Chapter
5

RUNOFF

5.1 INTRODUCTION

Runoff means the draining or flowing off of precipitation from a catchment area through a surface channel. It thus represents the output from the catchment in a given unit of time.

Consider a catchment area receiving precipitation. For a given precipitation, the evapotranspiration, initial loss, infiltration and detention storage requirements will have to be first satisfied before the commencement of runoff. When these are satisfied, the excess precipitation moves over the land surfaces to reach smaller channels. This portion of the runoff is called *overland flow* and involves building up of a storage over the surface and draining off of the same. Usually the lengths and depths of overland flow are small and the flow is in the laminar regime. 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 location away from the point of entry into the soil. This component of runoff is known variously as *interflow, through flow, storm seepage, subsurface storm flow* or *quick return flow* (Fig. 5.1). The amount of interflow

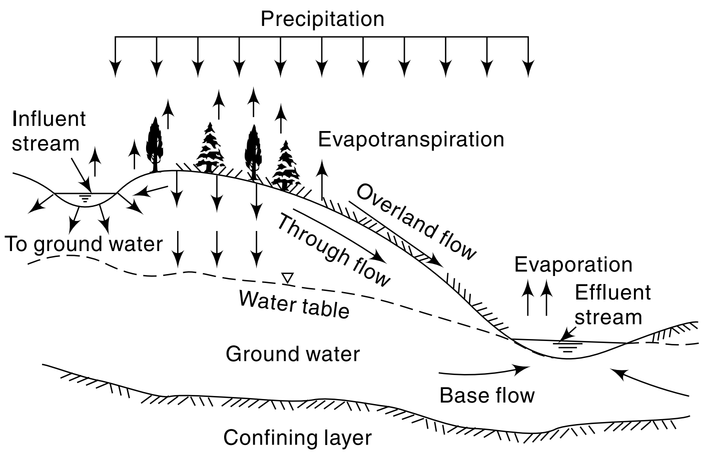

Fig. 5.1 Different routes of runoff

depends on the geological conditions of the catchment. A fairly pervious soil overlying a hard impermeable surface is conducive to large interflows. 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 in the soil. The groundwater follows a complicated and long path of travel and ultimately reaches the surface. 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.* Groundwater flow provides the dry-weather flow in perennial streams.

Based on the time delay between the precipitation and the runoff, the runoff is classified into two categories; as

1. Direct runoff, and 2. Base flow.

These are discussed below.

Direct Runoff

It is that part of the 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. Sometimes terms such as *direct storm runoff* and *storm runoff* are used to designate direct runoff. Direct runoff hydrographs are studied in detail in Chapter 6.

Base Flow

The delayed flow that reaches a stream essentially as groundwater flow is called *base flow.* Many times delayed interflow is also included under this category. In the annual hydrograph of a perennial stream (Fig. 5.2) the base flow is easily recognized as the slowly decreasing flow of the stream in rainless periods. Aspects relating to the identification of base flow in a hydrograph are discussed in Chapter 6.

Natural Flow

Runoff representing the response of a catchment to precipitation reflects the integrated effects of a wide range of catchment, climate and rainfall characteristics. True runoff is therefore stream flow in its natural condition, i.e. without human intervention. Such a stream flow unaffected by works of man, such as reservoirs and diversion structures on a stream, is called *natural flow* or *virgin flow.* When there exists storage or diversion works on a stream, the flow on the downstream channel is affected by the operational and hydraulic characteristics of these structures and hence does not represent the true runoff, unless corrected for the diversion of flow and return flow.

The natural flow (virgin flow) volume in time Δt at the terminal point of a catchment is expressed by water balance equation as

$$R_N = (R_o - V_r) + V_d + E + E_X + \Delta S \tag{5.1}$$

where R_N = Natural flow volume in time Δt

R_o = Observed flow volume in time Δt at the terminal site

V_r = Volume of return flow from irrigation, domestic water supply and industrial use

V_d = Volume diverted out of the stream for irrigation, domestic water supply and industrial use

E = net evaporation losses from reservoirs on the stream

E_X = Net export of water from the basin

ΔS = Change in the storage volumes of water storage bodies on the stream

In hydrological studies, one develops relations for natural flows. However, natural flows have to be derived based on observed flows and data on abstractions from the stream. In practice, however, the observed stream flow at a site includes return flow and is influenced by upstream abstractions. As such, natural flows have to be derived based on observed flows and data on abstractions from the stream. Always, it is the natural flow that is used in all hydrological correlations. Example 5.1 explains these aspects clearly.

Example 5.1 *The following table gives values of measured discharges at a stream–gauging site in a year. Upstream of the gauging site a weir built across the stream diverts 3.0 Mm^3 and 0.50 Mm^3 of water per month for irrigation and for use in an industry respectively. The return flows from the irrigation is estimated as 0.8 Mm^3 and from the industry at 0.30 Mm^3 reaching the stream upstream of the gauging site. Estimate the natural flow. If the catchment area is 180 km^2 and the average annual rainfall is 185 cm, determine the runoff-rainfall ratio.*

Month	1	2	3	4	5	6	7	8	9	10	11	12
Gauged flow (Mm^3)	2.0	1.5	0.8	0.6	2.1	8.0	18.0	22.0	14.0	9.0	7.0	3.0

Solution: In a month the natural flow volume R_N is obtained from Eq. (5.1) as

$$R_N = (R_o - V_r) + V_d + E + E_X + \Delta S$$

Here E, E_X and ΔS are assumed to be insignificant and of zero value.

V_r = Volume of return flow from irrigation, domestic water supply and industrial use = 0.80 + 0.30 = 1.10 Mm^3

V_d = Volume diverted out of the stream for irrigation, domestic water supply and industrial use = 3.0 + 0.5 = 3.5 Mm^3

The calculations are shown in the following Table:

Month	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**
R_o(Mm^3)	2.0	1.5	0.8	0.6	2.1	8.0	18.0	22.0	14.0	9.0	7.0	3.0
V_d(Mm^3)	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5
V_r(Mm^3)	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1
R_N(Mm^3)	4.4	3.9	3.2	3.0	4.5	10.4	20.4	24.4	16.4	11.4	9.4	5.4

Total R_N = 116.8 Mm^3

Annual natural flow volume = Annual runoff volume = 116.8 Mm^3

Area of the catchment = 180 km^2 = 1.80 × 10^8

$$\text{Annual runoff depth} = \frac{1.168 \times 10^8}{1.80 \times 10^8} = 0.649 \text{ m} = 64.9 \text{ cm}$$

Annual rainfall = 185 cm (Runoff/Rainfall) = 64.9/185 = 0.35

5.2 HYDROGRAPH

A plot of the discharge in a stream plotted against time chronologically is called a *hydrograph*. Depending upon the unit of time involved, we have

- Annual hydrographs showing the variation of daily or weekly or 10 daily mean flows over a year.
- Monthly hydrographs showing the variation of daily mean flows over a month.
- Seasonal hydrographs depicting the variation of the discharge in a particular season such as the monsoon season or dry season.
- Flood hydrographs or hydrographs due to a storm representing stream flow due to a storm over a catchment.

