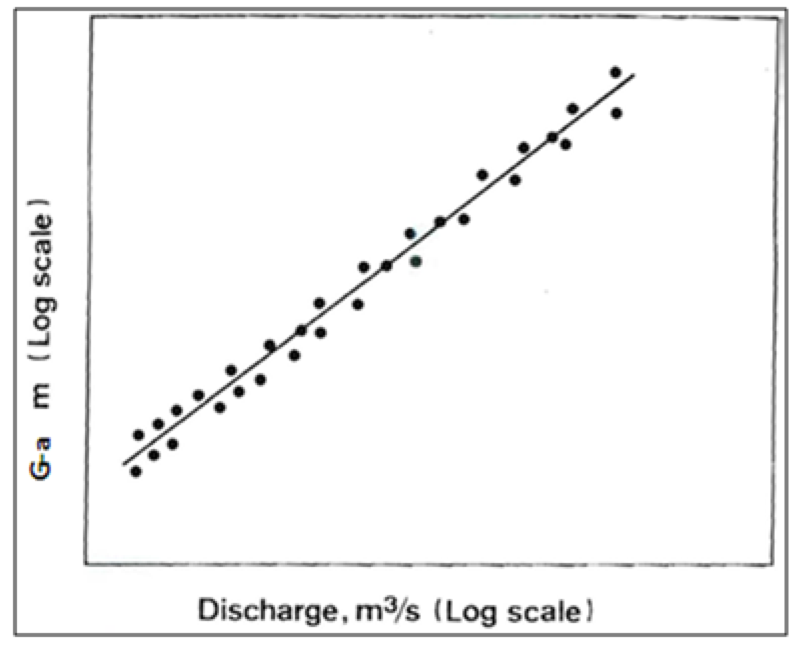


Fig. 21.20.Stage-discharge relationship on logarithmic paper.

(Source: Singh, 1994)

The best values of constants and can be obtained using the least square, method.

1. Constant a must be found beforehand, and this can be estimated using following methods.
2. Trial and error method.
3. To extrapolate the rating curve corresponding to Q = 0 and then plot log Q versus log (G-a).
4. Analytical method.

The simple rating curve is generally satisfactory for a majority of streams where rapid fluctuations of stage are not experienced at the gauging section. The adequacy of the curve is measured by the scatter of data around the fitted curve. When there is a permanent control, the rating curve is essentially permanent. If the rating curve is made using a range of stages from low to high, it can be used to interpolate the discharge for any stage of flow between the measured stages without measuring that flow. It is important to check the stability of the curve by periodic discharge measurements, and to extend it with each new observed high stage. Changes in channel shape, due to scouring or sedimentation, can change the effect of control and thereby change the rating curve.

### 21.4.2 Shifting Control

When the control of a gauging station changes, its rating curve changes. The change may result from (1) scour or deposition, (2) varying back water, (3) rapidly changing flow, and (4) changes in flow caused by dredging, channel encroachment and weed growth, etc. For the shifting control due to cases (1) and (4), frequent current meter gauging is needed. The discharge is then estimated by noting the difference between the stage at the time of a discharge measurement and the stage obtained from the rating curve corresponding to the same discharge. This difference serves as a correction and is applied to all stages before reading the rating curve. If this correction varies from one measurement to another, it can be assumed to vary linearly in time. The effect of backwater and unsteady flow is amenable to analytical treatment.

### 21.4.3 Constant-fall Rating Curve

The backwater may develop as a result of an obstruction downstream or high stages in an intersection stream, and may be variable in time. Under the condition of shifting control due to backwater effects, a given stage will indicate different discharge values. This results from differences in water-surface slope at the control. The discharge is estimated by establishing another gage some distance downstream of the main gauging station. This other gage is called the secondary, or auxiliary, gage. Measurements are made at both gages.

The difference between stages of these gages indicates the fall (F) or slope (S = F/L) of the water surface in the reach.

The discharge then is a function of stage G (at the main gage) and fall F, and is expressed as

$$\frac{Q}{Q_0} = \left(\frac{S}{S_0}\right)^k = \left(\frac{F}{F_0}\right)^m \quad (21.25)$$

Whereis the normalized discharge at the stage when  = or , and k and m are exponents.

This method is called the slope-stage-discharge rating-curve method.

All observed values of stage and discharge for values of F = $F_0$ are plotted as a simple rating curve, as shown in Fig. 21.21. This is the $Q_0$ versus G curve and is called the constant-fall curve.

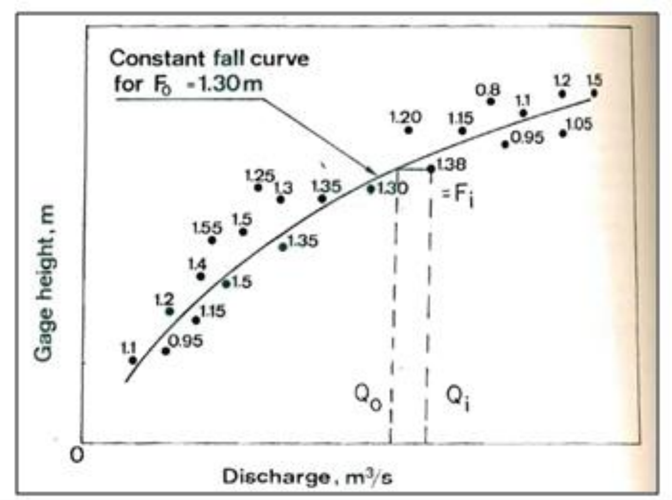


Fig. 21.21.Constant-fall rating curve.(Source: Singh, 1994)

For each measurement with F ≠$F_0$, values of $Q/Q_0$and $F/F_0$are calculated and plotted as shown in Fig. 21.22. The plot of $Q/Q_0$versus $F/F_0$is called the auxiliary curve or adjustment curve.

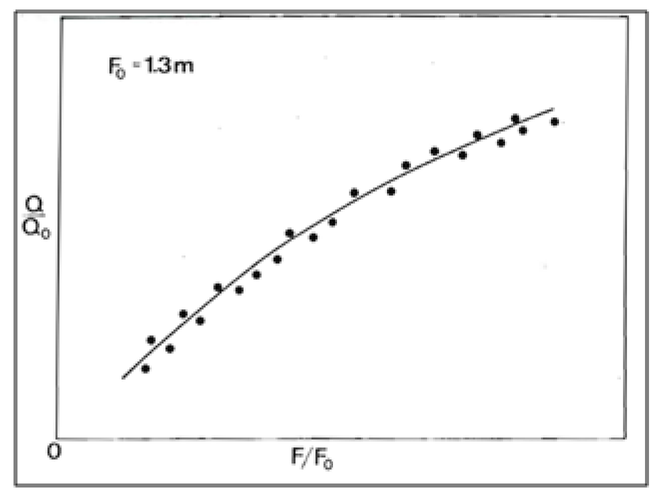


Fig. 21.22.Adjustment curve for constant-fall rating curve.

(Source: Singh, 1994)

### 21.4.4 Normal-fall Rating Curve

Under some conditions, Fvaries widely and its variation is related to stage. Then a normal fall, $F_n$, is defined as a function of stage, which replaces $F_0$in Eq. (21.25), as shown in Fig. 21.23. This is a second auxiliary curve. Correspondingly, normal Row $Q_n$replaces $Q_0$.

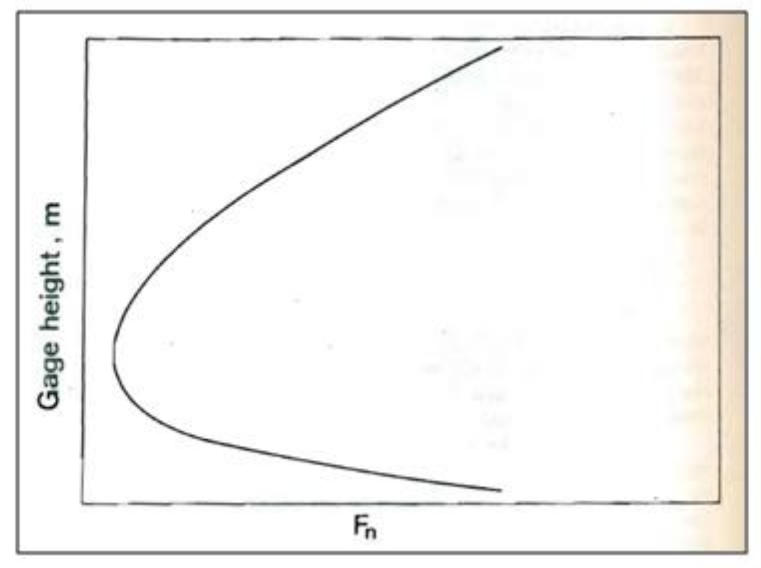


Fig. 21.23.Normal-fall rating curve.(Source: Singh, 1994)

Correspondingly, normal flow$Q_n$replaces$Q_0$. Analogous to a constant-fall rating, a normal fall rating is employed, as shown in Fig. 21.24.

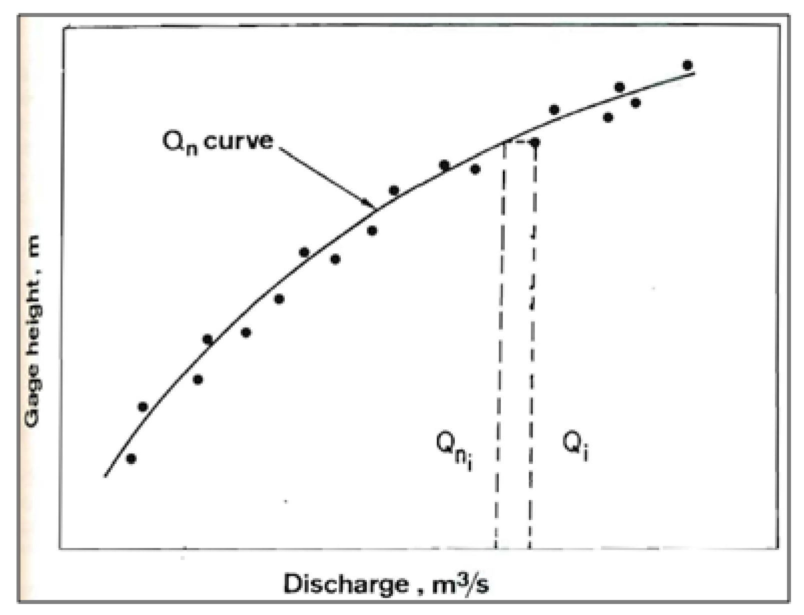


Fig. 21.24.$Q_n$ for normal-fall rating curve.(Source: Singh, 1994)

In the same vein, the auxiliary curve is obtained by plotting $Q/Q_n$versus $F/F_n$as shown in Fig. 21.25.

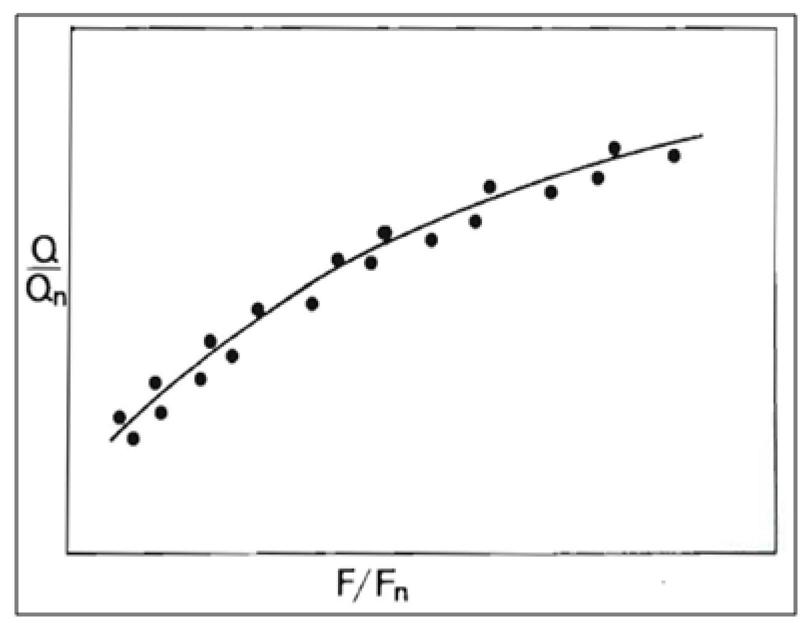


Fig. 21.25.Auxiliary curve for normal-fall rating curve.(Source: Singh, 1994)

For a given stage, the value of actual fall F is computed first. Then a value of $F_n$is obtained from the second auxiliary curve, and the ratio $F/F_n$is computed. Against this value of $F/F_n$, the value of $Q/Q_n$is obtained from the first auxiliary curve. The value of $Q_n$is read from the raring curve, which then is multiplied by $Q/Q_n$to yield the actual value of discharge, that is, $Q = (Q/Q_n) \times Q_n$.

### 21.4.5 Loop Rating Curve

During the passage of a flood past a gauging section, the stage-discharge relation for the rising limb of the hydrograph is different from the one for the falling limb. During the rise of flood, the velocity and discharge are greater for a given stage than they are for the same stage when the flow is steady and uniform. During the falling stage, the reverse is true. Thus, the stage-discharge relation for unsteady flows is a loop, as shown in Fig. 21.26.

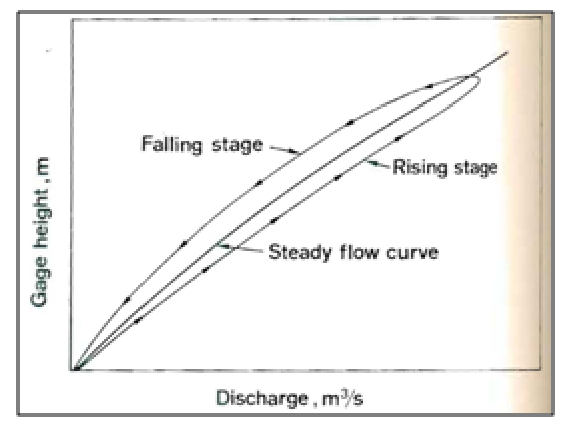


Fig. 21.26.Stage-discharge relationship for unsteady flow.(Source: Singh, 1994)

It may be seen that for the same stage, there is greater discharge through the stream reach during the rising stage than it is during the falling stage. This loop may be plotted from stage and discharge measurements of a flood. This curve can be used as an approximation for other floods of about the same magnitude and duration.

For floods with multiple peaks, the loop may be complicated. Thus, a better procedure is to relate for the same stage the normal discharge, $Q_N$under steady uniform flow to the measured unsteady-flow discharge $Q_M$ (Chow. 1959) as

$$\frac{Q_M}{Q_N} = \left(1 + \frac{1}{V_c S_0} \frac{dG}{dt}\right)^{0.5} \quad (21.26)$$

Whereis the velocity of the flood wave or wave celerity;  is the channel slope, the slope of the water surface for uniform flow, and dG/dtis the rate of change of the stage.

Module 6. Hydrograph

## Lesson 22 Stream Flow Hydrograph

### 22.1 Definition of Hydrograph

The hydrograph which results due to an isolated storm is typically singlepeaked skew distribution of discharge and is known variously as storm hydrograph, flood hydrograph or simply hydrograph.

The hydrograph is the response of a given catchment to a rainfall input. It consists of flow in all the three phases of runoff, viz. surface runoff, interflow and base flow and embodies in itself the integrated effects of a wide variety of catchment and rainfall parameters having complex interactions.

### 22.2 Elements of Hydrograph

Hydrograph has three characteristic regions: (i) the rising limb AB, joining point A, the starting point of the rising curve and point B, the point of inflection, (ii) the crest segment BC between the two points of inflection with a peak P in between, (iii) the falling limb or depletion curve CD starting from the second point of inflection C (Fig. 22.1).

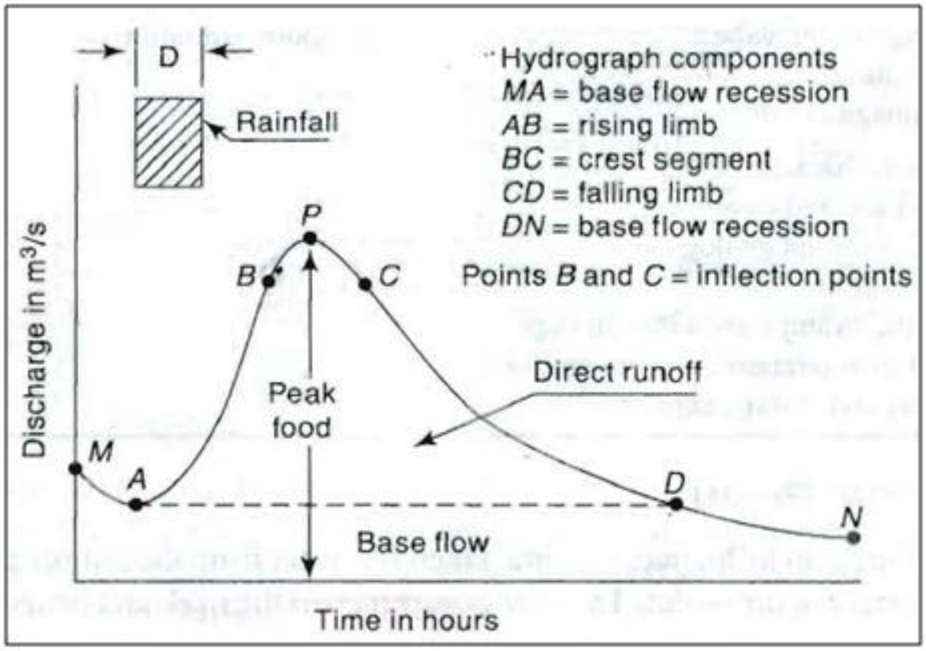


Fig. 22.1.Elements of a hydrograph. (Source: Subramanya, 2008)

### 22.2.1 Rising Limb

The rising limb f a hydrograph, also known as concentration curve represents the increase in discharge due to the gradual building up of storage in channel and over the catchment surface. The initial losses and high infiltration losses during the early period of a storm cause the discharge to rise rather slowly in the initial periods. The basin and storm characteristics control the shape of the rising limb of a hydrograph.

### 22.2.2 Crest Segment

The crest segment is one of the most important parts of hydrograph as it contains the peak flow. The peak now occurs when the runoff from various parts of the catchment simultaneously contribute amounts to achieve the maximum amount of flow at the basin outlet. Generally for large catchments, the peakflow occurs after the cessation of rainfall, the time interval from the centre of mass of rainfall to the peak being essentially controlled by basin and storm characteristics. Multiple-peaked complex hydrographs in a basin can occur when two or more storms occur in succession. Estimation of the peak flow and its occurrence, being important in flood-flow studies are dealt with in detail elsewhere in this book.

### 22.2.3 Recession Limb

The recession limb, which extends from the point of inflection at the end of the crest segment (point C in Fig. 22.1) to the commencement of the natural groundwater flow (point D in Fig. 22.1) represents the withdrawal of water from the storage built up in the basin during the earlier phase of the hydrograph. The starting point of the recession limb, i.e. the point of inflection represents the condition of maximum storage. Since the depletion of storage takes place after the cessation of rainfall, the shape of this part of the hydrograph is independent of storm characteristics and depends entirely on the basin characteristics.

### Stream flow Recession

The storage of water in the basin exists as (i) surface storage, which includes both surface detention and channel storage, (ii) interflow storage, and (iii) groundwater storage, i.e. base-flow storage.

Barnes (1940) showed that the recession of storage can be expressed as

$$Q_t = Q_0 K_r^t \quad (22.1)$$

Where Q is the discharge at a time t and $Q_0$is the discharge at t= 0; $K_r$is a recession constant of value less than unity.

Equation 22.1 can also be expressed in an alternative form of the exponential decay as

$$Q_t = Q_0 e^{-at} \quad (22.1a)$$

Where a = $-\ln K_r$

### Example 1

On June 1, 1980 the discharge in a stream was measured as 80 $m^3/s$. Another measurement on June 21, 1980 yielded the stream discharge as 40 $m^3/s$. There was no rainfall in the catchment from April 15, 1980. Estimate the recession coefficient.

### Answer:

Given = 40 $m^3/s$, = 80 $m^3/s$, t = 21-1 = 20

From Eq. 22.1

$$Q_t = Q_0 K_r^t$$

$$\log(Q_t/Q_0) = t\,logK_r$$

$$logK_r = (\log(Q_t/Q_0))/t$$

$$logK_r = (\log(40/80))/20$$

$$logK_r = -0.015051499$$

$$\boldsymbol{K_r = 0.965} \qquad \ldots\ldots\ldots(\boldsymbol{recession\ coeffcient})$$

The time characteristics of hydrograph (Fig. 22.2) are described below:

### Time Base of Hydrograph ($T_B$)

It is the time from the beginning to the end of the direct runoff.

### Lag Time ($T_L$)

It is the difference in time between the center of mass of net rainfall and center of mass runoff.

### Time to Peak ($T_P$)

It is the time difference between the beginnings of direct runoff (point B in Fig. 22.2) to peak.

### Rainfall Duration ($T_r$)

It is the effective rainfall duration, which causes direct runoff. Curve between point M and A represents recession from previous storm.

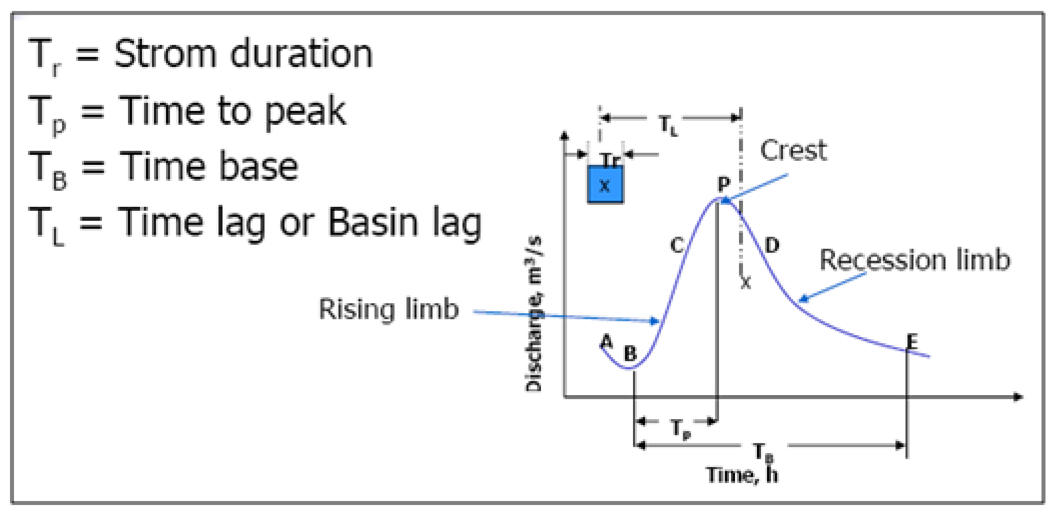


Fig. 22.2.Hydrograph time characteristics. (Source: Subramanya, 2008)

## 22.3 Factors Affecting Hydrograph

| **Physiographic factors** | **Climatic factors** |
|---|---|
| 1. Basin characteristics<br>(a) Shape<br>(b) Size<br>(c) Slope<br>(d) Nature of the valley<br>(e) Elevation<br>(f) Drainage density<br>2. Infiltration characteristics<br>(a) Land use and cover<br>(b) Soil type and geological conditions<br>(c) Lakes, swamps and other storage<br>3. Channel characteristics: cross-section, roughness and storage capacity | 1. Storm characteristics: precipitation, intensity, duration, magnitude and movement of storm<br>2. Initial loss<br>3. Evapotranspiration |

# Lesson 23 Base Flow Separation

## 23.1 Methods of Base Flow Separation

The surface-flow hydrograph is obtained from the total storm hydrograph by separating the quick-response flow from the slow response runoff. It is usual to consider the interflow as a part of the surface flow in view of its quick response. Thus only the base flow is to be deducted from the total storm hydrograph to obtain the surface flow hydrograph.

There are three methods of base-flow separation that are in common use.

### 23.1.1. Method 1

In this method the separation of the base flow is achieved by joining with a straight line the beginning of the surface runoff to a point on the recession limb representing the end of the direct runoff.

In Fig. 23.1, pointA represents the beginning ofthe direct runoff off and it is usually easy to identify in view of the sharp change in the runoff rate at that point. Point B, marking the end of the direct runoff is rather difficult to locate exactly.

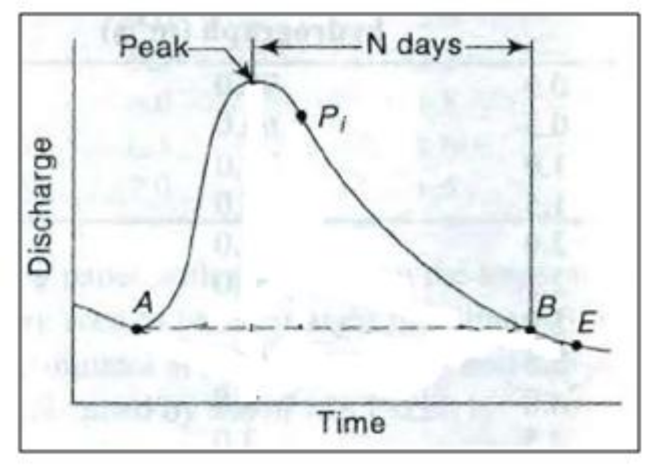


Fig. 23.1. Method 1 for base flow separation.(Source:Subramanya, 2008)

An empirical equation for the time interval N (days) from the peak to the point B is

$$N = 0.83\,A^{0.2} \quad (23.1)$$

WhereA is drainage area in km²and N is in days. Points A and B are joined by a straight line to demarcate to the base flow and surface runoff. This method of base-flow separation is the simplest of all the three methods.

### 23.1.2 Method 2

In this method the base flow curve existing prior to the commencement of the surface runoff is extended till it intersects the ordinate drawn at the peak (point C in Fig. 23.2). This point is joined to point B by a straight line. Segment AC and CB demarcate the base flow and surface runoff. This is probably the most widely used base-flow separation procedure.

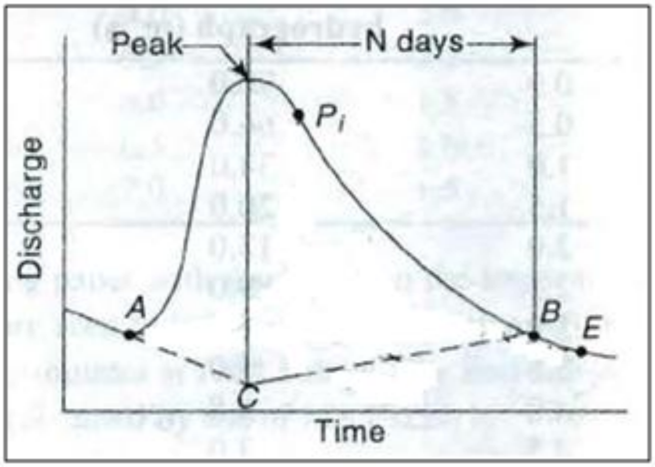


Fig. 23.2. Method 2 for base flow separation. (Source:Subramanya, 2008)

### 23.1.3 Method 3

In this method the base flow recession curve after the depletion of the flood water is extended backwards till it intersects the ordinate at the point of inflection (line EF in Fig. 23.3). Points A and F are joined by an arbitrary smooth curve. This method of base-flow separation is realistic in situations where the groundwater contributions are significant and reach the stream quickly.

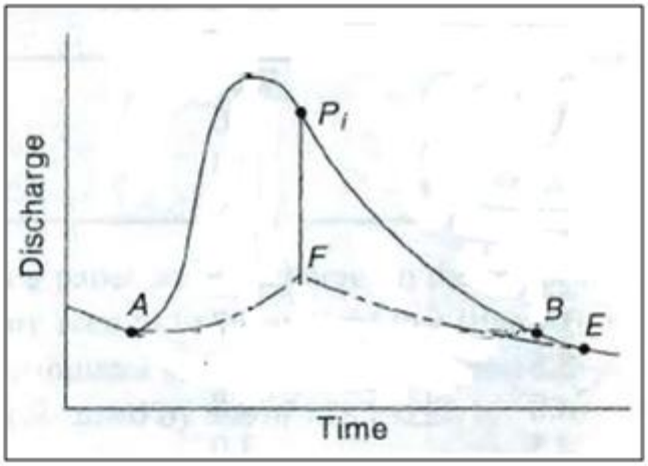


Fig. 23.3. Method 3 for base flow separation. (Source:Subramanya, 2008)

The surface runoff hydrograph obtained after the base-flow separation is also known as direct runoff hydrograph (DRH).

Example 1

The following are the ordinates of the hydrograph of flow from a catchment area of 770 km² due to a 6-h rainfall. Derive the ordinates of DRH. Make suitable assumptions regarding the base flow.

| Time from beginning of storm | (h) | 0 | 6 | 12 | 18 | 24 | 30 | 36 |
|---|---|---|---|---|---|---|---|---|
| Discharge | (m³/s) | 42 | 65 | 215 | 360 | 400 | 350 | 270 |
| Time from beginning of storm | (h) | 42 | 48 | 54 | 60 | 66 | 72 | |
| Discharge | (m³/s) | 205 | 145 | 100 | 70 | 50 | 42 | |

Answer:

Given: catchment area (A) = 770 km²

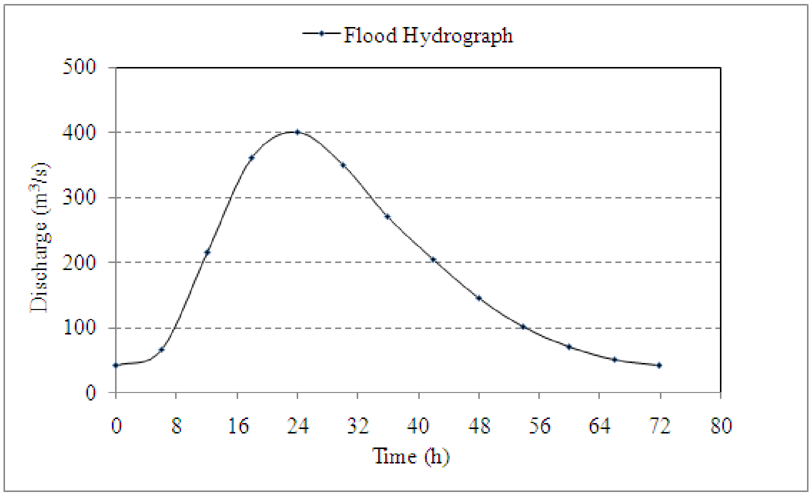
Using equation 23.1,

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

$N = 0.83\,(770)^{0.2}$

$N = 3.13 \text{ day}$

$N = 75.26 \text{ h} \qquad \textit{from peak}$



From given data, with our convenience, base flow = 42 $m^3/s$ at 72 h

Therefore, DRH = Flood Hydrograph – Base flow

| Time from beginning of storm | Discharge | Base flow | DRH |
|---|---|---|---|
| h | $m^3/s$ | $m^3/s$ | $m^3/s$ |
| 0 | 40 | 42 | -2 |
| 6 | 65 | 42 | 23 |
| 12 | 215 | 42 | 173 |
| 18 | 360 | 42 | 318 |
| 24 | 400 | 42 | 358 |
| 30 | 350 | 42 | 308 |
| 36 | 270 | 42 | 228 |
| 42 | 205 | 42 | 163 |
| 48 | 145 | 42 | 103 |
| 54 | 100 | 42 | 58 |
| 60 | 70 | 42 | 28 |
| 66 | 50 | 42 | 8 |
| 72 | 42 | 42 | 0 |

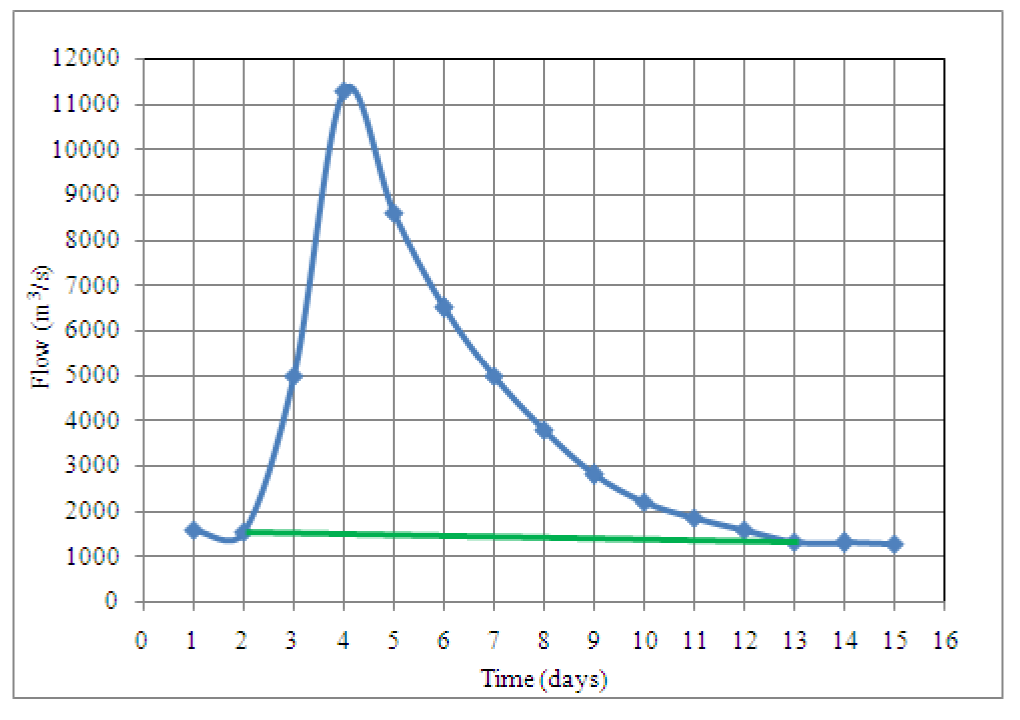
Example 2

The daily stream flow data at a site having a drainage area of 6500 km$^2$ are given in the following table. Separate the base flow using the above three methods.

| Time (days) | Discharge (m3/s) |
|---|---|
| 1 | 1600 |
| 2 | 1550 |
| 3 | 5000 |
| 4 | 11300 |
| 5 | 8600 |
| 6 | 6500 |
| 7 | 5000 |
| 8 | 3800 |
| 9 | 2800 |
| 10 | 2200 |
| 11 | 1850 |
| 12 | 1600 |
| 13 | 1330 |
| 14 | 1300 |
| 15 | 1280 |

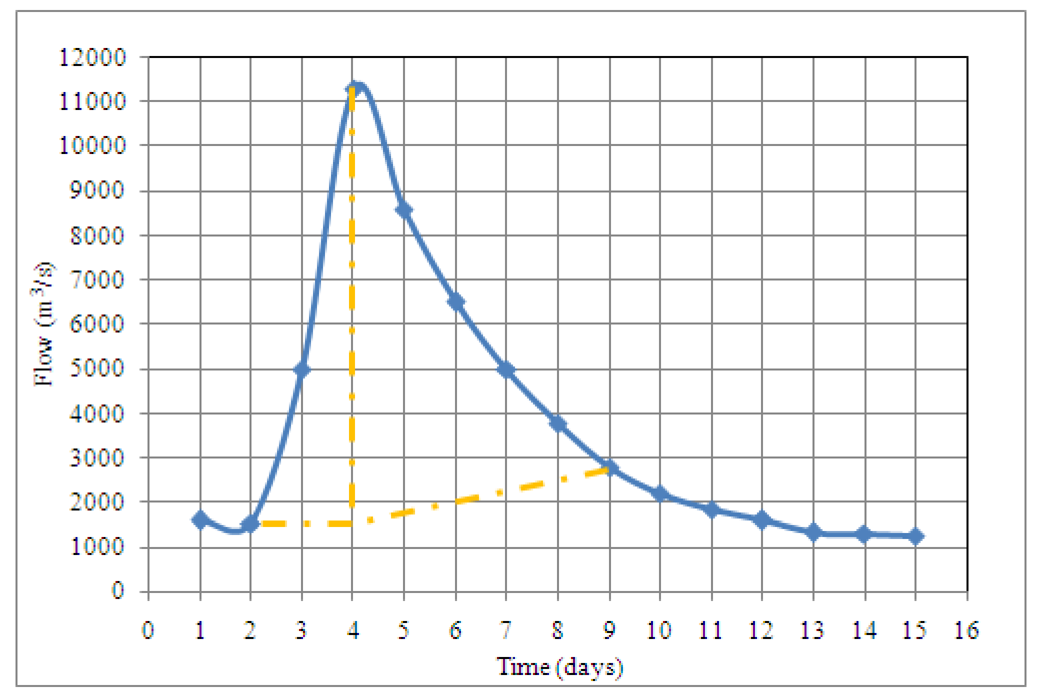
Answer

1. Plot the total runoff hydrographMethod 1: join point A, the beginning of direct runoff, to point B, the end of direct runoff. Both points are selected by judgment.