Each of these types have particular applications. Annual and seasonal hydrographs are of use in (i) calculating the surface water potential of stream, (ii) reservoir studies, and (iii) drought studies. Flood hydrographs are essential in analysing stream characteristics associated with floods. This chapter is concerned with the estimation and use of long-term runoffs. The study of storm hydrograph forms the subject matter of the next chapter.

Water Year

In annual runoff studies it is advantageous to consider a water year beginning from the time when the precipitation exceeds the average evapotranspiration losses. In India, June 1st is the beginning of a water year which ends on May 31st of the following calendar year. In a water year a complete cycle of climatic changes is expected and hence the water budget will have the least amount of carryover.

5.3 RUNOFF CHARACTERISTICS OF STREAMS

A study of the annual hydrographs of streams enables one to classify streams into three classes as (i) perennial, (ii) intermittent and (iii) ephemeral.

A perennial stream is one which always carries some flow (Fig. 5.2). There is considerable amount of groundwater flow throughout the year. Even during the dry seasons the water table will be above the bed of the stream.

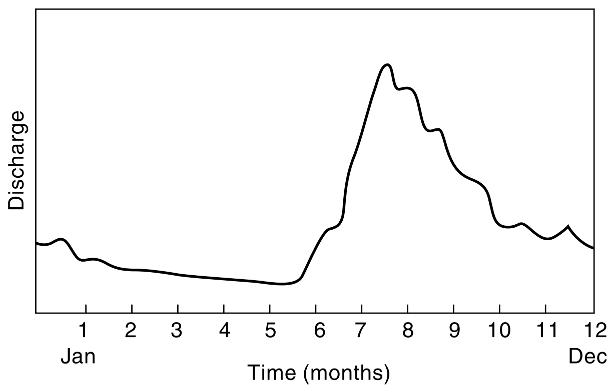

Fig. 5.2 Perennial stream

An intermittent stream has limited contribution from the groundwater. During the wet season the water table is above the stream bed and there is a contribution of the base flow to the stream flow. However, during dry seasons the water table drops to a level lower than that of the stream bed and the stream dries up. Excepting for an occasional storm which can produce a short-duration flow, the stream remains dry for the most part of the dry months (Fig. 5.3).

An ephemeral stream is one which does not have any base-flow contribution. The annual hydrograph of such a river shows series of short-duration spikes marking flash flows in response to storms (Fig. 5.4). The stream becomes dry soon after the end of

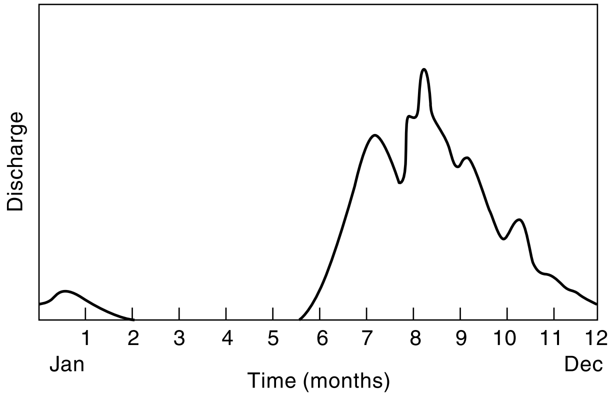

Fig. 5.3 Intermittent stream

the storm flow. Typically an ephemeral stream does not have any well-defined, channel. Most of the rivers in arid zones are of the ephemeral kind.

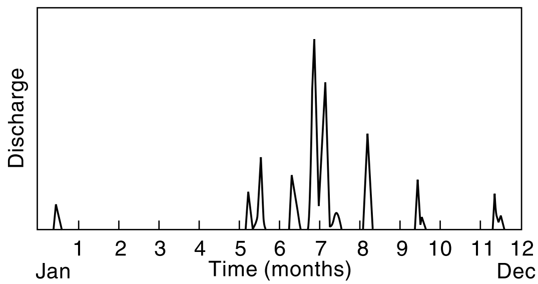

Fig. 5.4 Ephemeral stream

The flow characteristics of a stream depend upon:

- The rainfall characteristics, such as magnitude intensity, distribution according to time and space, and its variability.
- Catchment characteristics such as soil, land use/cover, slope, geology, shape and drainage density.
- Climatic factors which influence evapotranspiration.

The interrelationship of these factors is extremely complex. However, at the risk of oversimplification, the following points can be noted.

- The seasonal variation of rainfall is clearly reflected in the runoff. High stream discharges occur during the monsoon months and low flow which is essentially due to the base flow is maintained during the rest of the year.
- The shape of the stream hydrograph and hence the peak flow is essentially controlled by the storm and the physical characteristics of the basin. Evapotranspiration plays a minor role in this.
- The annual runoff volume of a stream is mainly controlled by the amount of rainfall and evapotranspiration. The geology of the basin is significant to the extent of deep percolation losses. The land use/cover play an important role in creating infiltration and evapotranspiration opportunities and retarding of runoff.

5.4 RUNOFF VOLUME

YIELD

The total quantity of surface water that can be expected in a given period from a stream at the outlet of its catchment is known as *yield* of the catchment in that period.

Depending upon the period chosen we have annual yield and seasonal yield signifying yield of the catchment in an year and in a specified season respectively. Unless otherwise qualified the term yield is usually used to represent annual yield. The term *yield* is used mostly by the irrigation engineering professionals in India.

The annual yield from a catchment is the end product of various processes such as precipitation, infiltration and evapotranspiration operating on the catchment. Due to the inherent nature of the various parameters involved in the processes, the yield is a random variable. A list of values of annual yield in a number of years constitutes an annual time series which can be analyzed by methods indicated in Chapter 2 (Sec. 2.11) to assign probabilities of occurrences of various events. A common practice is to assign a dependability value (say 75% dependable yield) to the yield. Thus, 75% dependable annual yield is the value that can be expected to be equalled to or exceeded 75% times (i.e. on an average 15 times in a span of 20 years). Similarly, 50% dependable yield is the annual yield value that is likely to be equalled or exceeded 50% of times (i.e. on an average 10 times in 20 years).

It should be remembered that the yield of a stream is always related to the natural flow in the river. However, when water is diverted from a stream for use in activities such as irrigation, domestic water supply and industries, the non-consumptive part of the diverted water returns back to the hydrologic system of the basin. Such additional flow, known as *return flow*, is available for the suitable use and as such is added to the natural flow to estimate the yield. (Details pertaining to the return flow are available in Sec. 5.9). The annual yield of a basin at a site is thus taken as the annual natural water flow in the river at the site plus the return flow to the stream from different uses upstream of the site.