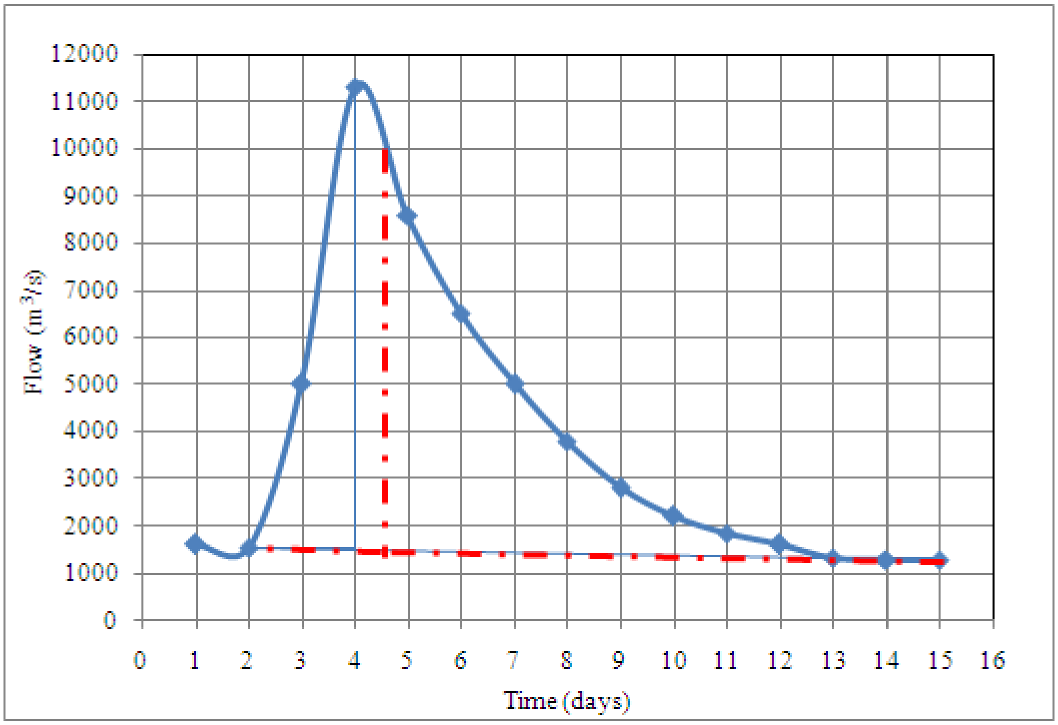


2. Method 2: Extend the recession curve before the storm up to point C below the peak. Join point C to D, computed using equation

$$N = 0.83\,A^{0.2} = 0.83\,(6500)^{0.2} = 4.6 \text{ days(Approx. 5 days)}$$



3. Method 3: Extend the recession curve backward to point E. Join point E to A

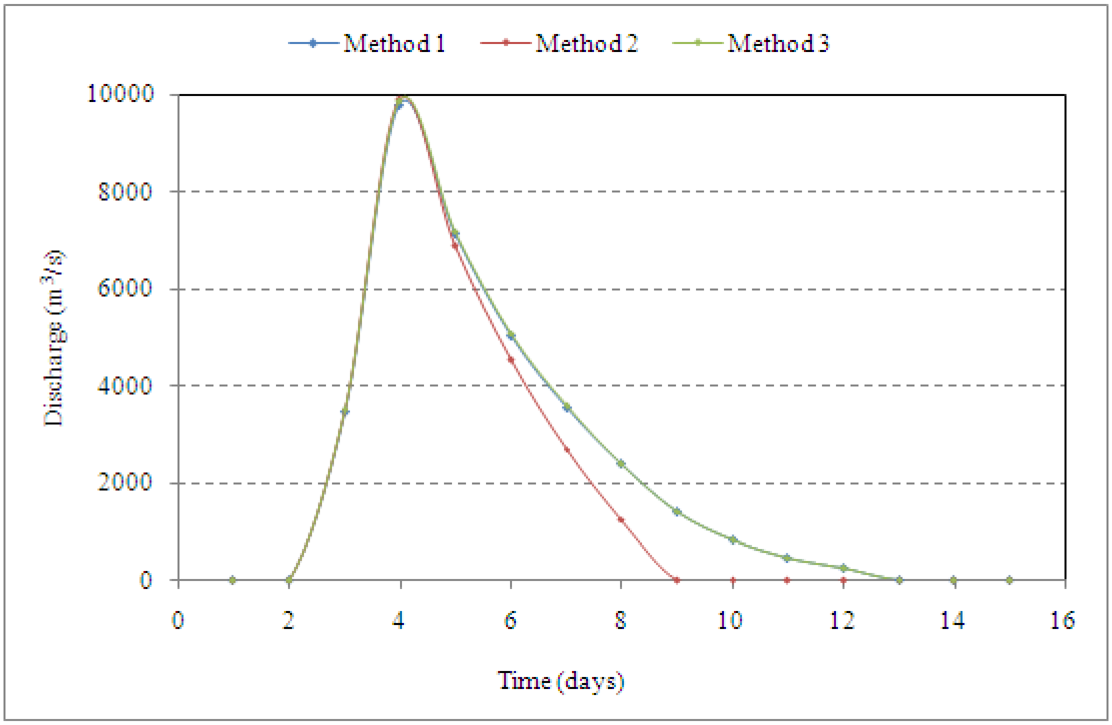


4. The ordinates DRH by three methods are given in Table

Table Ordinates of DRH by different methods

| Time | Total runoff | Base flow | | | Direct runoff | | |
|---|---|---|---|---|---|---|---|
| | | Method 1 | Method 2 | Method 3 | Method 1 | Method 2 | Method 3 |
| (days) | (m$^3$/s) | (m$^3$/s) | (m$^3$/s) | (m$^3$/s) | (m$^3$/s) | (m$^3$/s) | (m$^3$/s) |
| 1 | 1600 | 1600 | 1600 | 1600 | 0 | 0 | 0 |
| 2 | 1550 | 1550 | 1550 | 1550 | 0 | 0 | 0 |
| 3 | 5000 | 1520 | 1480 | 1500 | 3480 | 3520 | 3500 |
| 4 | 11300 | 1500 | 1400 | 1450 | 9800 | 9900 | 9850 |
| 5 | 8600 | 1450 | 1700 | 1400 | 7150 | 6900 | 7200 |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| 6 | 6500 | 1450 | 1950 | 1400 | 5050 | 4550 | 5100 |
| 7 | 5000 | 1450 | 2300 | 1400 | 3550 | 2700 | 3600 |
| 8 | 3800 | 1400 | 2550 | 1400 | 2400 | 1250 | 2400 |
| 9 | 2800 | 1380 | 2800 | 1380 | 1420 | 0 | 1420 |
| 10 | 2200 | 1380 | 2200 | 1380 | 820 | 0 | 820 |
| 11 | 1850 | 1380 | 1850 | 1380 | 470 | 0 | 470 |
| 12 | 1600 | 1350 | 1600 | 1350 | 250 | 0 | 250 |
| 13 | 1330 | 1330 | 1330 | 1330 | 0 | 0 | 0 |
| 14 | 1300 | 1300 | 1300 | 1300 | 0 | 0 | 0 |
| 15 | 1280 | 1280 | 1280 | 1280 | 0 | 0 | 0 |



## 23.2 Effective Rainfall Hyetograph

Effective rainfall (also known as Excess rainfall) (ER) is that part of the rainfall that becomes direct runoff at the outlet of the watershed. It is thus the total rainfall in a given duration from which abstractions such as infiltration and initial losses are subtracted.

For purposes of correlating DRH with the rainfall which produced the flow, the hyetograph of the rainfall is also pruned by deducting the losses. Figure 23.4 shows the hyetograph of a storm. The initial loss and infiltration losses are subtracted from it. The resulting hyetograph is known as effective rainfall hyetograph (ERH). It is also known as excess rainfall hyetograph.

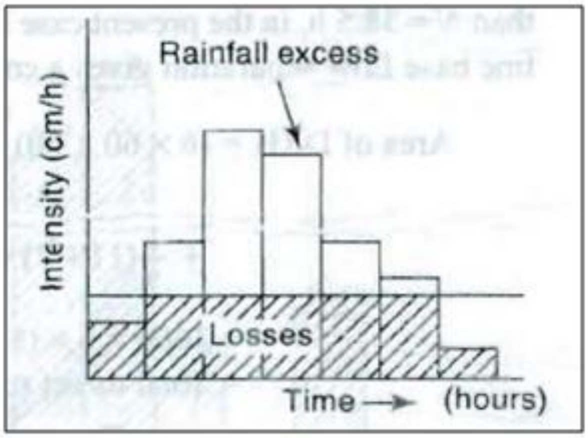


Fig. 23.4.Effective rainfall hyetograph. (Source:Subramanya, 2008)

Both DRH and ERH represent the same total quantity but in different units. Since ERH is usually in cm/h plotted agains1 time, the area of ERH multiplied by the catchment area gives the total volume of direct runoff which is the same as the area of DRH. Theinitial loss and infiltration losses are estimated based on the available data of the catchment.

Example 3

A 4-hour storm occurs over an 80 km² watershed. The details of the catchment are as follows:

| Sub Area | Φ index | Hourly rain (mm) | | | |
|---|---|---|---|---|---|
| km$^2$ | mm/h | 1st hour | 2nd hour | 3rd hour | 4th hour |
| 15 | 10 | 16 | 48 | 22 | 10 |
| 25 | 15 | 16 | 42 | 20 | 8 |
| 35 | 21 | 12 | 40 | 18 | 6 |
| 5 | 16 | 15 | 42 | 18 | 8 |

Calculate the runoff from catchment and the hourly distribution of the effective rainfall whole catchment.

Answer:

$$= \frac{(16-10)\times 15 + (16-15)\times 25 + (12-21)\times 35 + (15-16)\times 5}{15+25+35+5}$$

$$= \frac{6\times 15 + 1\times 25 + 0\times 35 + 0\times 5}{80}$$

$$= 1.4375 \text{ mm}$$

2nd hour,

$$= \frac{(48-10)\times 15 + (42-15)\times 25 + (40-21)\times 35 + (42-16)\times 5}{15+25+35+5}$$

$$= \frac{38\times 15 + 27\times 25 + 19\times 35 + 24\times 5}{80}$$

$$= 25.375 \text{ mm}$$

3rd hour,

$$= \frac{(22-10)\times 15 + (20-15)\times 25 + (18-21)\times 35 + (18-16)\times 5}{15+25+35+5}$$

$$= \frac{12\times 15 + 5\times 25 + 0\times 35 + 2\times 5}{80}$$

$$= 3.9375 \text{ mm}$$

4th hour,

$$= \frac{(10-10)\times 15 + (8-15)\times 25 + (6-21)\times 35 + (8-16)\times 5}{15+25+35+5}$$

$$= \frac{0\times 15 + 0\times 25 + 0\times 35 + 0\times 5}{80}$$

$$= 0 \text{ mm}$$

$$\text{Total runoff} = 1.4375 + 25.375 + 3.9375 = 30.75 \text{ mm}$$

$$\text{Total runoff} = \left(\frac{30.75}{1000}\right)\times 80 \times 10^{6}$$

Totalrunoff = 2.46Mm$^3$

Hourly distribution of the effective rainfall for the whole catchment:

| | Effective rainfall (mm) |
|---|---|
| 1st hour | 1.4375 |
| 2nd hour | 25.375 |
| 3rd hour | 0 |
| 4th hour | 3.9375 |

Example 4

A storm in a certain catchment had three successive 6-h intervals of rainfall magnitude of 3.0 cm, 5.0 cm and 4.0 cm, respectively. The flood hydrograph at the outlet of the catchment resulting from this storm is as follows:

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Time | (h) | 0 | 6 | 12 | 18 | 24 | 30 | 36 | 42 |
| Flood hydrograph ordinates | ($m^3/s$) | 30 | 480 | 2060 | 4450 | 6010 | 6010 | 5080 | 3996 |
| Time | (h) | 48 | 54 | 60 | 66 | 72 | 78 | | |
| Flood hydrograph ordinates | ($m^3/s$) | 2866 | 1866 | 1060 | 500 | 170 | 30 | | |

If the area of the catchment is 8791.2 $km^2$, estimate the index of the storm. Assume the base flow as 30 $m^3/s$.

Answer

Flood hydrograph ordinates = DRH ordinates +Base flow ordinates

Direct runoff(cm) = $$0.36\frac{\left(\sum_{i=1}^{N} DRO_i\right)\Delta t}{A}$$

Where

is direct runoff ordinates ($m^3/s$), is time interval between successive ordinates (h), A is catchment area ($km^2$)

| Time | Flood hydrograph Ordinates | Base flow | DRO |
|---|---|---|---|
| h | $m^3/s$ | $m^3/s$ | $m^3/s$ |
| 0 | 30 | 30 | 0 |
| 6 | 480 | 30 | 450 |
| 12 | 2060 | 30 | 2030 |

| | | | |
|---|---|---|---|
| 18 | 4450 | 30 | 4420 |
| 24 | 6010 | 30 | 5980 |
| 30 | 6010 | 30 | 5980 |
| 36 | 5080 | 30 | 5050 |
| 42 | 3996 | 30 | 3966 |
| 48 | 2866 | 30 | 2836 |
| 54 | 1866 | 30 | 1836 |
| 60 | 1060 | 30 | 1030 |
| 66 | 500 | 30 | 470 |
| 72 | 170 | 30 | 140 |
| 78 | 30 | 30 | 0 |
| | | = 34188 | |

Therefore,

Direct runoff (cm) = $0.36 \times \frac{34188 \times 6}{8791.2}$

Directrunoff (cm) = 8.4

Therefore

$(3 - 6\varphi) + (5 - 6\varphi) + (4 - 6\varphi) = 8.4$

$\varphi$ = 0.2cm/h

| | | | |
|---|---|---|---|
| Rainfall (cm) | 3 | 5 | 4 |
| Time interval (h) | 6 | 6 | 6 |
| Rainfall intensity (cm/h) | 0.5 | 0.833 | 0.667 |
| index (cm/h) | 0.2 | 0.2 | 0.2 |
| Excess rainfall intensity | 0.3 | 0.633 | 0.467 |

## 23.3 Elemental Hydrograph

If a small, impervious area is subjected to a constant rate rainfall, the resulting runoff hydrograph will appear much as above, and is known as elemental hydrograph (Fig. 23.5). In the beginning, there will be surface detention (rainfall-runoff) so as to start the sheet flow over the surface. At point B, known as point of equilibrium, outflow rate equals inflow rate. When rainfall ends (at C), recession starts, i.e., outflow rate and detention volume increases.

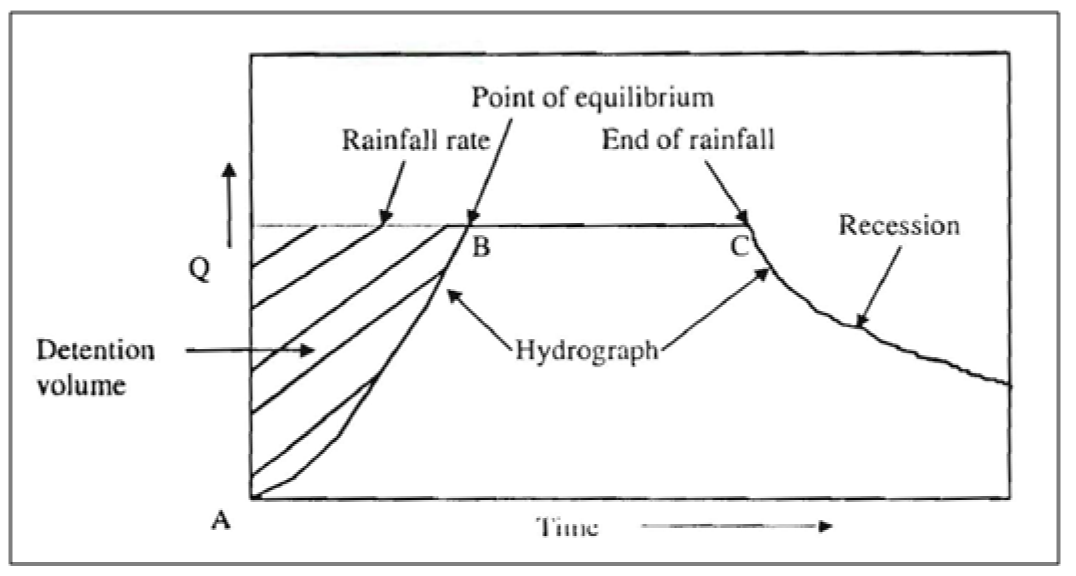


Fig. 23.5.Elemental hydrograph. (Source:Singh,1994)

## Lesson 24 Unit Hydrograph

### 24.1 Definition of Unit Hydrograph

This method was first suggested by Sherman in 1932

A unit hydrograph is defined as the hydrograph of direct runoff resulting from one unit depth (1 cm) of rainfall excess occurring uniformly over the basin and at a uniform rate for a specified duration (D hours).

The definition of a unit hydrograph implies the following:

- The unit hydrograph represents the lumped response or the catchment to a limit rainfall excess of D-hduration to produce a direct-runoff hydrograph. It relates only the direct runoff to the rainfall excess. Hence the volume of water contained in the unit hydrograph must be equal to the rainfall excess. As 1 cm depth of rainfall excess is considered the area of the unit hydrograph is equal to a volume given by 1cm over the catchment.
- The rainfall is considered to have an average intensity of excess rainfall (ER) of 1/D cm/h for the duration D-hof the storm.
- The distribution of the storm is considered to be uniform all over the catchment.

Example 1

The ordinates of a hydrograph of surface runoff resulting from 4.5 cm of rainfall excess of duration 8 h in a catchment are as follows:

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Time | (h) | 0 | 5 | 13 | 21 | 28 | 32 | 35 | 41 |
| Discharge | ($m^3/s$) | 0 | 40 | 210 | 400 | 600 | 820 | 1150 | 1440 |
| Time | (h) | 45 | 55 | 61 | 91 | 98 | 115 | 138 | |
| Discharge | ($m^3/s$) | 1510 | 1420 | 1190 | 650 | 520 | 290 | 0 | |

Determine the ordinates of the 8-h unit hydrograph for this catchment.