The yield of a catchment Y in a period Δt could be expressed by water balance equation (Eq. 5.1) as

$$Y = R_N + V_r = R_o + A_b + \Delta S \tag{5.1a}$$

where R_N = Natural flow in time Δt

V_r = Volume of return flow from irrigation, domestic water supply and industrial use

R_o = Observed runoff volume at the terminal gauging station of the basin in time Δt.

A_b = Abstraction in time, Δt for irrigation, water supply and industrial use and inclusive of evaporation losses in surface water bodies on the stream.

ΔS = Change in the storage volumes of water storage bodies on the stream.

The calculation of natural runoff volume (and hence yield), is of fundamental importance in all surface water resources development studies. The most desirable basis for assessing the yield characteristics of a catchment is to analyze the actual flow records of the stream draining the catchment. However, in general, observed discharge data of sufficient length is unlikely to be available for many catchments. As such, other alternate methods such as the *empirical equations* and *watershed simulations* (described in Secs 5.4.3 to 5.4.5) are often adopted.

It should be noted that the observed stream flow at a site includes return flow. For small catchments and for catchments where water resources developments are at a small scale, the return flow is likely to be a negligibly small part of the runoff. In the further parts of this chapter the term annual (or seasonal) runoff volume R and the term annual (or seasonal) yield are used synonymously with the implied assumption that the return flow is negligibly small. It is emphasized that when return flow is not negligible, it is the natural flow volume that is to be used in hydrological correlations with rainfall.

Rainfall—Runoff Correlation

The relationship between rainfall in a period and the corresponding runoff is quite complex and is influenced by a host of factors relating to the catchment and climate. Further, there is the problem of paucity of data which forces one to adopt simple correlations for adequate estimation of runoff. One of the most common methods is to correlate seasonal or annual measured runoff values *(R)* with corresponding rainfall *(P)* values. A commonly adopted method is to fit a linear regression line between R and P and to accept the result if the correlation coefficient is nearer unity. The equation of the straight-line regression between runoff R and rainfall P is

$$R = aP + b \tag{5.2}$$

and the values of the coefficient a and b are given by

$$a = \frac{N(\Sigma PR) - (\Sigma P)(\Sigma R)}{N(\Sigma P^2) - (\Sigma P)^2} \tag{5.3a}$$

and

$$b = \frac{\Sigma R - a(\Sigma P)}{N} \tag{5.3b}$$

in which N = number of observation sets R and P. The coefficient of correlation r can be calculated as

$$r = \frac{N(\Sigma PR) - (\Sigma P)(\Sigma R)}{\sqrt{[N(\Sigma P^2) - (\Sigma P)^2][N(\Sigma R^2) - (\Sigma R)^2]}} \tag{5.4}$$

The value of r lies between 0 and 1 as R can have only positive correlation with P. The value of $0.6 < r < 1.0$ indicates good correlation. Further, it should be noted that $R \geq 0$.

For large catchments, sometimes it is found advantageous to have exponential relationship as

$$R = \beta P^m \tag{5.5}$$

where β and m are constants, instead of the linear relationship given by Eq. (5.2). In that case Eq. (5.5) is reduced to linear form by logarithmic transformation as

$$\ln R = m \ln P + \ln \beta \tag{5.6}$$

and the coefficients m and $\ln \beta$ are determined by using methods indicated earlier.

Since rainfall records of longer periods than that of runoff data are normally available for a catchment, the regression equation [Eq. (5.2) or (5.5)] can be used to generate synthetic runoff data by using rainfall data. While this may be adequate for preliminary studies, for accurate results sophisticated methods are adopted for synthetic generation of runoff data. Many improvements of the above basic rainfall-runoff correlation by considering additional parameters such as soil moisture and antecedent rainfall have been attempted. Antecedent rainfall influences the initial soil moisture and hence the infiltration rate at the start of the storm. For calculation of the annual runoff from the annual rainfall a commonly used antecedent precipitation index P_a is given by

$$P_a = aP_i + bP_{i-1} + cP_{i-2} \tag{5.7}$$

where P_i, p_{i-1} and P_{i-2} are the annual precipitation in the i^{th}, $(i-1)^{th}$ and $(i-2)^{th}$ year and i = current year, a, b and c are the coefficients with their sum equal to unity. The coefficients are found by trial and error to produce best results. There are many other types of antecedent precipitation indices in use to account for antecedent soil moisture condition. For example, in *SCS – CN* method (Sec. 5.4.5) the sum of past five-day rainfall is taken as the index of antecedent moisture condition.

Example 5.2 *Annual rainfall and runoff values (in cm) of a catchment spanning a period of 21 years are given below. Analyze the data to (a) estimate the 75% and 50% dependable annual yield of the catchment and (b) to develop a linear correlation equation to estimate annual runoff volume for a given annual rainfall value.*

Year	Annual rainfall (cm)	Annual runoff (cm)	Year	Annual rainfall (cm)	Annual runoff (cm)
1975	118	54	1986	75	17
1976	98	45	1987	107	32
1977	112	51	1988	75	15
1978	97	41	1989	93	28
1979	84	21	1990	129	48
1980	91	32	1991	153	76
1981	138	66	1992	92	27
1982	89	25	1993	84	18
1983	104	42	1994	121	52
1984	80	11	1995	95	26
1985	97	32			

Solution: (a) The annual runoff values are arranged in descending order of magnitude and a rank (m) is assigned for each value starting from the highest value (Table 5.1). The exceedence probability p is calculated for each runoff value as $p = \frac{m}{N+1}$. In this m = rank number and N = number of data sets. (Note that in Table 5.1 three items have the same value of R = 32 cm and for this set p is calculated for the item having the highest value of m, i.e m = 12). For estimating 75% dependable yield, the value of p = 0.75 is read from Table 5.1 by linear interpolation between items having p = 0.773 and p = 0.727. By this method, the 75% dependable yield for the given annual yield time series is found to be R_{75} = 23.0 cm.

Similarly, the 50% dependable yield is obtained by linear interpolation between items having p = 0.545 and p = 0.409 as R_{50} = 34.0 cm.

(b) The correlation equation is written as $R = aP + b$

The coefficients of the best fit straight line for the data are obtained by the least square error method as mentioned in Table 5.1.

From the Table 5.1,

$$\Sigma P = 2132 \qquad \Sigma R = 759 \qquad \Sigma PR = 83838$$

$$\Sigma P^2 = 224992 \qquad \Sigma R^2 = 33413$$

$$(\Sigma P)^2 = 4545424 \qquad (\Sigma R)^2 = 576081 \qquad N = 21$$

By using Eq. (5.3-a)

$$a = \frac{N(\Sigma PR) - (\Sigma P)(\Sigma R)}{N(\Sigma P^2) - (\Sigma R)^2} = \frac{(21 \times 83838) - (2132)(759)}{(21 \times 224992) - (2132)^2} = 0.7938$$

Table 5.1 Calculations for Example 5.2

1	2	3	4	5	6	7	8	9
Year	P rainfall (cm)	R runoff (cm)	P^2	R^2	PR	rank, m	R (Sorted annual runoff) (cm)	Exceedence probability, p
1975	118	54	13924	2916	6372	1	76	0.045
1976	98	45	9604	2025	4410	2	66	0.091
1977	112	51	12544	2601	5712	3	54	0.136
1978	97	41	9409	1681	3977	4	52	0.182
1979	84	21	7056	441	1764	5	51	0.227
1980	91	32	8281	1024	2912	6	48	0.273
1981	138	66	19044	4356	9108	7	45	0.318
1982	89	25	7921	625	2225	8	42	0.364
1983	104	42	10816	1764	4368	9	41	0.409
1984	80	11	6400	121	880	10	32	
1985	97	32	9409	1024	3104	11	32	
1986	75	17	5625	289	1275	12	32	0.545
1987	107	32	11449	1024	3424	13	28	0.591
1988	75	15	5625	225	1125	14	27	0.636
1989	93	28	8649	784	2604	15	26	0.682
1990	129	48	16641	2304	6192	16	25	0.727
1991	153	76	23409	5776	11628	17	21	0.773
1992	92	27	8464	729	2484	18	18	0.818
1993	84	18	7056	324	1512	19	17	0.864
1994	121	52	14641	2704	6292	20	15	0.909
1995	95	26	9025	676	2470	21	11	0.955
SUM	**2132**	**759**	**224992**	**33413**	**83838**			

By Eq. (5.3-b)

$$b = \frac{\Sigma R - a(\Sigma P)}{N} = \frac{(759) - 0.7938 \times (2138)}{21} = -44.44$$

Hence the required annual rainfall–runoff relationship of the catchment is given by

$R = 0.7938\,P - 44.44$ with both P and R being in cm and $R \geq 0$.

By Eq. (5.4) coefficient of correlation

$$r = \frac{N(\Sigma PR) - (\Sigma P)(\Sigma R)}{\sqrt{[N(\Sigma P^2) - (\Sigma P)^2][N(\Sigma R^2) - (\Sigma R)^2]}}$$

$$= \frac{(21 \times 83838 - (2132)(759)}{\sqrt{[(21 \times 224992) - (4545424)][(21 \times 33413) - (576081)}} = 0.949$$

As the value of r is nearer to unity the correlation is very good. Figure 5.5 represents the data points and the best fit straight line.

Empirical Equations

The importance of estimating the water availability from the available hydrologic data for purposes of planning water-resource projects was recognised by engineers even in

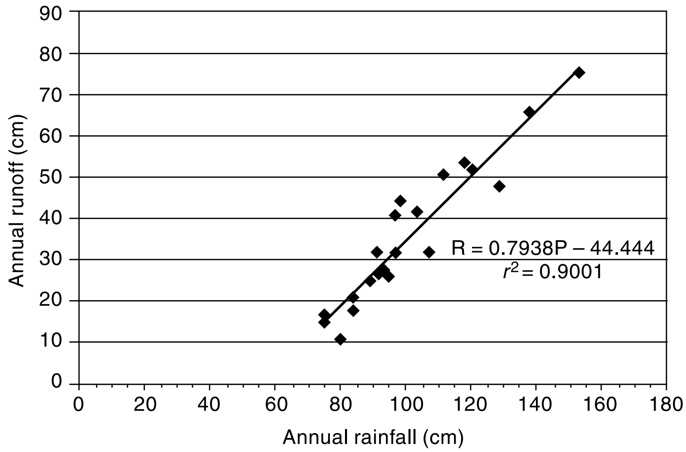

Fig. 5.5 Annual Rainfall–Runoff Correlation—Example 5.2

the last century. With a keen sense of observation in the region of their activity many engineers of the past have developed empirical runoff estimation formulae. However, these formulae are applicable only to the region in which they were derived. These formulae are essentially rainfall–runoff relations with additional third or fourth parameters to account for climatic or catchment characteristics. Some of the important formulae used in various parts of India are given below.

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

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

$$R = K_b\, P \tag{5.8}$$

where K_b = runoff coefficient which depends upon the type of catchment and nature of monsoon rainfall. Values of K_b are given in Table 5.2.

Table 5.2 Barlow's Runoff Coefficient K_b in Percentage (Developed for use in UP)

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

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

Strange'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 are classified as *good, average* and *bad* according to the relative magnitudes of yield they give. For example, catchments with good forest/vegetal cover and having soils of high permeability would be classified as *bad,* while catchments having soils of low permeability and having little or no vegetal cover is termed *good.* Two methods using tables for estimating the runoff volume in a season are given.

1. 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)* are given in Table 5.3-a. The correlation equations of best fitting lines relating percentage yield ratio (Y_r) to precipitation (P) could be expressed as

Table 5.3(a) Strange's Table of Total Mansoon Rainfall and estimated Runoff

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

(*Contd.*)

(*Contd.*)

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

For *Good* catchment:

For $P < 250$ mm, $\quad Y_r = 7 \times 10^{-5} P^2 - 0.0003\, P$ having $r^2 = 0.9994$ (5.9a)

For $250 < P < 760$ $\quad Y_r = 0.0438\, P - 7.1671$ having $r^2 = 0.9997$ (5.9b)

For $760 < P < 1500$ $\quad Y_r = 0.0443\, P - 7.479$ having $r^2 = 1.0$ (5.9c)

For *Average* catchment:

For $P < 250$ mm, $\quad Y_r = 6 \times 10^{-5} P^2 - 0.0022\, P + 0.1183$ having $r^2 = 0.9989$ (5.10a)

For $250 < P < 760$ $\quad Y_r = 0.0328\, P - 5.3933$ having $r^2 = 0.9997$ (5.10b)

For $760 < P < 1500$ $\quad Y_r = 0.0333\, P - 5.7101$ having $r^2 = 0.9999$ (5.10c)

For *Bad* catchment:

For $P < 250$ mm, $\quad Y_r = 4 \times 10^{-5} P^2 - 0.0011\, P + 0.0567$ having $r^2 = 0.9985$ (5.11a)

For $250 < P < 760$ $\quad Y_r = 0.0219\, P - 3.5918$ having $r^2 = 0.9997$ (5.11b)

For $760 < P < 1500$ $\quad Y_r = 0.0221\, P - 3.771$ having $r^2 = 1.0$ (5.11c)

where Y_r = Percentage yield ratio = ratio of seasonal runoff to seasonal rainfall in percentage and P = monsoon season rainfall in mm.

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.

Example 5.3 illustrates this procedure.

2. 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 events 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:

Wetting Process

(a) **Transition from Dry to Damp**
- (i) 6 mm rainfall in the last 1 day
- (ii) 12 mm in the last 3 days
- (iii) 25 mm in the last 7 days
- (iv) 38 mm in the last 10 days

(b) **Transition from Damp to Wet**
- (i) 8 mm rainfall in the last 1 day
- (ii) 12 mm in the last 2 days
- (iii) 25 mm in the last 3 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.