Answer

| C1 | C2 | C3 | C4=C2/C3 |
|---|---|---|---|
| Time | DRH | Excess Rainfall | 8-h UH |
| h | $m^3/s$ | cm | $m^3/s$ |
| 0 | 0 | 4.5 | 0.0 |
| 5 | 40 | 4.5 | 8.9 |
| 13 | 210 | 4.5 | 46.7 |
| 21 | 400 | 4.5 | 88.9 |
| 28 | 600 | 4.5 | 133.3 |
| 32 | 820 | 4.5 | 182.2 |
| 35 | 1150 | 4.5 | 255.6 |
| 41 | 1440 | 4.5 | 320.0 |
| 45 | 1510 | 4.5 | 335.6 |
| 55 | 1420 | 4.5 | 315.6 |
| 61 | 1190 | 4.5 | 264.4 |
| 91 | 650 | 4.5 | 144.4 |
| 98 | 520 | 4.5 | 115.6 |
| 115 | 290 | 4.5 | 64.4 |
| 138 | 0 | 4.5 | 0.0 |

## 24.2 Assumptions of Unit Hydrograph Theory

1. The effective rainfall is uniformly distributed within its duration or specified period of time.
2. The effective rainfall is uniformly distributed over the whole area of drainage basin
3. The time base of the direct runoff hydrograph, i.e., the duration of the direct runoff hydrograph, depends only on the effective rainfall duration, and is independent of the effective rainfall intensity.
4. The response of the drainage basin is linear. This implies that the principles of proportionality and superpositionare applicable.

As per proportionality principle, the DRH ordinates are proportional to the effective rainfall intensity.

Similarly, as per superposition principle, DRH ordinates due to a complex storm, having varying effective rainfall intensities, can be obtained by superimposing the DRH due to each element of effective rainfall in succession.

5. The unit hydrograph reflects the basic effects of various physical characteristics of the basin, which do not change in time. This implies that the principle of time invarianceis valid.

The definition of the UH together with these assumptions constitutes what is now called the unit hydrograph theory.

Since in practice, assumption (1) and (2) are never satisfied, these forms the limitations of unit hydrograph theory.

Unit hydrograph theory can be applied only for a basin having drainage area between 200 ha to 5, 00,000 ha

## 24.3 Uses of Unit Hydrograph

1. Development of flood hydrograph for extreme rainfall magnitudes for use in the design of hydraulic structures.
2. Extension of flood-flow records based on rainfall records.
3. Development of flood forecasting and warning systems based on rainfall.

## 24.4 Application of Unit Hydrograph

Let it be assumed that a D-hunit-hydrograph and the storm hyetograph are available. The initial losses and infiltration losses arc estimated and deducted from the storm hydrograph to obtain the ERH. The ERH is then divided into M blocks of D-h duration each. The rainfall excess in each D-hduration is then operated upon the unit hydrograph successively to get the various DRH curves. The ordinates of these DRHs are suitably lagged to obtain the proper time sequence and are then collected and added at each time element to obtain the required net DRH due to the storm.

Consider Fig. 24.1 in which a sequence of M rainfall excess values $R_1$, $R_2$, $R_3$… $R_m$ each of duration D-h duration is shown. The line u[t]is the ordinate of a D-h unit hydrograph at t h from the beginning.

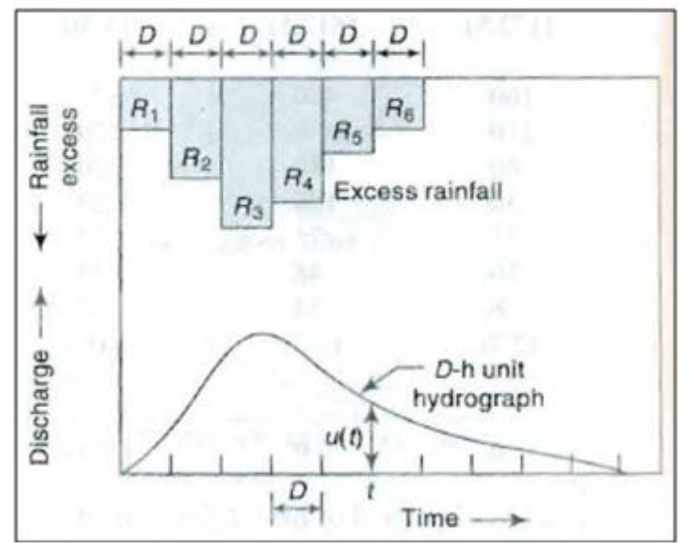


Fig. 24.1.DRH due to an ERH. (Source: Subramanya,2008)

The direct runoff' due to R1 at time t is

$$Q_1 = R_1 \cdot u[t]$$

The direct runoff due to R1at time (t -D)is

$$Q_i = R_i \cdot u[t - (i - 1)D]$$

Similarly,

$$Q_M = R_M \cdot u[t - (M - 1)D]$$

and

$$Q_t = \sum_{i=1}^{M} Q_i = \sum_{i=1}^{M} R_i \cdot u[t - (i - 1)D]$$

Thus at any time t, the total direct runoff is

After deriving the net DRH, the estimated base flow is then added to obtain the total flood hydrograph.

### Example 2

The ordinates of a 6-h unit hydrograph area given:

| Time | (h) | 0 | 3 | 6 | 9 | 12 | 15 | 18 | 21 |
|---|---|---|---|---|---|---|---|---|---|
| 6-h UH Ordinates | ($m^3$/s) | 0 | 150 | 250 | 450 | 600 | 700 | 800 | 750 |
| Time | (h) | 24 | 30 | 36 | 42 | 48 | 54 | 60 | 66 |
| 6-h UH Ordinates | ($m^3$/s) | 700 | 600 | 450 | 320 | 200 | 100 | 50 | 0 |

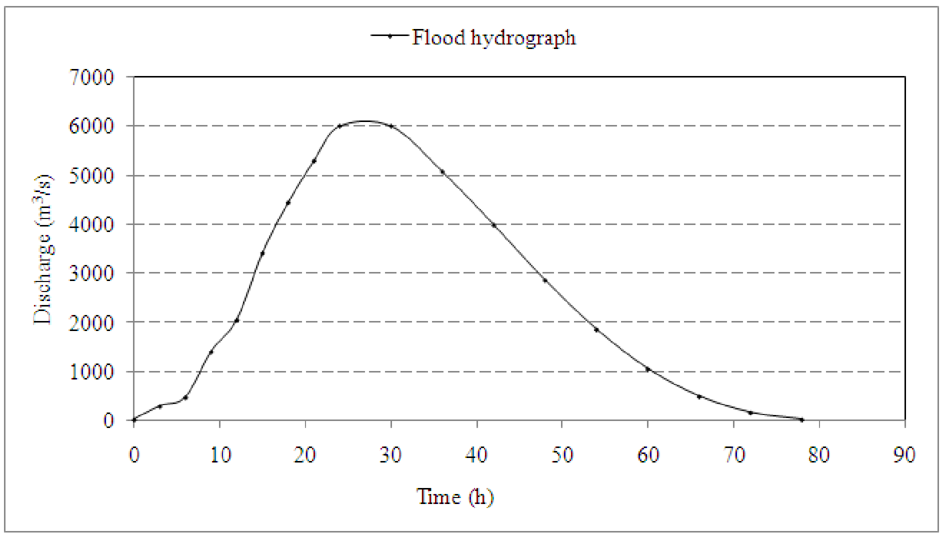
A storm had three successive 6-h intervals of rainfall magnitude of 3.0, 5.0, and 4.0 cm, respectively. Assuming aindex of 0.20 cm/h and base flow of 30 $m^3$/s, determine and plot the resulting hydrograph of flow.

Answer

|  | (1) | (2) | (3) |
|---|---|---|---|
| Rainfall, (cm) | 3 | 5 | 4 |
| Ø index, (cm/h) | 0.20 | 0.20 | 0.20 |
| Time interval ,(h) | 6 | 6 | 6 |
| losses (Ø * Δt), (cm) | 1.2 | 1.2 | 1.2 |
| Excess rainfall (Rainfall-Initial losses), (cm) | 1.8 | 3.8 | 2.8 |

| C1 | C2 | C3=C2*1.8 | C4=C2*3.8 | C5=C2*2.8 | C6= C3+C4+C5 | C7 | C8= C7+C6 |
|---|---|---|---|---|---|---|---|
| Time | 6-h UH | DRH due to | DRH due to | DRH due to |  | Base flow | Ordinates of flood |
|  |  | 1.8 cm ER | 3.8 cm ER | 2.8 cm ER |  |  | hydrograph |
|  |  |  | lagged by 6-h | lagged by 12-h |  |  |  |
| h | m3/s | m3/s | m3/s | m3/s | m3/s | m3/s | m3/s |
| 0 | 0 | 0 |  |  | 0 | 30 | 30 |
| 3 | 150 | 270 |  |  | 270 | 30 | 300 |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| 6 | 250 | 450 | 0 | | 450 | 30 | 480 |
| 9 | 450 | 810 | 570 | | 1380 | 30 | 1410 |
| 12 | 600 | 1080 | 950 | 0 | 2030 | 30 | 2060 |
| 15 | 700 | 1260 | 1710 | 420 | 3390 | 30 | 3420 |
| 18 | 800 | 1440 | 2280 | 700 | 4420 | 30 | 4450 |
| 21 | 750 | 1350 | 2660 | 1260 | 5270 | 30 | 5300 |
| 24 | 700 | 1260 | 3040 | 1680 | 5980 | 30 | 6010 |
| 30 | 600 | 1080 | 2660 | 2240 | 5980 | 30 | 6010 |
| 36 | 450 | 810 | 2280 | 1960 | 5050 | 30 | 5080 |
| 42 | 320 | 576 | 1710 | 1680 | 3966 | 30 | 3996 |
| 48 | 200 | 360 | 1216 | 1260 | 2836 | 30 | 2866 |
| 54 | 100 | 180 | 760 | 896 | 1836 | 30 | 1866 |
| 60 | 50 | 90 | 380 | 560 | 1030 | 30 | 1060 |
| 66 | 0 | 0 | 190 | 280 | 470 | 30 | 500 |
| 72 | | 0 | 0 | 140 | 140 | 30 | 170 |
| 78 | | 0 | 0 | 0 | 0 | 30 | 30 |



## Lesson 25 Derivation of Unit Hydrographs

### 25.1 Derivation of Unit Hydrograph from a Simple Storm

A number of isolated storm hydrographs caused by short spells of rainfall excess each of approximately same duration are elected from a. study of the continuously gauged runoff of the stream. For each of these storm hydrograph, the base flow is separated.

The area under each DRH is evaluated and the volume of the direct runoff obtained is divided by the catchment area to obtain the depth of ER. The ordinates of the various DRHs are divided by the respective ER values to obtain the ordinates of the Unit hydrograph.

Flood hydrograph used in the analysis should be selected to meet the following desirable features with respect to the storm responsible for them.

The storms should be isolated storms occurring individually.

- The rainfall should be fairly uniform during the duration and should cover the entire catchment area.
- The duration of the rainfall should be 1/5 to 1/3of the basin lag.
- The rainfall excess of the selected storm should be high. A range of ER values of 1.0 to 4.0 cm is sometimes preferred.

Example 1

A flood hydrograph of a river draining a catchment of 189 km$^2$ due to a 6 h isolated storm is in the form of a triangle with a base of 66 h and a peak ordinate of 30 m$^3$/s occurring at 10 hours from the start. Assuming zero base flow, develop the 6-hour unit hydrograph for this catchment.

Answer

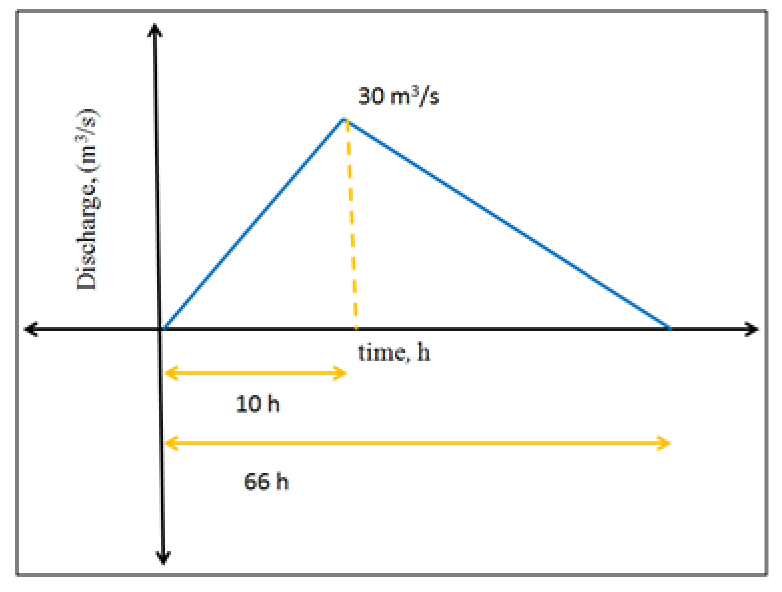
Given: catchment area (A) = 189 km$^2$

Time interval = 6 h

Time base = 66 h

Peak = 30 m$^3$/s

To find: 6-h UH for catchment



From figure

$$\text{Runoff volume} = \frac{1}{2} \times B \times H$$

$$\text{Runoff volume} = \frac{1}{2} \times 66 \times 60 \times 60 \times 30$$

$$\textbf{Runoff volume} = \mathbf{3564000\ m^3}$$

Therefore,

$$\text{Excess rainfall} = \frac{3564000}{189 \times 10^6}$$

Excess Rainfall = 1.89 cm

$$\text{Peak of UH} = \frac{30}{1.89}$$

Peak of UH = 15.909 $m^2/s$ at 10 h from start

25.2 Derivation of Unit Hydrograph from a Complex Storm

When suitable simple isolated storm are not available, data from complex storms of long duration will have to be used in unit-hydrograph derivation.

Consider a rainfall excess made up of three consecutive durations of D-h and ER values of $R_1$, $R_2$, $R_3$.Figure 25.1 shows the ERR By base flow separation of the resulting composite flood hydrograph a composite DRH is obtained (Fig. 25.1).

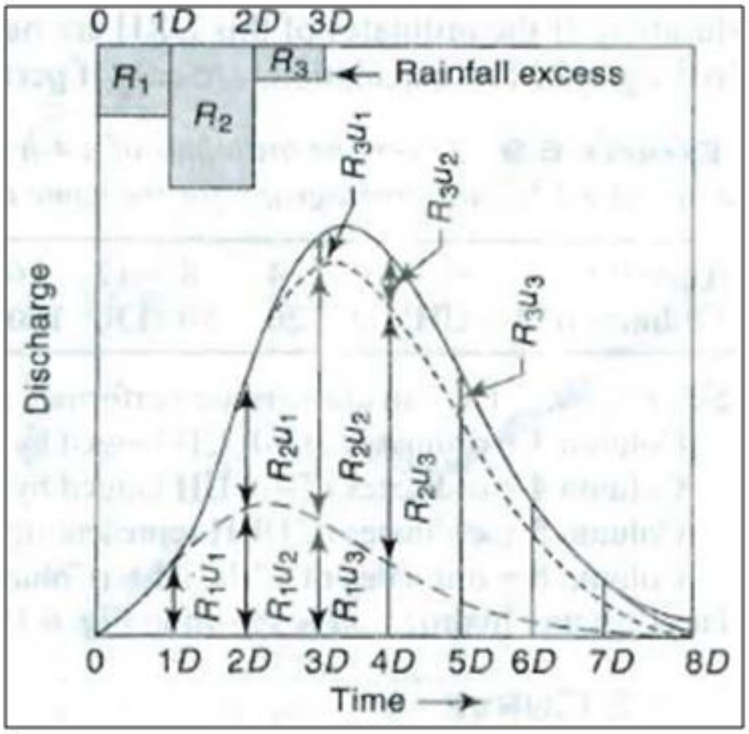


Fig. 25.1.Unit hydrograph from a Complex storm. (Source: Subramanya, 2008)

Let the ordinates of composite DRH be drawn at a time interval of D-h. At various time intervals 1D, 2D, 3D,… from the start of ERH, let the ordinates of the unit hydrograph be $u_1$, $u_2$, $u_3$,… and the ordinates of the composite DRH be $Q_1$, $Q_2$, $Q_3$,…,

Then

$$Q_1 = R_1 u_1$$

$$Q_2 = R_1 u_2 + R_2 u_1$$

$$Q_3 = R_1 u_3 + R_2 u_2 + R_3 u_1$$

$$Q_4 = R_1 u_4 + R_2 u_3 + R_3 u_2$$

$$Q_4 = R_1 u_5 + R_2 u_4 + R_3 u_3$$

so on

From Eq. 25.1 the values of $u_1$, $u_2$, $u_3$…… can be determined.

However, this method suffers from the disadvantage that the errors propagate and increase as the calculations proceed. In the presence of errors the recession limb of the derived D-hunit hydrograph can contain oscillations and even negative values.

## Example 1

The ordinates of the 6-h unit hydrograph of a basin are given:

| Time | h | 0 | 6 | 12 | 18 | 24 | 30 | 36 | 42 | 48 | 54 | 60 | 66 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Ordinate of 6-h UH | $m^3/s$ | 0 | 20 | 60 | 150 | 120 | 90 | 66 | 50 | 32 | 20 | 10 | 0 |

If two storms, each of 1-cm rainfall excess and 6-h duration occur in succession, calculate the resulting hydrograph of flow. Assume the base flow to be uniform at $10 m^3/s$.

Answer

| C1 | C2 | C3=C2*1 | C4=C2*1 | C5= C3+C4 | C6 | C7= C5+C6 |
|---|---|---|---|---|---|---|
| Time | Ordinate of 6-h UH | DRH due to 1. cm ER | DRH due to 1 cm ER lagged by 6-h | | Base flow | Ordinates of flood hydrograph |
| h | $m^3/s$ | $m^3/s$ | $m^3/s$ | $m^3/s$ | $m^3/s$ | $m^3/s$ |
| 0 | 0 | 0 | | 0 | 10 | 10 |
| 6 | 20 | 20 | 0 | 20 | 10 | 30 |
| 12 | 60 | 60 | 20 | 80 | 10 | 90 |
| 18 | 150 | 150 | 60 | 210 | 10 | 220 |
| 24 | 120 | 120 | 150 | 270 | 10 | 280 |
| 30 | 90 | 90 | 120 | 210 | 10 | 220 |
| 36 | 66 | 66 | 90 | 156 | 10 | 166 |
| 42 | 50 | 50 | 66 | 116 | 10 | 126 |
| 48 | 32 | 32 | 50 | 82 | 10 | 92 |
| 54 | 20 | 20 | 32 | 52 | 10 | 62 |
| 60 | 10 | 10 | 20 | 30 | 10 | 40 |
| 66 | 0 | 0 | 10 | 10 | 10 | 20 |
| 72 | | | 0 | 0 | 10 | 10 |

## Lesson 26 Unit Hydrographs of Different Durations

Lack ofadequate data normally precludes development ofunit hydrographscovering a wide range ofdurations for a given catchment. Undersuch conditions aD hour unit hydrograph is used to develop unit hydrographs of differing durations nD. Two methods are available for this purpose.

### 26.1 Method of Superposition

If a D-h unit hydrograph is available and it is desired todevelop a unit hydrograph of nDh, where n isan integer, it is easily accomplished by superposing n unit hydrograph with each graph separated from the previous on by D-h.

Example 1

The ordinates of a 6-h unit hydrograph are given

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Time | (h) | 0 | 6 | 12 | 18 | 24 | 30 |
| Ordinate of 6-h UH | ($m^3/s$) | 0 | 20 | 60 | 150 | 120 | 90 |
| Time | (h) | 36 | 42 | 48 | 54 | 60 | 66 |
| Ordinate of 6-h UH | ($m^3/s$) | 66 | 50 | 32 | 20 | 10 | 0 |

Derive a 12-h unit hydrograph for the catchment.

Answer

| C1 | C2 | C3 | C4= C2+C3 | C5 = (C4/(12/6)) |
|---|---|---|---|---|
| Time | Ordinate of 6-h UH | Ordinates of 6-h UH lagged by 6-h | | C5 = (C4/2)<br>Ordinates of 12-h UH |
| h | $m^3/s$ | $m^3/s$ | $m^3/s$ | $m^3/s$ |
| 0 | 0 | | 0 | 0 |
| 6 | 20 | 0 | 20 | 10 |
| 12 | 60 | 20 | 80 | 40 |
| 18 | 150 | 60 | 210 | 105 |

| | | | | |
|---|---|---|---|---|
| 24 | 120 | 150 | 270 | 135 |
| 30 | 90 | 120 | 210 | 105 |
| 36 | 66 | 90 | 156 | 78 |
| 42 | 50 | 66 | 116 | 58 |
| 48 | 32 | 50 | 82 | 41 |
| 54 | 20 | 32 | 52 | 26 |
| 60 | 10 | 20 | 30 | 15 |
| 66 | 0 | 10 | 10 | 5 |
| 72 | | 0 | 0 | 0 |

## 26.2 S-curve

If it is desired to develop a unit hydrograph of durationmD, where m is a fraction, the method of superposition cannot be used. A different technique known as the S-curve method is adopted in such cases, and this method isapplicable forrational values of m.

The S-curve, also known as S-hydrograph is a hydrograph produced by a continuous effective rainfall at a constant rate for an infinite period. It is a curve obtained by summation of an infinite series of D-h unit hydrographs spaced D-hapart.

Fig 26.1 shows such a series of D-hhydrograph arranged with their starting points D-hapart.

At any given time the ordinates of the various curves occurring at that time coordinate are summed up to obtain ordinates of the S-curve. A smooth curve through these ordinate results in an S-shaped curve called S-curve.

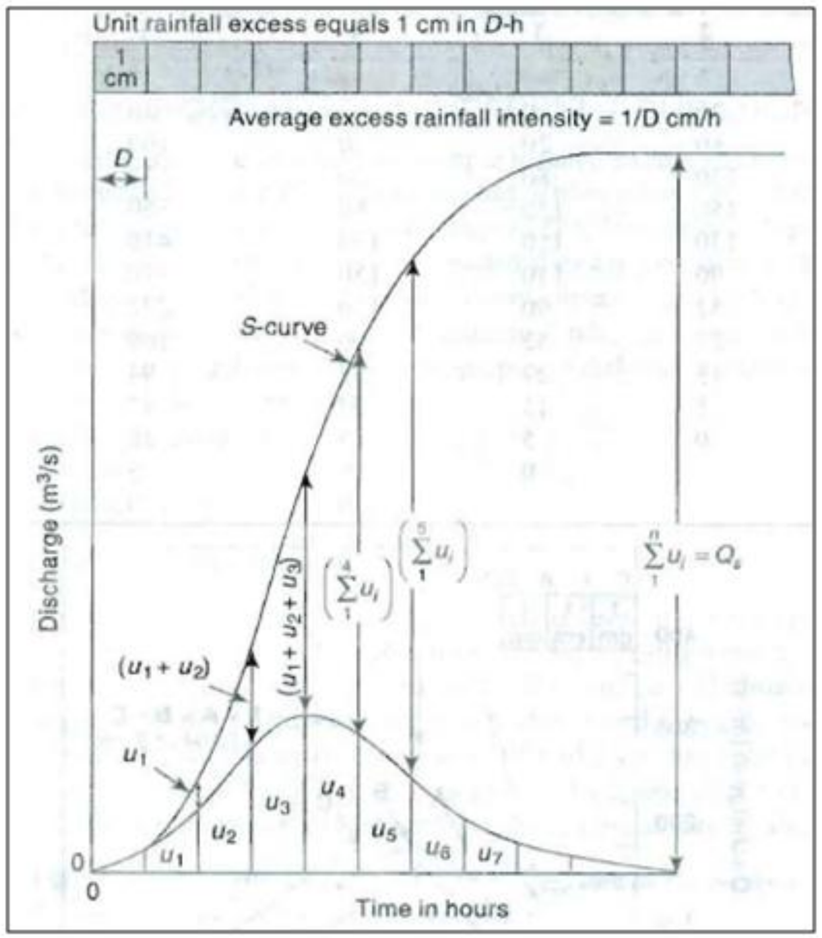


Fig. 26.1S-curve. (Source: Subramanya, 2008)

This S-curve is due to a D-h unit hydrograph. It has an initial steep portion and reaches a maximum equilibrium discharge at a time equal to the first unit hydrograph. The average intensity of ER producing the S-curve is 1/D cm/h and the equilibrium discharge,

$$Q_s = \left(\frac{A}{D} \times 10^4\right) m^3/h$$

Where A is area of catchment in km² and D is duration in hours of ER of the unit hydrograph used in deriving the S-curve.

By definition an S-curve is obtained by adding a string of D-h unit hydrographs each lagged by D-hours from one another. Further, if $T_b$ = base period of the unit hydrograph, addition of only $T_b/D$ unit hydrographs are sufficient to obtain the S-curve. However, an easier procedure based on the basic property of the S-curve is available for the construction of S-curves.

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

or

$$S(t) = U(t) + (t - D) \quad (26.1)$$

The term S (t-D)could be called S-curve addition at time t

For all

Example 2

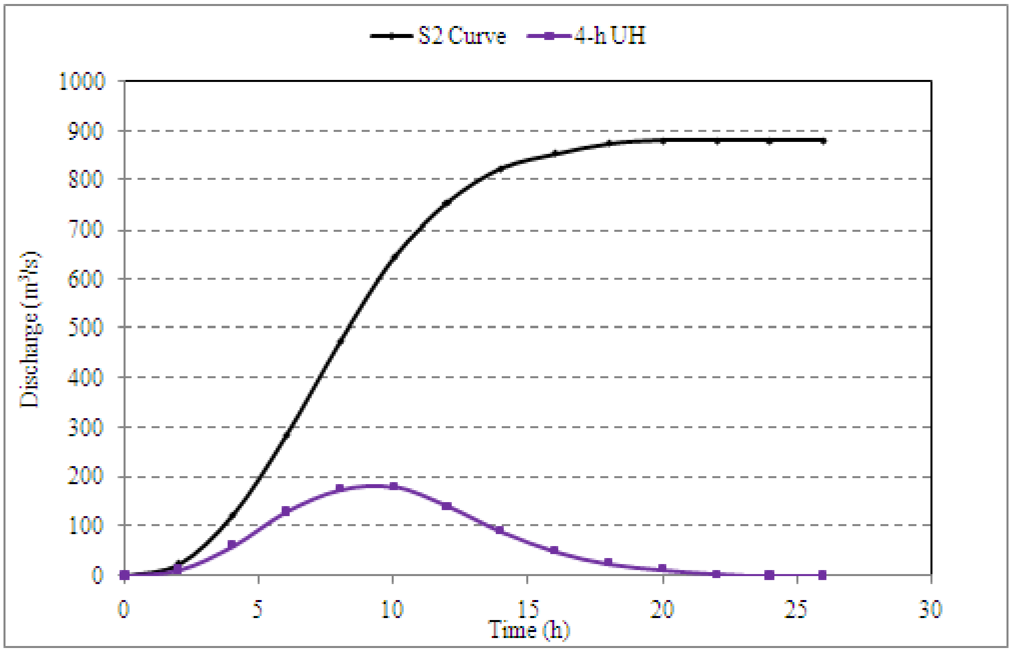
The ordinate of 2-h unit hydrograph of a basin are given:

| Time | (h) | 0 | 2 | 4 | 6 | 8 | 10 | 12 |
|---|---|---|---|---|---|---|---|---|
| 2-h UH Ordinates | ($m^3/s$) | 0 | 25 | 100 | 160 | 190 | 170 | 110 |
| Time | (h) | 14 | 16 | 18 | 20 | 22 | 24 | 26 |
| 2-h UH Ordinates | ($m^3/s$) | 70 | 30 | 20 | 6 | 0 | 0 | 0 |

Compute a 4-h unit hydrograph ordinate and plot: (i) the S-curve (ii) the 4-h UG

| C1 | C2 | C3 | C4 | C5 | C6 = C4-C5 | C7 = C6/ (4/2) |
|---|---|---|---|---|---|---|
| Time | 2-h UH Ordinates | S curve addition | S2 curve ordinate | S2 curve lagged by 4 h | DRH of (4/2)= 2 cm | 4-h UH Ordinates |
| h | $m^3/s$ | | | | $m^3/s$ | $m^3/s$ |
| 0 | 0 | 0 | 0 | | 0 | 0.0 |
| 2 | 25 | 0 | 25 | | 25 | 12.5 |
| 4 | 100 | 25 | 125 | 0 | 125 | 62.5 |
| 6 | 160 | 125 | 285 | 25 | 260 | 130.0 |
| 8 | 190 | 285 | 475 | 125 | 350 | 175.0 |
| 10 | 170 | 475 | 645 | 285 | 360 | 180.0 |
| 12 | 110 | 645 | 755 | 475 | 280 | 140.0 |

| | | | | | | |
|---|---|---|---|---|---|---|
| 14 | 70 | 755 | 825 | 645 | 180 | 90.0 |
| 16 | 30 | 825 | 855 | 755 | 100 | 50.0 |
| 18 | 20 | 855 | 875 | 825 | 50 | 25.0 |
| 20 | 6 | 875 | 881 | 855 | 26 | 13.0 |
| 22 | 0 | 881 | 881 | 875 | 6 | 3.0 |
| 24 | 0 | 881 | 881 | 881 | 0 | 0.0 |
| 26 | 0 | 881 | 881 | 881 | 0 | 0.0 |



## Lesson 27 Synthetic Unit Hydrograph

### 27.1 Definition and Uses of Synthetic Unit Hydrograph

To develop unit hydro graphs to a catchment, detailed information about the rainfall and the resulting flood hydrograph are needed. However, such information would be available only at a few locations and in a majority of catchments, especially those which are at remote locations; the data would normally be very scanty. In order to construct unit hydrographs for such areas, empirical equations of regional validity which relate the salient hydrograph characteristics to the basin characteristics are available. Unit hydrographs derived from such relationships are known as .synthetic-unit hydrographs.

These methods being based on empirical correlations are applicable only to the specific regions in which they were developed and should not be considered as general relationships for use in all regions.

### 27.2 Snyder's Method

Snyder (1938), based on a study of a large number of catchments in the Appalachian Highlands of eastern United States developed a set of empirical equations for synthetic-unit hydrographs in those areas.

These equations are in use in the USA, and with some modifications in many other countries, and constitute what is known as Snyder's synthetic-unit hydrograph.The most important characteristic of a basin affecting a hydrograph due to a storm is basin lag. Basin lag (also known as lag time) is the time difference between the centroid of the input (rainfall excess) and the output (direct runoff hydrograph). However, difficulty in determining the centroid of the direct runoff hydrograph (DRH), lag time is defined for practical purposes as the elapsed time between the centroid of rainfall excess and peak of DRH. Physically, lag time represents the mean time of travel of water from all parts of the watershed to the outlet during a given storm. Its value is determined essentially on the topographical features, such as the size, shape, stream density, length of main stream, slope, land use and land cover. The modified definition of basin time is very commonly adopted in the derivation of synthetic unit hydrographs for a given watershed.

The first of the Snyder's equation relate the basin lag $t_p$,defined as the time interval from the mid-point of rainfall excess to the peak of the unit hydrograph (Fig. 27.1) to the basin characteristics as

$$t_p = C_t(LL_{ca})^{0.3} \quad (27.1)$$

where $t_p$ is basin lag in hours, L is basin length measured along the water course from the basin divide to the gauging station in km, $L_{ca}$ distance along the main water course from the gauging station to a point opposite to the watershed centroid in km, $C_t$ is a regional constant representing watershed slope and storage effects (1.3 to 1.65).

Snyder adopted a standard duration $t_r$hours of effective rainfall given by

$$t_r = \frac{t_p}{5.5} \quad (27.2)$$

The peak discharge $Q_{ps}$($m^3/s$)of a unit hydrograph of Standard duration $t_r$h is given by Snyder as

$$Q_{ps} = \frac{2.78 C_p A}{t_p} \quad (27.3)$$

WhereA is catchment area km$^2$, $C_p$ is a regional constant (0.56-0.69).

This equation is based on the assumption that the peak discharge is proportional to the average discharge of $\left(\frac{1\ cm \times catchmentarea}{durationofrainfallexcess}\right)$.

Lf a non-standard rainfall duration $t_R$h is adopted, instead of the standard value $t_r$to derive a unit hydrograph the value of the basin lag is affected. The modified basin lag is given by

$$t_p^{'} = t_p + \frac{t_R + t_r}{4} = \frac{21}{22} t_p + \frac{t_R}{4} \quad (27.4)$$

Where= basin lag in hours for an effective duration of$t_R$hand $t_p$is as given by Eq. (27.1).The value of must be used instead of $t_p$in Eq. (27.2). Thus the peak discharge for a nonstandard ER of duration $t_R$is in m$^3$/s.

$$Q_{ps} = \frac{2.78 C_p A}{t_p^{'}} \quad (27.3a)$$

When $t_R = t_r$

$Q_p = Q_{ps}$

The time base of a unit hydrograph (Fig. 27.1) is given by Snyder as

$$T_b = 3 + \frac{t_p'}{8} \text{ days}$$
$$= \left(72 + 3\, t_p'\right) \text{hours} \quad (27.5)$$

With$t_b$(given in h) taken as the next larger integer value divisible by $t_R$, i.e. $T_b$is about five times the time-to-peak.

To assist in the sketching the unit hydrographs, the widths of unit hydrographs at 50 and 75% of the peak are given by

$$W_{50} = \frac{5.87}{q^{1.08}} \quad (27.6)$$

$$W_{75} = \frac{W_{50}}{1.75} \quad (27.7)$$

Where$W_{50}$ is width of UH in h at 50% peak discharge, $W_{75}$ is width of UH in h at 75% peak discharge

$q = Q_p/A$ = peak discharge per unit catchment area in $m^3/s/km^2$

Example 1

Characteristics of two catchments M and N measured from a map are given below:

| Item | Catchment M | Catchment N |
|---|---|---|
| $L_{ca}$ | 76 km | 52 km |
| L | 148 km | 106 km |
| A | 2718 km$^2$ | 1400 km$^2$ |

For the 6-h unit hydrograph in catchment M, the peak discharge is at 200 $m^3/s$ and occurs at 37 h from the start of the rainfall excess. Assuming the catchments M and N are meteorologically similar; determine the elements of the 6-h synthetic unit hydrograph for catchment N by using Snyder's method.

Answer

a. For catchment M

$t_r$ = 6 h

Time to peak from beginning of ER

$$T_p = \frac{t_R}{2} + t_p' = 37\ h$$

$$\frac{6}{2} + t_p' = 37\ h$$

$t'_p$ = 34h

From equation

$$t_p' = \frac{21}{22} t_p + \frac{t_R}{4} = 34$$

$$\therefore t_p = \left(34 - \frac{6}{4}\right) \times \frac{22}{21}$$

tp = 34.047

$$t_p = C_t (LL_{ca})^{0.3}$$

$$34.047 = C_t (148 \times 76)^{0.3}$$

ct = 2.073

$$Q_{ps} = \frac{2.78 C_p A}{t_p'}$$

$$200 = \frac{2.78 \times C_p \times 2718}{34}$$

$C_p$ = 0.899 ≈ 0.9

b. For catchment N

$$t_p = 2.073(106 \times 52)^{0.3}$$

$$\boldsymbol{t_p = 27.47 \approx 27.48h}$$

$$t_r = \frac{t_p}{5.5}$$

$$t_r = \frac{27.48}{5.5}$$

$$\boldsymbol{t_r = 4.99 \approx 5h}$$

$$\text{Using } t_r = 6\ h$$

$$t_p^{'} = \frac{21}{22} t_p + \frac{t_R}{4}$$

$$t_p^{'} = \frac{21}{22} \times 27.48 + \frac{6}{4}$$

$$\boldsymbol{t_p^{'} = 27.73\mathrm{h}}$$

$$Q_{ps} = \frac{2.78 C_p A}{t_p^{'}}$$

$$Q_{ps} = \frac{2.78 \times 0.9 \times 1400}{27.73}$$

$$\boldsymbol{Q_{ps} = 126.32\ \mathrm{m}^3/\mathrm{s}}$$

$$q = \frac{Q_p}{A}$$

$$q = \frac{126.32}{1400}$$

$$\boldsymbol{q = 0.09}$$

$$W_{50} = \frac{5.87}{q^{1.08}}$$

$$W_{50} = \frac{5.87}{0.09^{1.08}}$$

$$\boldsymbol{W_{50} = 78.86\mathrm{h}}$$

$$W_{75} = \frac{W_{50}}{1.75}$$

$$W_{75} = \frac{78.86}{1.75}$$

$$\boldsymbol{W_{75} = 45.06h}$$

$$T_b = 3 + \frac{t_p^{'}}{8} \text{ days} = \left(72 + 3\ t_p^{'}\right) h$$

$$T_b = (72 + 3\ \times 27.73)$$

$$\boldsymbol{T_b = 155.91\mathrm{h}}$$

Using

# Lesson 28 Other Unit Hydrographs

## 28.1 Distribution Graph

The distribution graph introduced by Bernard (1935) is a variation of the unit hydrograph. It is basically D-h unit hydro graph with ordinates showing the percentage of the surface runoff occurring in successive periods of equal percentile intervals ofD-h. The duration of the rainfall excess (D-h) is taken as the unit interval and distribution-graph ordinates are indicated at successive such unit intervals. Note the ordinates plotted at D-h intervals and the total area under the distribution graph adds up to 100%. Distribution graphs are useful in comparing the runoff characteristics of different catchments.