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 1 day (iii) 12 mm in the last 7 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 5.3(b). For *good* (yield producing) and *bad* (yield producing) catchments *add* or *deduct* 25% of the yield corresponding to the *average* catchment.

Table 5.3(b) Strange's Table of Runoff Volume from Daily Rainfall for an *Average* Catchment

Daily rainfall (mm)	Percentage of 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

Best fitting linear equations for the above table would read as below with K_s = runoff volume percentage and P = daily rainfall (mm):

For Dry AMC: $K_s = 0.5065\ P - 2.3716$ for $P > 20$ mm (5.12a)
with coefficient of determination $r^2 = 0.9947$

For Damp AMC: $K_s = 0.3259\ P - 5.1079$ for $P > 7$ mm (5.12b)
with coefficient of determination $r^2 = 0.9261$

For Wet AMC: $K_s = 0.6601\ P + 2.0643$ (5.12c)
with coefficient of determination $r^2 = 0.9926$

Use of Strange's Tables Strange's monsoon rainfall-runoff table (Table 5.3-a) and Table (5.3-b) for estimating daily runoff corresponding to a daily rainfall event are in use in parts of Karnataka, Andhra Pradesh and Tamil Nadu. A calculation procedure using Table (5.3-a) to calculate monthly runoff volumes in a monsoon season using cumulative monthly rainfalls is shown in Example 5.3.

Example 5.3 *Monthly rainfall values of the 50% dependable year at a site selected for construction of an irrigation tank is given below. Estimate the monthly and annual runoff volume of this catchment of area 1500 ha.*
[Assume the catchment classification as Good catchment].

Month	June	July	Aug	Sept	Oct
Monthly rainfall (mm)	90	160	145	22	240

SOLUTION: Calculations are shown in the Table 5.4 given below.

Table 5.4 Calculation of Monthly Yields by Strange's Method—Example 5.3

No.	Month	June	July	August	September	October
1.	Monthly Rainfall (mm)	90	160	145	22	240
2.	Cumulative monthly rainfall (mm)	90	250	395	417	657
3.	Runoff/rainfall as % (From Strange's Table 5.3-a)	0.56	4.17	10.01	11.08	21.69
4.	Cumulative Runoff (mm)	0.50	10.43	39.54	46.20	142.50
5.	Monthly Runoff (mm)	0.50	9.92	29.11	6.66	96.30

Row 4 is obtained by using Strange's Tables 5.3. Note that cumulative monthly rainfall is used to get the cumulative runoff-ratio percentage at any month.

Total monsoon runoff = 142.50 mm = $(142.5/1000) \times (1500 \times 10^4)/10^6$ Mm^3
= 2.1375 Mm^3

Annual Runoff is taken as equal to monsoon runoff.

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

$$R = 0.85\,P - 30.5 \tag{5.13}$$

2. For Deccan plateau

$$R = \frac{1}{254}\,P\,(P - 17.8) \tag{5.14}$$

KHOSLA'S FORMULA Khosla (1960) analysed 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

$$R_m = P_m - L_m \tag{5.15}$$

and $L_m = 0.48\,T_m$ for $T_m > 4.5°$ C

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 L_m may provisionally be assumed as

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

Annual runoff = ΣR_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.

EXAMPLE 5.4 For a catchment in UP, India, the mean monthly temperatures are given. Estimate the annual runoff and annual runoff coefficient by Khosla's method.

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Temp°C	12	16	21	27	31	34	31	29	28	29	19	14
Rainfall (P_m)(cm)	4	4	2	0	2	12	32	29	16	2	1	2

SOLUTION: In Khosla's formula applicable to the present case, $R_m = P_m - L_m$ with L_m = (0.48 × T °C) having a maximum value equal to corresponding P_m. The calculations are shown below:

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Rainfall (P_m)(cm)	4	4	2	0	2	12	32	29	16	2	1	2
Temp°C	12	16	21	27	31	34	31	29	28	29	19	14
L_m (cm)	4	4	2	0	2	12	14.9	13.9	13.4	2	1	2
Runoff (R_m)(cm)	0	0	0	0	0	0	17.1	15.1	2.6	0	0	0

Total annual runoff = 34.8 cm

Annual runoff coefficient = (Annual runoff/Annual rainfall) = (34.8/116.0) = 0.30

WATERSHED SIMULATION The hydrologic water-budget equation for the determination of runoff for a given period is written as

$$R = R_s + G_0 = P - E_{et} - \Delta S \tag{5.16}$$

in which R_s = surface runoff, P = precipitation, E_{et} = actual evapotranspiration, G_0 = net groundwater outflow and ΔS = change in the soil moisture storage. The sum of R_s and G_0 is considered to be given by the total runoff R, i.e. streamflow.

Starting from an initial set of values, one can use Eq. (5.16) to calculate R by knowing values of P and functional dependence of E_{et}, ΔS and infiltration rates with catchment and climatic conditions. For accurate results the functional dependence of various parameters governing the runoff in the catchment and values of P at short time intervals are needed. Calculations can then be done sequentially to obtain the runoff at any time. However, the calculation effort involved is enormous if attempted manually. With the availability of digital computers the use of water budgeting as above to determine the runoff has become feasible. This technique of predicting the runoff, which is the catchment response to a given rainfall input is called *deterministic watershed simulation.* In this the mathematical relationships describing the interdependence of various parameters in the system are first prepared and this is called the *model.* The model is then calibrated, i.e. the numerical values of various coefficients determined by simulating the known rainfall-runoff records. The accuracy of the model is further checked by reproducing the results of another string of rainfall data for which runoff values are known. This phase is known as *validation* or *verification* of the model. After this, the model is ready for use.

Crawford and Linsley (1959) pioneered this technique by proposing a watershed simulation model known as the Stanford Watershed Model (SWM). This underwent successive refinements and the Stanford Watershed Model-IV (SWM-IV) suitable for use on a wide variety of conditions was proposed in 1966. The flow chart of SWM-IV is shown in Fig. 5.6. The main inputs are hourly precipitation and daily evapotranspiration in addition to physical description of the catchment. The model considers the soil in three zones with distinct properties to simulate evapotranspiration, infiltration, overland flow, channel flow, interflow and baseflow phases of the runoff phenomenon. For calibration about 5 years of data are needed. In the calibration phase, the initial guess value of parameters are adjusted on a trial-and-error basis until the simulated response matches the recorded values. Using an additional length of rain-fall-runoff of about 5 years duration, the model is verified for its ability to give proper response. A detailed description of the application of SWM to an Indian catchment is given in Ref. 11.