Example 1

The 4-h, distribution graph of catchment of 50 km$^2$ area has the following ordinates:

| Unit periods (4-h units) | 1 | 2 | 3 | 4 | 5 | 6 |
|---|---|---|---|---|---|---|
| Distribution (percentage) | 5 | 20 | 40 | 20 | 10 | 5 |

If the catchment has rainfall of 3.5, 2.2, and 1.8 cm in three consecutive 4-h periods, determine the resulting direct runoff hydrograph by assuming the  index for the storm as 0.25 cm/h.

| C1 | C2 | C3 | C4 = C3*4 | C5 = C2 – C4 | C6 | C7 = C6/100 | C8 = 2.5*C7; 1.2*C7 (lagged by 4h); 0.8*C7 (lagged by 8 h) | | | C9 = Summation of C8 | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Time interval | Rainfall | Infiltration loss | Infiltration loss | Effective rainfall | Avg distribution ratio | Avg distribution ratio | Distributed runoff for rainfall excess of | | | Runoff | |
| h | cm | cm/h | cm | cm | (percentage) | (decimal) | 2.5 cm | 1.2 cm | 0.8 cm | cm | $m^3$/s |
| 0-4 h | 3.5 | 0.25 | 1 | 2.5 | 5 | 0.05 | 0.125 | | | 0.125 | 4.3 |
| 4-8 h | 2.2 | 0.25 | 1 | 1.2 | 20 | 0.2 | 0.5 | 0.06 | | 0.56 | 19.4 |
| 8-12 | 1.8 | 0.25 | 1 | 0.8 | 40 | 0.4 | 1 | 0.24 | 0.04 | 1.28 | 44. |

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| h | | | | | | | | | | | 4 |
| 12-16 h | | | | | 20 | 0.2 | 0.5 | 0.48 | 0.16 | 1.14 | 39.6 |
| 16-20 h | | | | | 10 | 0.1 | 0.25 | 0.24 | 0.32 | 0.81 | 28.1 |
| 20-24 h | | | | | 5 | 0.05 | 0.125 | 0.12 | 0.16 | 0.405 | 14.1 |
| 24-28 h | | | | | | | | 0.06 | 0.08 | 0.14 | 4.9 |
| 28-32 h | | | | | | | | | 0.04 | 0.04 | 1.4 |

Answer

## 28.2 Dimensionless Unit Hydrograph

Dimensionless unit hydrographs based on a study of a large number of unit hydrographs. They are also used to facilitate construction of synthetic unit hydrographs. A typical dimensionless unit hydrograph contains ordinate ($Q/Q_p$) which is the discharge Q expressed as a ratio to the peak discharge $Q_p$and the abscissa ($t/T_p$), which is the time I expressed as a ratio of the time to peak $T_p$. By definition, $Q/Q_p = 1.0$ when $t/T_p = 1.0$.

## 28.3 Instantaneous Unit Hydrograph

Fig 28.1 shows a typical variation of the shape of unit hydrographs for different values of D. As D is reduced, the intensity of rainfall excess being equal to 1/D increases and the unit hydrograph becomes more skewed. A finite unit hydrograph is indicated as the duration D → 0. The limiting case of a unit hydrograph of zero duration is known as instantaneous unit hydrograph(IUH).

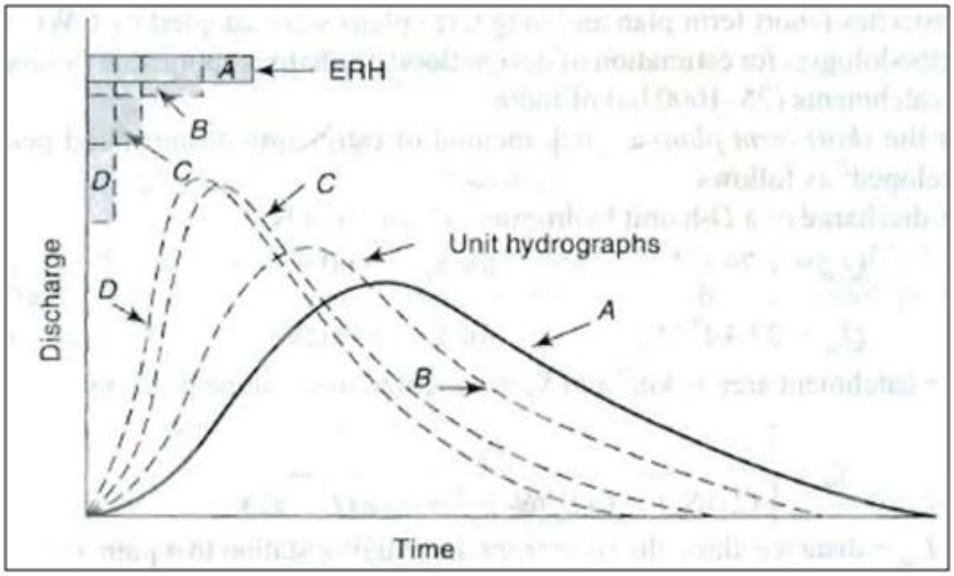


Fig. 28.1.Unit hydrograph of different durations. (Source:Subramanya, 2008)

IUH is a fictitious, conceptual unit hydrograph which represents the surface runoff from the catchment due to an instantaneous precipitation of the rainfall excess volume of 1 cm.

IUH is designated as u (t)or sometimes as u(0, t). It is a single-peaked hydrograph with a finite base width and its important properties can be listed as below:

1. $0 \le u(t) \le$ a positive value, for $t > 0$;
2. $u(t) = 0$ for $t \le 0$;
3. $u(t) \rightarrow = 0$ as $t \rightarrow \infty$;
4. = unit depth over the catchment; and
5. Time to the peak time to the centroid of the curve.

The main advantage of IUH is that it is independent of the duration of ERH and thus has one parameter less than a D-h unit hydrograph. This fact and the definition of IUH make it eminently suitable for theoretical analysis of rainfall excess-runoff relationship of a catchment. For a given catchment IUH, being independent of rainfall characteristics, is indicative of the catchment storage characteristics.

Module 7. Flood Routing

## Lesson 29 Hydrologic Reservoir Routing

### 29.1 Definition and Types of Flood Routing

Flood routing is the technique of determining the flood hydro graph at a section of a river by utilizing the data of flood flow at one or more upstream sections.In these applications two broad categories or routing can be recognized. These are:

#### 29.1.1Reservoir Routing

In Reservoir routing the effect of a flood wave entering a reservoir is studied. Knowing the volume-elevation characteristic of the reservoir and the outflow-elevation relationship for the spillways and other outlet structures in the reservoir, the effect of a flood wave entering the reservoir is studied to predict the variations of reservoir elevation and outflow discharge with time. This form of reservoir routing is essential (i) in the design of the capacity of spillways and other reservoir outlet structures, and (ii) in the location and sizing of the capacity of reservoirs to meet specific requirements.

#### 29.1.2Channel Routing

In Channel routing the change in the shape of a hydrograph as it travels down a channel is studied. By considering a channel reach and an input hydrograph at the upstream end, this form of routing aims to predict the flood hydrograph at various sections of the reach. Information on the flood-peak attenuation and the duration of high-water levels obtained by channel routing is of utmost importance in flood-forecasting operations and flood-protection works.

### 29.2 Uses of Flood Routing

The hydrologic analysis of problems such as flood forecasting, flood protection, reservoir design and spillway design invariably include flood routing.

### 29.3 Methods of Flood Routing

A variety of routing methods are available and they can be broadly classified into two categories as: (i) hydrologic routing, and (ii) hydraulic routing.

Hydrologic-routing methods employ essentially the equation of continuity.

Hydraulic methods, on the other hand, employ the continuity equation together with the equation of motion of unsteady flow. The basic differential equations used in the hydraulic routing, known as St. Venant equations afford a better description of unsteady flow than hydrologic methods.

## 29.4 Basic Equations

The passage of a flood hydrograph through a reservoir or a channel reach is an unsteady-flow phenomenon. It is classified in open-channel hydraulics as gradually varied unsteady flow. The equation of continuity used in all hydrologic routing as the primary equation states that the difference between the inflow and outflow rate is equal to the rate of change of storage, i.e.

$$I - Q = \frac{dS}{dt}$$

(29.1)

WhereI is inflow rate, Q is outflow rate and S= storage

Alternatively, in a small time interval Δt the difference between the total inflow volume and total outflow volume in a reach is equal to the change in storage in that reach

$$\bar{I}\Delta t - \bar{Q}\Delta t = \Delta S$$ (29.2)

Where  is average inflow in time Δt,  is average outflow in time Δt, and  change in storage.

By taking $\bar{I} = (I_1 + I_2)/2, \bar{Q} = (Q_1 + Q_2)/2$ , and $\Delta S = S_2 - S_1$ with suffixes 1 and 2 to denote the beginning and end of time interval Δt, Eq. 29.2 is written as

$$\frac{I_1 + I_2}{2}\Delta t - \frac{Q_1 + Q_2}{2}\Delta t = S_2 - S_1$$ (29.3)

The time interval Δt should be sufficiently short so that the inflow and outflow hydrographs can be assumed to be straight lines in that time interval. Further Δt must be shorter than the time of transit of the flood wave through the reach.

In the differential form the equation or continuity for unsteady now in a reach with no lateral flow is given by

$$\frac{\partial Q}{\partial x} + T\frac{\partial y}{\partial t} = 0$$ (29.4)

Where T is top width of section, y isdepth of flow

The equation of motion for a flood wave is derived from the application of the momentum equation as

$$\frac{\partial y}{\partial x}+\frac{V}{g}\frac{\partial V}{\partial x}+\frac{1}{g}\frac{\partial V}{\partial t}= S_0 - S_f \quad (29.5)$$

WhereV is velocity of flow at any section, $S_0$is channel bed slope;$S_f$ is slope of the energy line.

Equation 29.4 and 29.5 are commonly known as St. Venant equations

$$\left(\frac{I_1+I_2}{2}\right)\Delta t+\left(S_1-\frac{Q_1\Delta t}{2}\right)=\left(S_2+\frac{Q_2\Delta t}{2}\right) \quad (29.3)$$

## 29.5 Hydrologic Reservoir Routing using Modified Pul's Method

Equation 29.3 is rearranged as

At the starting of flood routing, the initial storage and outflow discharges are known.

This method requires construction of only two curves (i) S curve and (ii) $S+\frac{Q\,\Delta t}{2}$ curve.

Following steps are followed to compute reservoir hydrologic routing using Modified Pul's method.

1. From the inflow hydrograph, obtain the volume of water entering the reservoir in the short time interval, i.e., Compute average inflow $\left(\frac{I_1+I_2}{2}\right)\Delta t$.
2. For an initial outflow obtain from S curve and compute $S_1-\frac{Q_1\Delta t}{2}$.
3. Add average inflow $\left(\frac{I_1+I_2}{2}\right)\Delta t$ and $\left(S_1-\frac{Q_1\Delta t}{2}\right)$ to obtain $\left(S_2+\frac{Q_2\Delta t}{2}\right)$.
4. Using computed values $\left(S_2+\frac{Q_2\Delta t}{2}\right)$, obtain $Q_2$ from the $S+\frac{Q\,\Delta t}{2}$ curve.
5. Repeat the entire procedure to complete the routing.

Example 1

A small reservoir has the following storage elevation relationship

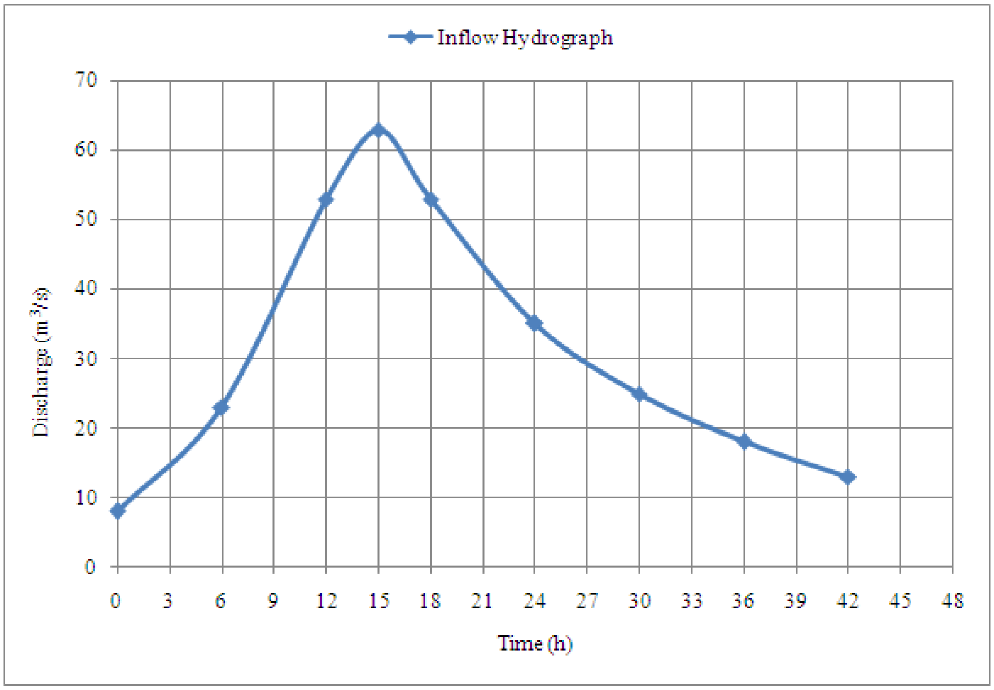
| Elevation (m) | 55.00 | 58.00 | 60.00 | 61.00 | 62.00 | 63.00 |
|---|---|---|---|---|---|---|
| Storage ($10^3$ $m^3$) | 250 | 650 | 1000 | 1250 | 1500 | 1800 |

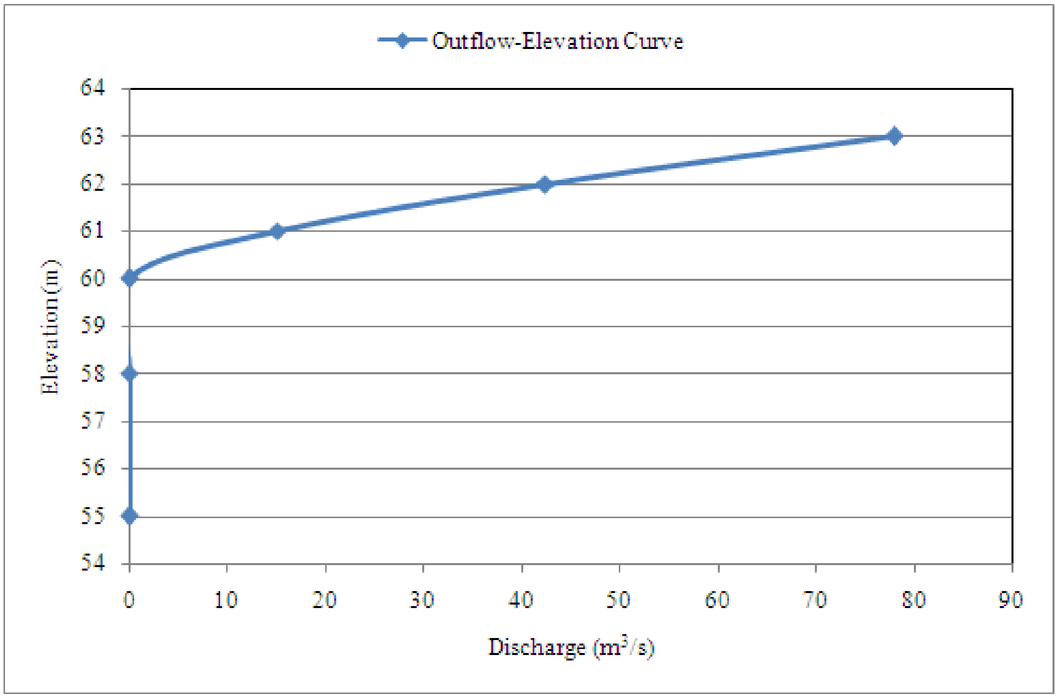
A spillway provided with its crest at elevation 60.00 m has the discharge relationship. $Q = 15H^{3/2}$, where H = head of water over the spillway crest.When the reservoir elevation is at 58.00 m a flood as given below enters the reservoir.

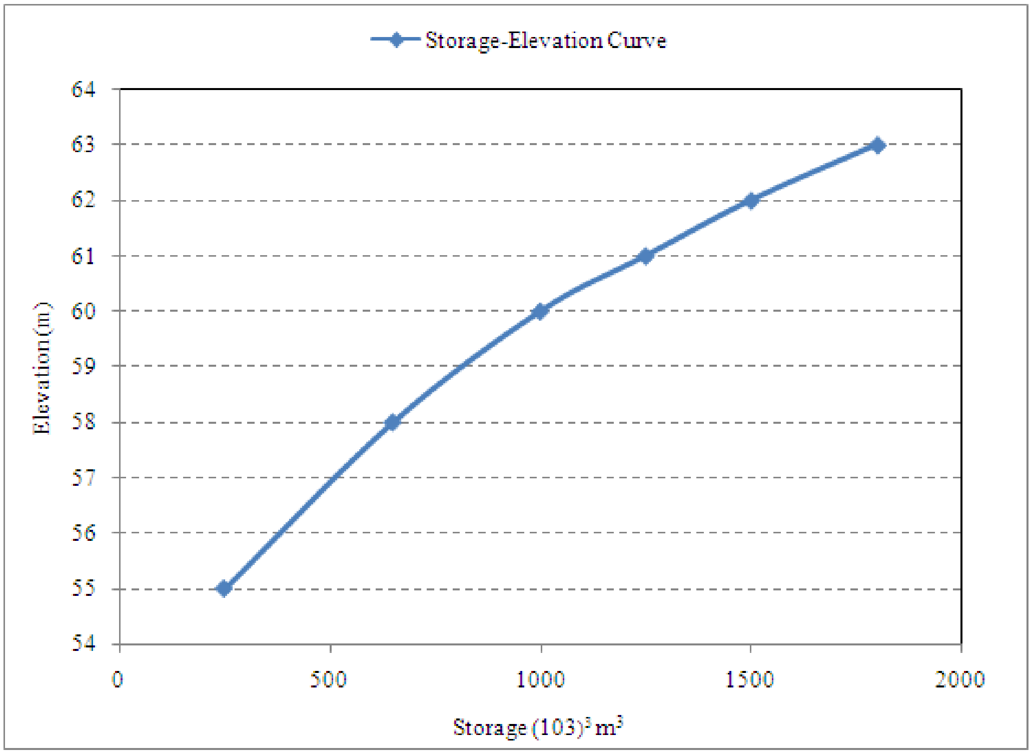
Route the flood and determine the maximum reservoir elevation, peak outflow and attenuation of the flood peak using Modified Pul's method.