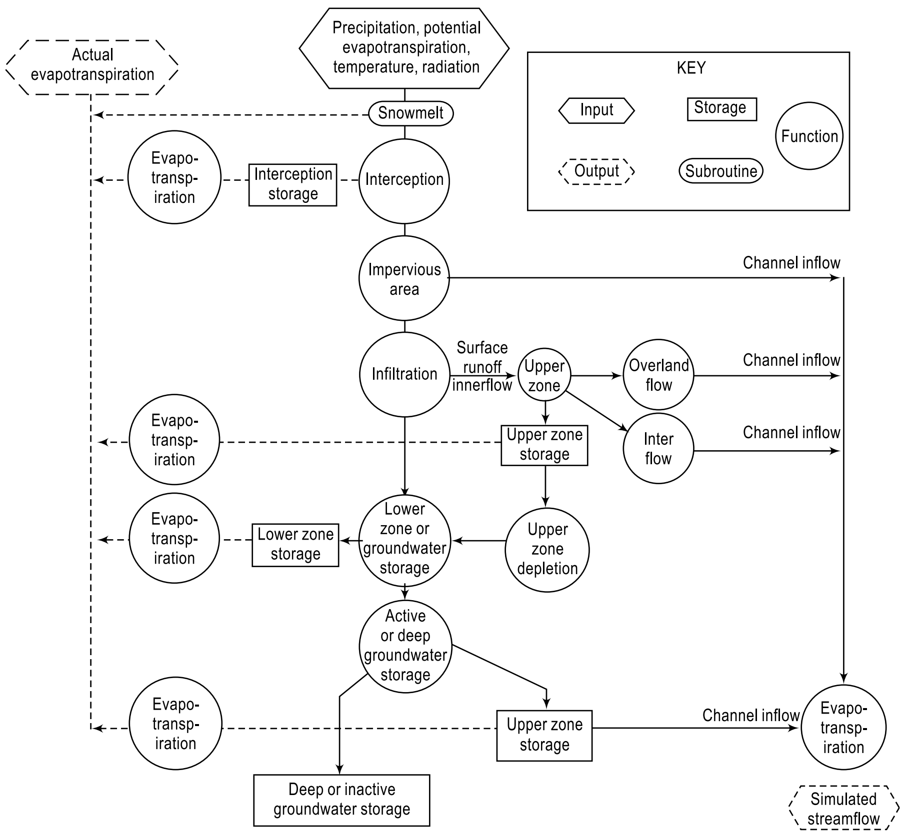

Fig. 5.6 Flow chart of SWM-IV

Based on the logic of SWM-IV many models and improved versions such as HSP (1966), SSARR (1968) and KWM (1970) were developed during late sixties and seventies. These models which simulate stream flow for long periods of time are called

Continuous Simulated Models. They permit generation of simulated long records for yield, drought and flood flow studies. In the early 1980s there were at least 75 hydrologic simulation models that were available and deemed suitable for small watersheds. In the past two decades considerable effort has been directed towards the development of process-based, spatially explicit, and physically-based models such as MIKE SHE (Refsgaard and Storm, 1955), and GSSHA—Gridded Surface/Subsurface Hydrologic Analysis (Downer et al., 2006). These are new generation of models that utilize GIS technology.

SCS-CN Method of Estimating Runoff Volume

SCS-CN method, developed by Soil Conservation Services (SCS) of USA in 1969, is a simple, predictable, and stable conceptual method for estimation of direct runoff depth based on storm rainfall depth. It relies on only one parameter, *CN*. Currently, it is a well-established method, having been widely accepted for use in USA and many other countries. The details of the method are described in this section.

Basic Theory The *SCS-CN* method is based on the water balance equation of the rainfall in a known interval of time Δt, which can be expressed as

$$P = I_a + F + Q \tag{5.17}$$

where P = total precipitation, I_a = initial abstraction, F = Cumulative infiltration excluding I_a and Q = direct surface runoff (all in units of volume occurring in time Δt). Two other concepts as below are also used with Eq. (5.17).

(i) The first concept is that the ratio of actual amount of direct runoff (Q) to maximum potential runoff (= $P - I_a$) is equal to the ratio of actual infiltration (F) to the potential maximum retention (or infiltration), S. This proportionality concept can be schematically shown as in Fig. 5.7

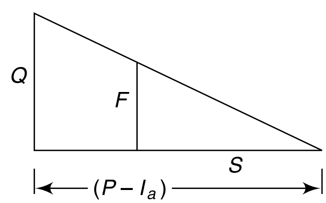

Fig. 5.7 Proportionality concept

Thus $$\frac{Q}{P - I_a} = \frac{F}{S} \tag{5.18}$$

(ii) The second concept is that the amount of initial abstraction (I_a) is some fraction of the potential maximum retention (S).

Thus $$I_a = \lambda S \tag{5.19}$$

Combining Eqs. (5.18) and (5.19), and using (5.17)

$$Q = \frac{(P - I_a)^2}{P - I_a + S} = \frac{(P - \lambda S)^2}{P + (1 - \lambda)S} \quad \text{for } P > \lambda S \tag{5.20a}$$

Further $$Q = 0 \text{ for } P \le \lambda S \tag{5.20b}$$

For operation purposes a time interval Δt = 1 day is adopted. Thus P = daily rainfall and Q = daily runoff from the catchment.

Curve Number (CN) The parameter S representing the potential maximum retention depends upon the soil–vegetation–land use complex of the catchment and also upon the antecedent soil moisture condition in the catchment just prior to the commencement of the rainfall event. For convenience in practical application the Soil Conservation Services (SCS) of USA has expressed S (in mm) in terms of a dimensionless parameter CN (the Curve number) as

$$S = \frac{25400}{CN} - 254 = 254\left(\frac{100}{CN} - 1\right) \tag{5.21}$$

The constant 254 is used to express S in mm.

The curve number CN is now related to S as

$$CN = \frac{25400}{S + 254} \tag{5.22}$$

and has a range of $100 \geq CN \geq 0$. A CN value of 100 represents a condition of zero potential retention (i.e. impervious catchment) and $CN = 0$ represents an infinitely abstracting catchment with $S = \infty$. This curve number CN depends upon

- Soil type
- Land use/cover
- Antecedent moisture condition

Soils In the determination of *CN,* the hydrological soil classification is adopted. Here, soils are classified into four classes A, B, C and D based upon the infiltration and other characteristics. The important soil characteristics that influence hydrological classification of soils are effective depth of soil, average clay content, infiltration characteristics and permeability. Following is a brief description of four hydrologic soil groups:

- **Group-A**: **(Low Runoff Potential)**: Soils having high infiltration rates even when thoroughly wetted and consisting chiefly of deep, well to excessively drained sands or gravels. These soils have high rate of water transmission. [Example: Deep sand, Deep loess and Aggregated silt]
- **Group-B: (Moderately Low runoff Potential)**: Soils having moderate infiltration rates when thoroughly wetted and consisting chiefly of moderately deep to deep, moderately well to well-drained soils with moderately fine to moderately coarse textures. These soils have moderate rate of water transmission. [Example: Shallow loess, Sandy loam, Red loamy soil, Red sandy loam and Red sandy soil]
- **Group-C: (Moderately High Runoff Potential)**: Soils having low infiltration rates when thoroughly wetted and consisting chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These soils have moderate rate of water transmission. [Example: Clayey loam, Shallow sandy loam, Soils usually high in clay, Mixed red and black soils]
- **Group-D: (High Runoff Potential)**: Soils having very low infiltration rates when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high-water table, soils with a clay pan, or clay layer at or near the surface, and shallow soils over nearly impervious material. [Example: Heavy plastic clays, certain saline soils and deep black soils].

Antecedent Moisture Condition (AMC) Antecedent Moisture Condition (AMC) refers to the moisture content present in the soil at the beginning of the rainfall-runoff event under consideration. It is well known that initial abstraction and infiltration are governed by AMC. For purposes of practical application three levels of AMC are recognized by SCS as follows:

AMC-I: Soils are dry but not to wilting point. Satisfactory cultivation has taken place.

AMC-II: Average conditions

AMC-III: Sufficient rainfall has occurred within the immediate past 5 days. Saturated soil conditions prevail.

The limits of these three AMC classes, based on total rainfall magnitude in the previous 5 days, are given in Table 5.5. It is to be noted that the limits also depend upon the seasons: two seasons, viz. growing season and dormant season are considered.

Table 5.5 Antecedent Moisture Conditions (AMC) for Determining the Value of CN

AMC Type	Total Rain in Previous 5 days	
	Dormant Season	Growing Season
I	Less than 13 mm	Less than 36 mm
II	13 to 28 mm	36 to 53 mm
III	More than 28 mm	More than 53 mm

LAND USE The variation of CN under AMC-II, called CN_{II}, for various land use conditions commonly found in practice are shown in Table 5.6(a, b and c).

Table 5.6(a) Runoff Curve Numbers [CN_{II}] for Hydrologic Soil Cover Complexes [Under AMC-II Conditions]

Land Use	Cover		Hydrologic soil group			
	Treatment or practice	Hydrologic condition	A	B	C	D
Cultivated	Straight row		76	86	90	93
Cultivated	Contoured	Poor	70	79	84	88
		Good	65	75	82	86
Cultivated	Contoured &	Poor	66	74	80	82
	Terraced	Good	62	71	77	81
Cultivated	Bunded	Poor	67	75	81	83
		Good	59	69	76	79
Cultivated	Paddy		95	95	95	95
Orchards	With understory cover		39	53	67	71
	Without understory cover		41	55	69	73
Forest	Dense		26	40	58	61
	Open		28	44	60	64
	Scrub		33	47	64	67
Pasture	Poor		68	79	86	89
	Fair		49	69	79	84
	Good		39	61	74	80
Wasteland			71	80	85	88
Roads (dirt)			73	83	88	90
Hard surface areas			77	86	91	93

[Source: Ref.7]

Note: Sugarcane has a separate supplementary Table of CN_{II} values (Table 5.6(b)).

The conversion of CN_{II} to other two AMC conditions can be made through the use of following correlation equations.[10]

For AMC-I: $$CN_I = \frac{CN_{II}}{2.281 - 0.01281\, CN_{II}} \qquad (5.23)$$

Table 5.6(b) CN_{II} Values for Sugarcane

[Source: Ref.7]

Cover and treatment	Hydrologic soil group			
	A	B	C	D
Limited cover, Straight Row	67	78	85	89
Partial cover, Straight row	49	69	79	84
Complete cover, Straight row	39	61	74	80
Limited cover, Contoured	65	75	82	86
Partial cover, Contoured	25	59	45	83
Complete cover, Contoured	6	35	70	79

Table 5.6(c) CN_{II} Values for Suburban and Urban Land Uses (Ref. 3)

Cover and treatment	Hydrologic soil group			
	A	B	C	D
Open spaces, lawns, parks etc				
(i) In good condition, grass cover in more than 75% area	39	61	74	80
(ii) In fair condition, grass cover on 50 to 75% area	49	69	79	84
Commercial and business areas (85% impervious)	89	92	94	95
Industrial Districts (72% impervious)	81	88	91	93
Residential, average 65% impervious	77	85	90	92
Paved parking lots, paved roads with curbs, roofs, driveways, etc	98	98	98	98
Streets and roads				
Gravel	76	85	89	91
Dirt	72	82	87	89

For AMC-III:
$$CN_{\text{III}} = \frac{CN_{II}}{0.427 + 0.00573\, CN_{\text{II}}} \tag{5.24}$$

The equations (5.23) and (5.24) are applicable in the CN_{II}, range of 55 to 95 which covers most of the practical range. Values of CN_{I}, and CN_{III} covering the full range of CN_{II} are available in Refs 3 and 7. Procedures for evaluation of CN from data on small watersheds are available in Ref. 7.

VALUE OF λ On the basis of extensive measurements in small size catchments SCS (1985) adopted $\lambda = 0.2$ as a standard value. With this Eq. (5.20-a) becomes

$$Q = \frac{(P - 0.2\,S)^2}{P + 0.8\,S} \quad \text{for } P > 0.2S \tag{5.25}$$

where Q = daily runoff, P = daily rainfall and S = retention parameter, all in units of mm. Equation 5.25, which is well established, is called as the *Standard SCS-CN equation.*

SCS-CN EQUATION FOR INDIAN CONDITIONS Values of λ varying in the range $0.1 \le \lambda \le 0.4$ have been documented in a number of studies from various geographical locations, which include USA and many other countries. For use in Indian conditions $\lambda = 0.1$ and 0.3 subject to certain constraints of soil type and AMC type has been recommended (Ref. 7) as below:

$$Q = \frac{(P - 0.1\,S)^2}{P + 0.9\,S} \quad \text{for } P > 0.1\,S,$$

valid *for Black soils under AMC of Type II and III* (5.26)

$$Q = \frac{(P - 0.3\,S)^2}{P + 0.7\,S} \quad \text{for } P > 0.3S \text{ valid}$$ *for Black soils under AMC of Type 1 and for all other soils having AMC of types I, II and III* (5.27)

These Eqs. (5.26 & 5.27) along with Table 5.6 (a & b) are recommended (Ref. 7) for use in Indian conditions in place of the Standard SCN-CN equation.

Procedure for Estimating Runoff Volume from a Catchment

(i) Land use/cover information of the catchment under study is derived based on interpretation of multi-season satellite images. It is highly advantageous if the GIS database of the catchment is prepared and land use/cover data is linked to it.

(ii) The soil information of the catchment is obtained by using soil maps prepared by National Bureau of Soil Survey and Land use planning (NBSS & LUP) (1966). Soil data relevant to the catchment is identified and appropriate hydrological soil classification is made and the spatial form of this data is stored in GIS database.