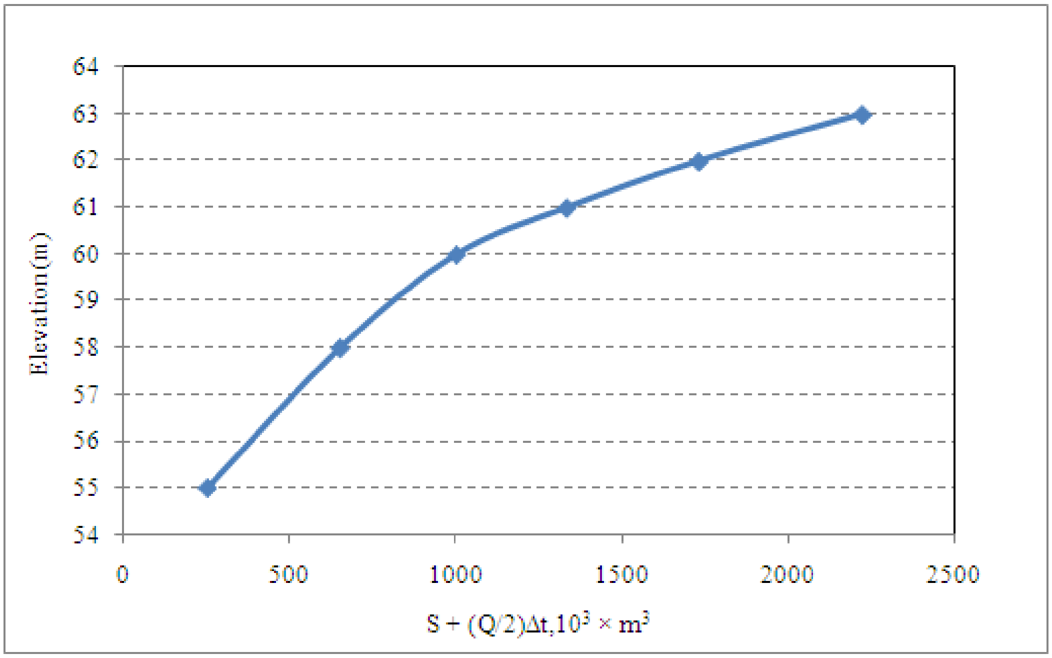
| Time (h) | 0 | 6 | 12 | 15 | 18 | 24 | 30 | 36 | 42 |
|---|---|---|---|---|---|---|---|---|---|
| Inflow ($m^3/s$) | 8 | 23 | 53 | 63 | 53 | 35 | 25 | 18 | 13 |

Answer




Outflow-Elevation Curve
Elevation (m)
64
63
62
61
60
59
58
57
56
55
54
0
10
20
30
40
50
60
70
80
90
Discharge (m³/s)


Storage-Elevation Curve
Elevation (m)
64
63
62
61
60
59
58
57
56
55
54
0
500
1000
1500
2000
Storage (103)³ m³



| Routing period | $I_1$(m$^3$/s) | $I_2$(m$^3$/s) | $\left(\frac{I_1+I_2}{2}\right)\times 10^3\,\mathrm{m}^3$ | $Q_1$(m$^3$/s) | $S_1 - \frac{Q_1}{2}\Delta t \times 10^3\,\mathrm{m}^3$ | $S_2 + \frac{Q_2}{2}\Delta t \times 10^3\,\mathrm{m}^3$ | $Q_2$(m$^3$/s) | Elevation at beginning (m) | Elevation at end (m) |
|---|---|---|---|---|---|---|---|---|---|
| 0-3 | 8 | 14.5 | 121.5 | 0 | 650 | 771.5 | 0 | 58 | 58.6 |
| 3-6 | 14.5 | 23 | 202.5 | 0 | 771.5 | 974 | 0 | 58.6 | 59.8 |
| 6-9 | 23 | 38.5 | 332.1 | 0 | 974 | 1306.1 | 13 | 59.8 | 60.8 |
| 9-12 | 38.5 | 53 | 494.1 | 13 | 1165.7 | 1659.8 | 39 | 60.8 | 61.9 |
| 12-15 | 53 | 63 | 626.4 | 39 | 1238.6 | 1865 | 52.5 | 61.9 | 62.4 |
| 15-18 | 63 | 53 | 626.4 | 52.5 | 1298 | 1924.4 | 57 | 62.4 | 62.5 |
| 18-21 | 53 | 43 | 518.4 | 57 | 1308.8 | 1827.2 | 50.5 | 62.5 | 62.3 |
| 21-24 | 43 | 35 | 421.2 | 50.5 | 1281.8 | 1703 | 42 | 62.3 | 62 |
| 24-27 | 35 | 29 | 345.6 | 42 | 1249.4 | 1595 | 35 | 62 | 61.8 |
| 27-30 | 29 | 25 | 291.6 | 35 | 1217 | 1508.6 | 29 | 61.8 | 61.6 |
| 30-33 | 25 | 22 | 253.8 | 29 | 1195.4 | 1449.2 | 24 | 61.6 | 61.4 |
| 33-36 | 22 | 18 | 216 | 24 | 1190 | 1406 | 20.5 | 61.4 | 61.2 |

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 36-39 | 18 | 15 | 178.2 | 20.5 | 1184.6 | 1362.8 | 17 | 61.2 | 61.05 |
| 39-42 | 15 | 13 | 151.2 | 17 | 1179.2 | 1330.4 | 14 | 61.05 | 60.9 |

## Lesson 30 Hydrologic Channel Routing

### 30.1 Introduction

In channel routing the storage is a function of both outflow and inflow discharges and hence a different routing method is needed. The flow in a river during a flood belongs to the category of gradually varied unsteady flow. The water surface in a channel reach is not only parallel to the channel bottom but also varies with time (Fig. 30.1). Considering a channel reach having a flood flow, the total volume in storage can be considered under two categories as (i) Prism storage (ii) Wedge storage.

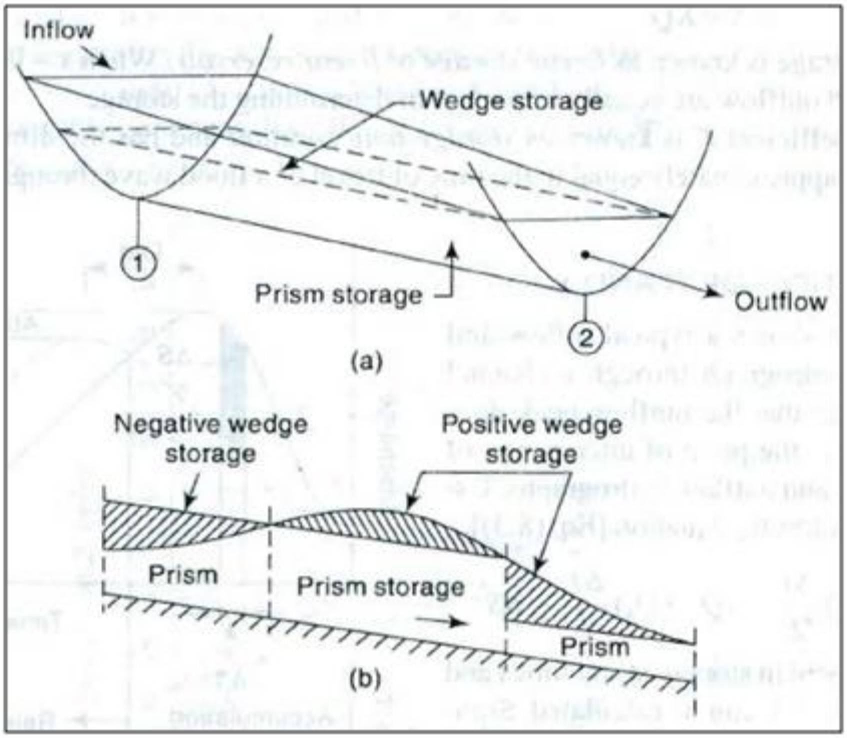


Fig. 30.1.Storage in Channel Reach. (Source: Subramanya, 2008)

#### 30.1.1Prism Storage

It is the volume that would exist if the uniform flow occurred at the downstream depth, i.e. the volume formed by an imaginary plane parallel to the channel bottom drawn at the outflow section water surface.

#### 30.1.2Wedge Storage

It is the wedge-like volume formed between the actual water surface profile and the top surface of the prism storage.

At a fixed depth at a downstream section of a river reach, the prism storage is constant while the wedge storage changes from a positive value at an advancing flood to a negative value during a receding flood.

The prism storage $S_p$is similar to a reservoir and can be expressed as a function of the outflow discharge, $S_p = f(Q)$. The wedge storage can be accounted for by expressing it as $S_w = f(I)$. The total storage in the channel reach can then be expressed as

$$S = K\,[xI^m + (1 - x)Q^m] \quad (30.1)$$

WhereK and x are coefficients and m is a constant exponent. It has been found that the value of m varies from 0.6 for rectangular channels to a value of about 1.0 for natural channels.

Muskingum Equation

Using m = 1.0, Eq. (30.1) reduces to a linear relationship for S in terms of I and Q as

$$S = K\,[xI + (1 - x)Q] \quad (30.2)$$

And this relationship is known as the Muskingum equation. In this the parameter x is known as weighting factor and takes a value between 0 and 0.5. When x = 0, the storage is a function of discharge only and Eq. (30.2) reduces to

$$S = K\,Q$$

(30.3)

Such storage is known as linear storage or linear reservoir.

The coefficient K is known as storage-time constant and has the dimensions of time. It is approximately equal to the time of travel of a flood wave through the channel reach.

Estimation of K and x

Figure 30.2 shows a typical inflow and outflow hydrograph through a channel reach.

Using the continuity equation, $\frac{I_1+I_2}{2}\Delta t - \frac{Q_1+Q_2}{2}\Delta t = S_2 - S_1$ the increment in storage at any time t and time element Δtcan be calculated. Summation of the various incremental storage values enable to find the channel storage Svstimet relationship (Fig. 30.2)

If an inflow and outflow hydrograph set isavailable for a given reach, values of S at various time intervals can be determined by the above technique. By choosing a trial value of x, values of S at any time tare plotted against the corresponding $[xI + \ (1 - x)Q]$ values. If the value of x is chosen correctly, a straight-line relationship will result. However, if an incorrect value of x is used, the plotted points will trace a looping curve. By trial and error a value of x is so chosen that the data very nearly describe a straight line. The inverse slope of this straight line will give the value of K. Normally, for natural channels, the value of x lies between 0to 0.3. For a given reach, the values of x and K are assumed to be constant.

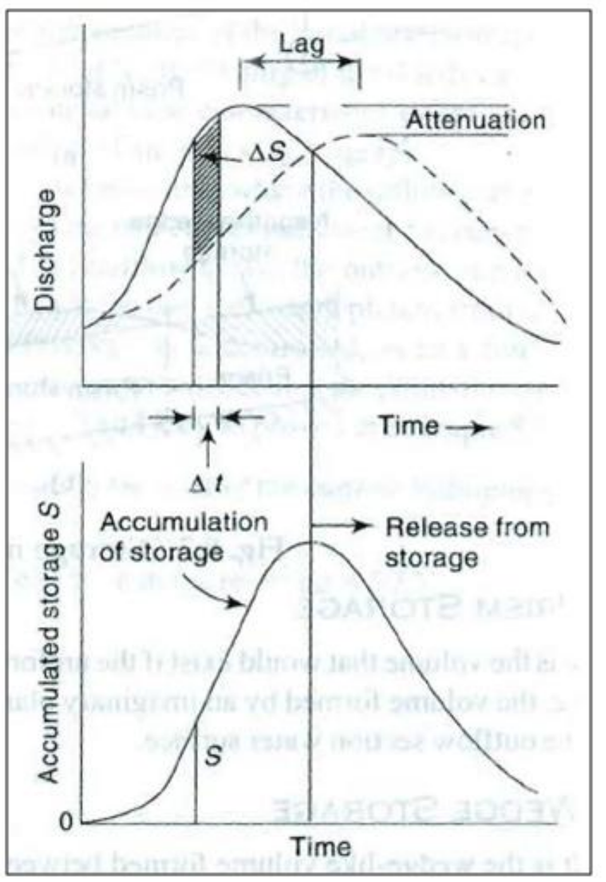


Fig. 30.2.Hydrographs and Storage in Channel Routing.

(Source: Subramanya, 2008)

## 30.2 Muskingum Method of Flood Routing

For a given channel reach by selecting a routing interval Δt and using the Muskingum equation, the change in storage is

$$S_2 - S_1 = K\,[x(I_2 - I_1) + (1-x)(Q_2 - Q_1)]$$

(30.4)

where suffixes 1 and 2 refer to the conditions before and after the time interval Δt. The continuity equation for the reach is

$$S_2 - S_1 = \frac{I_1 + I_2}{2}\Delta t - \frac{Q_1 + Q_2}{2}\Delta t$$

(30.5)

From Eqs (30.4) and (30.5),  is evaluated as

$$Q_2 = C_0 I_2 + C_1 I_1 + C_2 Q_1$$

(30.6)

where

$$C_0 = \frac{-Kx + 0.5\Delta t}{K - Kx + 0.5\Delta t}$$

(30.6a)

$$C_1 = \frac{Kx + 0.5\Delta t}{K - Kx + 0.5\Delta t}$$

(30.6b)

$$C_2 = \frac{K - Kx - 0.5\Delta t}{K - Kx + 0.5\Delta t}$$

(30.6C)

Note that C0 + C1 + C2 = 1.0, Eq. 30.6 can be written in a general form for the $n^{th}$ time step as

$$Q_n = C_0 I_n + C_1 I_{n-1} + C_2 Q_{n-1}$$

(30.6A)

Equation (30.6)isknown as Muskingum Routing Equation and provides a simple linear equation for channel routing. It has been found that for best results the routing interval Δt should be so chosen that K>Δt> 2Kx.

To use the Muskingum equation to route a given inflow hydrograph througha reach, the values of K and x for the reach and the value of the outflow, $Q_1$, from the reach at the start are needed. The procedure is described as follows

1. Knowing K and x, select an appropriate value of Δt.
2. Calculate $C_0$, $C_1$ and $C_2$.
3. Starting from the initial conditions $I_1$, $Q_1$ and known $I_2$at the end of the first time step Δt calculate $Q_2$by Eq. (30.6).
4. The outflow calculated in step (c) becomes the known initial outflow for the next lime step. Repeat the calculations for the entire inflow hydrograph.

Example 1

Route the following flood hydrograph through a river reach for which Muskingum coefficient K = 8 h and x = 0.25.

| Time (h) | 0 | 4 | 8 | 12 | 16 | 20 | 24 | 28 |
|---|---|---|---|---|---|---|---|---|
| Inflow ($m^3/s$) | 8 | 16 | 30 | 30 | 25 | 20 | 15 | 10 |

The initial outflow discharge from the reach is 8.0$m^3/s$

Answer

Given: K = 8 h; x = 0.25; Δt = 4

$$C_0 = \frac{-K \times x + 0.5 \times \Delta t}{K - K \times x + 0.5 \times \Delta t}$$

$$C_0 = \frac{-8 \times 0.25 + 0.5 \times 4}{8 - 8 \times 0.25 + 0.5 \times 4}$$

$$C_0 = 0$$

$$C_1 = \frac{K \times x + 0.5 \times \Delta t}{K - K \times x + 0.5 \times \Delta t}$$

$$C_1 = \frac{8 \times 0.25 + 0.5 \times 4}{8 - 8 \times 0.25 + 0.5 \times 4}$$

$$C_1 = 0.5$$

$$C_2 = \frac{K - K \times x - 0.5 \times \Delta t}{K - K \times x + 0.5 \times \Delta t}$$

$C_2 = 0.5$

$C_0 + C_1 + C_2 = 1.0$

$0 + 0.5 + 0.5 = 1.0$

For the first time interval, 0 to 4 h,

$I_1 = 8.0$ $C_1I_1 = 4$

$I_2 = 16.0$ $C_0I_2 = 0$

$Q_1 = 10.0$ $C_2Q_1 = 4$

$Q_2 = C_0I_2 + C_1L_1 + C_2Q_1$

$Q_2 = 0 + 4 + 4 = 8$

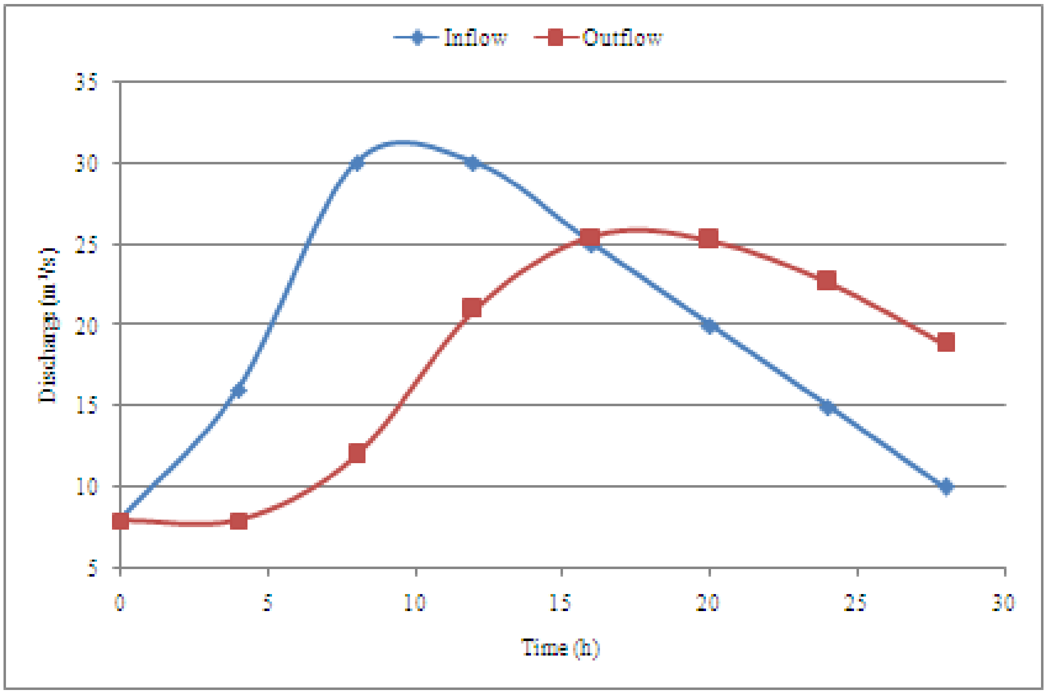
For the next time step, 4 to 8 h, $Q_1 = 8.0$ $m^3 / s$

Muskingum channel routing results for further time steps are given in following

Table

| Time (h) | Inflow | $0 \times I_2$ | $0.5 \times I_1$ | $0.5 \times Q_1$ | $Q_2 = C_0I_2 + C_1I_1 + C_2Q_1$ |
|---|---|---|---|---|---|
| 0 | 8 | | | | 8 |
| | | 0 | 4 | 4 | |

| | | | | | |
|---|---|---|---|---|---|
| 4 | 16 | | | | 8 |
| | | 0 | 8 | 4 | |
| 8 | 30 | | | | 12 |
| | | 0 | 15 | 6 | |
| 12 | 30 | | | | 21 |
| | | 0 | 15 | 10.5 | |
| 16 | 25 | | | | 25.5 |
| | | 0 | 12.5 | 12.75 | |
| 20 | 20 | | | | 25.25 |
| | | 0 | 10 | 12.625 | |
| 24 | 15 | | | | 22.625 |
| | | 0 | 7.5 | 11.3125 | |
| 28 | 10 | | | | 18.8125 |



Module 8. rought and Flood Management

## Lesson 31 Drought Management

### 31.1 Definition of Drought

In simple terms, a drought is a period of unusually dry weather that persists long enough to cause environmental or economic problems, such as crop damage and water supply shortages. But because dry conditions develop gradually and impact different regions differently, there's no agreed upon way to pinpoint when a drought begins or ends, or to objectively assess its severity.

Abnormally low rainfall is, of course, the primary cause of drought. But one can't say in general how little rainfall it takes for a region's Palmer index to sink into drought territory, because the index takes regional averages into account. This regional specificity of the categorizations makes sense in terms of land usage: Wetter regions tend to be filled more densely with people, wildlife and crops, and so more rain is required to maintain normal conditions.

Drought has many definitions, but mostly it originates from a deficiency of precipitation over an extended period of time, usually a season or more. This deficiency results in a water shortage for some activity, group, or environmental sector. Drought should be considered relative to some long term average condition of balance between precipitation and evapotranspiration (i.e., evaporation+transpiration) in a particular area, a condition often perceived as “normal”. It is also related to the timing (i.e., principal season of occurrence, delays in the start of the rainy season, occurrence of rains in relation to principal crop growth stages) and the effectiveness (i.e., rainfall intensity, number of rainfall events) of the rains. Other climatic factors such as high temperature, high wind, and low relative humidity are often associated with it in many regions of the world and can significantly aggravate its severity.

### 31.2 Classification of Drought

The wide variety of disciplines affected by drought, its diverse geographical and temporal distribution, and the many scales drought operates on make it difficult to develop both a definition to describe drought and an index to measure it. Many quantitative measures of drought have been developed in the United States, depending on the discipline affected, the region being considered, and the particular application. Several indices developed by Wayne Palmer, as well as the Standardized Precipitation Index, are useful for describing the many scales of drought. Of the many schemes for classifying droughts, the most widely used is the Palmer Drought Severity Index (PDSI), which combines temperature, precipitation, evaporation, transpiration, soil runoff and soil recharge data for a given region to produce a single negative number representing conditions there. This index serves as an estimate of soil moisture deficiency, which roughly correlates with a drought's severity, and thus, its impacts.

There are four disciplinary definitions of drought, which are as follows:

### 31.2.1 Meteorological Drought

Meteorological drought is defined usually on the basis of the degree of dryness (in comparison to some “normal” or average amount) and the duration of the dry period. Definitions of meteorological drought must be considered as region specific since theatmospheric conditions that result in deficiencies of precipitation are highly variable fromregion to region. For example, some definitions of meteorological drought identify periods of drought on the basis of the number of days with precipitation less than somespecified threshold.

### 31.2.2 Agricultural Drought

Agricultural drought links various characteristics of meteorological (or hydrological)drought to agricultural impactsfocusing on precipitation shortage, differencesbetween actual and potential evapotranspiration. Soil water deficits, reduced groundwater or reservoir levels, and so forth. Plant water demand depends on prevailingweather conditions, biological characteristics of the specific plant, its stage of growth,and the physical and biological properties of the soil. A good definition of agricultural drought should be able to account for the variable susceptibility of crops during differentstages of crop development, from emergence to maturity. Deficient topsoil moisture atplanting may hinder germination, leading to low plant populations per hectare and areduction of final yield. However, if topsoil moisture is sufficient for early growthrequirements, deficiencies in subsoil moisture at this early stage may not affect finalyield if subsoil moisture is replenished as the growing season progresses or if rainfallmeets plant water needs.

### 31.2.3 Hydrological Drought

Hydrological drought is associated with the effects of periods of precipitation (includingsnowfall) shortfalls on surface or subsurface water supply (i.e., streamflow, reservoir and lake levels, ground water). The frequency and severity of hydrological drought is often defined on a watershed or river basin scale. Although all droughts originate with adeficiency of precipitation, hydrologists are more concerned with how this deficiency plays out through the hydrologic system. Hydrological droughts are usually out of phasewith or lag the occurrence of meteorological and agricultural droughts. It takes longer for precipitation deficiencies to show up in components of the hydrological system such assoil moisture, streamflow, and ground water and reservoir levels. As a result, theseimpacts are out of phase with impacts in other economic sectors.

### 31.2.4 Socioeconomic Drought

Socioeconomic definitions of drought associate the supply and demand of someeconomic good with elements of meteorological, hydrological, and agricultural drought.It differs from the aforementioned types of drought because its occurrence depends onthe time and space processes of supply and demand to identify or classify droughts. Thesupply of many economic goods, such as water, forage, food grains, fish, andhydroelectric power, depends on weather. Because of the natural variability of climate,water supply is ample in some years but unable to meet human and environmentalneeds in other years. Socioeconomic drought occurs when the demand for economicgoods exceeds supply as a result of a weather-related shortfall in water supply.The sequence of impacts associated with meteorological, agricultural, and hydrologicaldrought further emphasizes their differences. When drought begins, the agriculturalsector is usually the first to be affected because of its heavy dependence on stored soilwater. Soil water can be rapidly depleted during extended dry periods. If precipitationdeficiencies continue, then people dependent on other sources of water will begin tofeel the effects of the shortage.

Indices for drought monitoring

1. Percent of normal
2. Standardized Precipitation Index (SPI)
3. Palmer Drought Severity Index (PDSI)
4. Crop Moisture Index (CMI)
5. Surface Water Supply Index (SWSI)
6. Reclamation Drought Index (RDI)
7. Deciles

## 31.3 Drought Management

Large variability of rainfall both in space and time, semi-arid regions are subjected to the problems of drought. The problems of arid areas wherever one good crop is not possible in normal years is quite different from those of semi-arid areas where one good crop is normally expected but it is frequently lost due to scanty rainfall or due to variability of rainfall. Even normally high rainfall areas face failure of rains and consequent upsetting of human water requirements. Water conservation and water management measures are need of the day to achieve a strong and stable economic base, especially in the arid and drought prone areas of the country. There are no general solutions possible.They will have to be area specific, because of the hydrological peculiarities. It has also to be remembered that development of drought prone areas cannot be modelled on the lines of the development of other favourably placed areas. The pattern of development of the drought-prone areas will have to be quite different from that of the others.

Some of the methods that may be suggested as technical strategies to mitigate the adversities of drought are mentioned below:

### 31.3.1 Creation of Surface Storage

Conventional approach to water conservation has been to go in for water developmentprojects – creating reservoirs by building dams, big and small, and diversion canals – tosupply water wherever and in whatever amounts desired.

### 31.3.2 Planning for Less Dependable Yield

For the drought areas, planning ofaverage flows or 50 percent dependability has been recommended by manyCommission and Committees to increase the availability of water mainly for theagricultural purposes.

### 31.3.3 Prevention of Evaporation Losses from Reservoirs

To save water in a critically water short region, an application of a layer of chemicals like cetyl,stearyl and fatty alcohol emulsions can effectively retard evaporation and savings in thefield can be around 40 percent of the normal evaporation losses.

### 31.3.4 Adjustment in Sanctioned Water to a Reservoir or Its Releases

The trend of reservoir filling or the ground water position for a water year gets fairlyknown by the middle of August. Re-adjustment of sanctions and releases has to becarefully carried out at this time keeping a close watch on the behaviour of the monsoon. The modern management techniques using probability analysis may help inassessing the situations of 'supply-variability' in the drought areas.

### 31.3.5 Reduction in Conveyance Losses

The conveyance system is an important facet of thewater conservation techniques because losses due to seepage are found to vary widelyin an irrigation system ranging from 35 percent to 45 percent of the diverted water.Lining of the canal system could be an appropriate step to conserve this preciseresource in such a situation.

### 31.3.6 Equitable Distribution

A rotational system of supplyof water if strictly implemented will not only meet the ends of equity but will alsoeconomise use of water.

### 31.3.7 Maintenance of Irrigation Systems

Over the years, maintenance of irrigation systems has deteriorated mainly due to thefact that water rates charged are not sufficient for carrying out the maintenance forkeeping the system fit and efficient.

### 31.3.8 Better Irrigation Practice

Simplemeasures like levelling of the fields so that water gets more evenly distributed cangreatly improve the performance. Wastage due to absence of field channels and lack offield levelling are now being eliminated through the Command Area Development (CAD)programmes.

### 31.3.9 Irrigation Scheduling

It is now well established that water is required more at critical stages of cropgrowth and water stress during other period has negligible impact on yields. Additionwaterings do not add proportionately more to the yield. Greater effort should be made totrain farmers in the use of irrigation scheduling methods appropriate to their mode ofproduction. Agricultural extension programmes could help spread the benefits of thesewater management techniques.

### 31.3.10 Cropping Pattern

Better water management involves all stages i.e. from pre-project formulation tooperation and maintenance. In the project formulation stage, a suitable cropping patternin conformity with soil and climatic conditions taking into account the farmerspreferences should be evolved. While designing the canal capacities, peak demand ofwater in critical periods by the high yielding varieties of crops should be kept in view.

### 31.3.11 Conjunctive use of Surface and Ground Water

The concept of conjunctive use of surface and ground water resources is very essentialespecially in drought areas in order to increase the production per unit of water. Themanner of using ground water and surface water varies considerably from region toregion. Where ground water quality is not good, canal water can be mixed in suitableproportion.

### 31.3.12 Watershed Development

Planning of watershed development involves an integrated approach uponphysiographic and hydrologic characteristics which include construction of soilconservation works on crop lands; Construction of structures, like check dams, Nallabunding, contour bunds, Gully plugging, percolation tanks, development of rainwaterharvesting and construction of wells etc.

### 31.3.13Creation of Large Storages

While planning various projects particularly in the regions depending on rainfall, it ispreferable to go in for large storages rather than a large number of small storages onthe tributaries, since small tanks are particularly vulnerable to drought. This is alsoessential in view of the fact that about 80 percent of the river flow occurs only during thefour monsoon months and this flow requires being stored for irrigation and powergeneration. The present storage capacity of all the reservoirs including major, mediumand minor schemes is 400 cubic kilometre as against the potential of 690 cubic kilometres which means the water scarcity problem may be solved to a great extent by creation of more storages.

### 31.3.14 Integrating Small Reservoirs with Major Reservoirs

There are persistent demands to abandon the schemes of large storages as it isfeared that they cause environmental disaster leading to non-sustainable developmentof water resources. Instead, numbers of small reservoirs are being advocated to replacea single large reservoir. However, in many cases, a group of small schemes may notprovide the same benefits as a large project can. It is, therefore, very important thatminor schemes are integrated with the canal systems of major reservoirs.

### 31.3.15 Transfer of Water from Water Excess Basins to Water-deficit Basins

A permanent long term solution to drought problem may be found in the basic principlesof transfer of water from surplus river basins to areas of deficit. One of the most effective ways to increase the irrigation potential for increasing the food grain production, mitigate floods and droughts and reduce regional imbalance in the availability of water is the Inter Basin Water Transfer (IBWT) from the surplus rivers to deficit areas.

# Lesson 32 Flood Management

## 32.1 Definition of Flood

According to the India National Commission on Irrigation and Drainage (INCID), 'Flood' is defined as a relatively high flow or stage in a river, marked by higher than the usual, causing inundation of low land or a body of water, rising, swelling and overflowing land that is not normally covered under water.

Flooding is caused by the inadequate capacity within the banks of the rivers to contain the high flows brought down from the upper catchment due to heavy rainfall. Areas having poor drainage characteristics get flooded by accumulation of water from heavy rainfall. Flooding is accentuated by erosion and silting of the river beds resulting in reduction of carrying capacity of river channel, earthquakes and landslides leading to changes in river courses, obstructions to flow, synchronization of floods in the main and tributary rivers and retardation due to tidal effects. Some parts of the country mainly coastal areas of Andhra Pradesh, Orissa, Tamil Nadu and West Bengal experiences cyclones which often are accompanied by heavy rainfall leading to flooding.

The damages caused by floods in terms of loss of life, property and economic loss due to disruption of economic activity are all too well-known.Floods can be mainly categorized in two types:

a) Flash floods, occurs when heavy rainfall persists only for a short time period (usually only a few hours) yet can cause major damage and death due to their sudden arrival. It can also be caused by dam brusts or overflows.

b) Riverine floods, occurs when water rises above its natural banks, often caused by prolonged and heavy precipitation. Riverine floods take days, weeks or months to rise it maximum and return to normal, much longer than it takes for a flash flood to.

### 32.1.1 When and where can flood occur?

a) Floods can occur anytime of the year.

b) In countries having monsoon season, like India, at the time of heavy rains flood often occur.

c) In countries of high latitude floods occur primarily in the spring season due to snow melting and frequent storms.

d) Flood occurs all over the world except Antarctica.

e) Highland are also unlikely to be stuck by flooding.

f) Floods usually affect floodplains, low lying flat areas near large water bodies.

### 32.1.2 Measurement of Flood Severity

There are six categories to measure the flood severity in a region as follows:

a) Depth of flow

b) Duration of flow

c) Velocity of flow

d) Rate of rise of water

e) Frequency of flood i.e. how often the floods occur in an area, which greatly depends on the topography and climate.

f) Seasonality i.e. time of year in which flood often occurs.

## _32.2 Structural Approach for Flood Management

The general approach to tackle the problem of floods in the past has been in the form of physical measures with a view to prevent the flood waters from reaching potential damage centres. The main thrust of the flood protection programme undertaken in India so far has been in the nature of taking structural measures like:

a) Embankments, flood walls, sea walls.

b) Dams and reservoir

c) Natural detention basin

d) Channel improvement

e) Drainage improvement

f) Diversion of flood waters

### 32.2.1 Embankments, Flood Walls, Sea Walls

The most common and generally economical form of protection to provide immediaterelief from inundation is construction of embankment. The embankment system along the river is planned to restrict the river in its existing course and they are designed to avoid over-flowing of banks by increasing the channel capacity to pass the probable floods. Generally, these are constructed with easily available earth in the nearby area.

Floodwall is a kind of retaining wall, constructed about parallel to the river course. The flood walls are constructed with RCC or masonary materials, depending on their availability in nearby area. Design of flood wall is done on the basis of water pressure likely to be developed on the wall. A key is provided at base for making the wall safe against sliding. For checking the overtopping of flood water, a sufficient free board is also provided to the flood wall. Sea wall is an example of flood wall. The sea walls are constructed in sea shore areas for protecting the sea bank from the soil erosion due to wave action.

### 32.2.2Dams and Reservoirs

Dams and reservoirs are constructed for storage of flood water to reduce the flood peaks. These are constructed at upstream end of the area, which is required to protect and from where the stored water can be safely discharged into the channel downstream side. The degree of flood reduction depends on the storage capacity of the reservoir. If reservoir capacity is more, then reduction in flood peak will be more and vice versa.

### 32.2.3Channel Improvement

Channel improvements involve providing of proper grade, deepening and widening of the channel section, to increase the flow carrying capacity. The main objective of this work is to decrease the stage and duration of the flood by increasing the flow velocity. The work of channel improvement is recommended for narrow and shallow depth channels having very small catchment.

### 32.2.4Drainage Improvement

Drainage improvements are used to carry out for providing relief against the recurring drainage congestion and water logging inside the urban as well as rural cropped areas.

## 32.3 Non-structural Measures for Flood Management

Structural measures can never completely eliminate the risk of flooding. Nevertheless, because of their physical presence, they have the potential to create a false sense of security, leading to inappropriate land use in the protected areas. Non-structural measures play an important role in reducing not only the catastrophic consequences of residual risks, but also adverse impacts on the environment. Non-structural flood management measures such as land use regulations; flood forecasting and warning; flood proofing; and disaster prevention, preparedness and response mechanisms; have limited environmental consequences and should be actively considered as viable options, both as independent or complementary measures.

### 32.3.1 Land use Regulations

Next to their important role in reducing the risk due to flooding land use regulations can contribute substantially to environmental preservation. For example certain land use regulations can lead to the reduction of surface runoff as well as to the conservation of ecosystems. Furthermore floodplain zoning can regulate the location of polluting industries and sewage treatment plants, thus preventing the dispersion of hazardous substantials due to flooding.

### 32.3.2 Flood Forecasting and Warning

Of all non-structural measures, flood forecasting and warning is the most widely accepted and has been used since the latter half of the 20th century. Flood forecasting allows concerned people and authorities to take preventive and emergency measures. Authorities can respond appropriately with dam operations, opening and closing the gates of various flood management structures, anticipatory releases to increase reservoir storage capacity, etc. Moreover hazardous material can be brought away from the affected area, thus preventing environmental damages.

### 32.3.3Flood Proofing

Flood proofing, a combination of long-term, non-structural and minor structural measures, as well as emergency actions, is important not only in reducing damage due to flooding, but also in preventing the negative impacts on the environment such as the spread of pollutants. It includes provision of quick drainage facilities such as the cleaning of primary and secondary drainage channels and clogged cross-drainage works before the onset of the flooding season. Moreover flood proofing measures include removing goods, equipment and harmful industrial, agricultural and domestic chemicals, beyond the area subject to flooding or out of contact with flood waters, by constructing high ground or small embankments.

### 32.3.4Emergency Preparedness, Response and Recovery

The awareness of the community at risk of flooding should be raised and maintained, with a clear understanding of their role in responding to emergency situations appropriately. This is critical in organizing coordinated evacuation from the affected area, maintaining healthy and hygienic conditions and preventing environmental pollution in the flooded areas. Flood-prone populations should be dissuaded from storing harmful chemicals during the flood season, made aware of likely pollutants in flood waters and advised of the ways to avoid their adverse impacts.