(iii) Available rainfall data of various rain gauge stations in and around the catchment is collected, screened for consistency and accuracy and linked to the GIS database. For reasonable estimate of catchment yield it is desirable to have a rainfall record of at least 25 years duration.

(iv) Thiessen polygons are established for each identified rain gauge station.

(v) For each Thiessen cell, appropriate area weighted CN_{II} value is established by adequate consideration of spatial variation of land use and/cover and soil types. Further, for each cell, corresponding CN_I and CN_{III} values are determined by using Eqs. (5.23) and (5.24).

(vi) Using the relevant *SCS-CN* equations sequentially with the rainfall data, the corresponding daily runoff series is derived for each cell. From this the needed weekly/monthly/annual runoff time series is derived. Further, by combining the results of various cells constituting the catchment, the corres-ponding catchment runoff time series is obtained.

(vii) Appropriate summing of the above time series, yields seasonal/annual runoff volume series and from this the desired dependable catchment yield can be estimated.

Current Status of SCS-CN Method The *SCS-CN* method has received considerable applications and research study since its introduction in 1969. Recently, Ponce and Hawkins[10] (1996) have critically examined the method, clarified its capabilities, limitations and uses. There is a growing body of literature on this method and a good bibliography on this subject is available in Ref. 10. The chief advantages of *SCS-CN* method can be summed up as:

- It is a simple, predictable, and stable conceptual method for estimation of direct runoff depth based on storm rainfall depth, supported by empirical data.
- It relies on only one parameter, *CN*. Even though *CN* can have a theoretical range of 0–100, in practice it is more likely to be in the range 40–98.

- It features readily grasped and reasonably well-documented environmental inputs.
- It is a well-established method, having been widely accepted for use in USA and many other countries. The modifications suggested by the Ministry of Agriculture, Govt. of India [7], (1972), make its use effective for Indian conditions.

EXAMPLE 5.5 *In a 350 ha watershed the CN value was assessed as 70 for AMC-III. (a) Estimate the value of direct runoff volume for the following 4 days of rainfall. The AMC on July 1st was of category III. Use standard SCS-CN equations.*

Date	July 1	July 2	July 3	July 4
Rainfall (mm)	50	20	30	18

(b) *What would be the runoff volume if the CN_{III} value were 80?*

SOLUTION:

(a) Given $CN_{III} = 70$ $\quad S = (25400/70) - 254 = 108.6$

$$Q = \frac{(P - 0.2\,S)^2}{P + 0.8\,S} \quad \text{for } P > 0.2S$$

$$= \frac{[P - (0.2 \times 108.86)^2}{P + (0.8 \times 108.86)} = \frac{[P - 21.78]^2}{P + 87.09} \quad \text{for } P > 21.78 \text{ mm}$$

Date	P (mm)	Q (mm)
July 1	50	5.81
July 2	20	0
July 3	30	0.58
July 4	18	0
Total	**118**	**6.39**

Total runoff volume over the catchment $V_r = 350 \times 10^4 \times 6.39/(1000)$
$= \mathbf{22{,}365}\ \text{m}^3$

(b) Given $CN_{III} = 80$ $\quad S = (25400/80) - 254 = 63.5$

$$Q = \frac{(P - 0.2\,S)^2}{P + 0.8\,S} \quad \text{for } P > 0.2\ S$$

$$= \frac{[P - (0.2 \times 63.5)]^2}{P + (0.8 \times 63.5)} = \frac{[P - 12.7]^2}{P + 50.8} \quad \text{for } P > 12.7 \text{ mm}$$

Date	P (mm)	Q (mm)
July 1	50	13.80
July 2	20	0.75
July 3	30	3.70
July 4	18	0.41
Total	**118**	**18.66**

Total runoff volume over the catchment $V_r = 350 \times 10^4 \times 18.66/(1000)$
$= \mathbf{65{,}310}\ \text{m}^3$

Example 5.6 *A small watershed is 250 ha in size has group C soil. The land cover can be classified as 30% open forest and 70% poor quality pasture. Assuming AMC at average condition and the soil to be black soil, estimate the direct runoff volume due to a rainfall of 75 mm in one day.*

Solution: AMC = II. Hence $CN = CN$(II). Soil = Black soil. Referring to Table (5.6-a) for C-group soil

Land use	%	*CN*	Product
Open forest	30	60	1800
Pasture (poor)	70	86	6020
Total	**100**		**7820**

Average $CN = 7820/100 = 78.2$ $S = (25400/78.2) - 254 = 70.81$

The relevant runoff equation for Black soil and *AMC*-II is

$$Q = \frac{(P - 0.1S)^2}{P + 0.9S} = \frac{[75 - (0.1 \times 70.81)]^2}{75 + (0.9 \times 70.81)} = 33.25 \text{ mm}$$

Total runoff volume over the catchment $V_r = 250 \times 10^4 \times 33.25/(1000) =$ **83,125** m^3

Example 5.7 *The land use and soil characteristics of a 5000 ha watershed are as follows:*

Soil: Not a black soil. Hydrologic soil classification: 60% is Group B and 40% is Group C

Land Use:

Hard surface areas = 10%
Waste Land = 5%
Orchard (without understory cover) = 30%
Cultivated (Terraced), poor condition = 55%

Antecedent rain: The total rainfall in past five days was 30 mm. The season is dormant season.

(a) *Compute the runoff volume from a 125 mm rainfall in a day on the watershed*
(b) *What would have been the runoff if the rainfall in the previous 5 days was 10 mm?*
(c) *If the entire area is urbanized with 60% residential area (65% average impervious area), 10% of paved streets and 30% commercial and business area (85% impervious), estimate the runoff volume under AMC-II condition for one day rainfall of 125 mm.*

Solution:

(a) Calculation of weighted *CN*

From Table 5.5 *AMC* = Type III. Using Table (5.6-a) weighted CN_{II} is calculated as below:

Land use	Total	Soil Group B (60%)			Soil Group C (40%)		
	(%)	%	*CN*	Product	%	*CN*	Product
Hard surface	10	6	86	516	4	91	364
Waste land	5	3	80	240	2	85	170
Orchard	30	18	55	990	12	69	828
Cultivated land	55	33	71	2343	22	77	1694
Total				**4089**			**3056**

$$\text{Weighted } CN = \frac{(4089 + 3056)}{100} = 71.45$$

$$\text{By Eq. (5.24) } CN_{III} = \frac{71.45}{0.427 + (0.00573 \times 71.45)} = 85.42$$

Since the soil is not a black soil, Eq. (5.27) is used to compute the surface runoff.

$$Q = \frac{(P - 0.3S)^2}{P + 0.7\,S} \quad \text{for } P > 0.3S \text{ and}$$

$$S = \frac{25400}{CN} - 254 = (25400/85.42) - 254 = 43.35$$

$$Q = \frac{[125 - (0.3 \times 43.35)]^2}{125 + (0.7 \times 43.35)} = 80.74 \text{ mm}$$

Total runoff volume over the catchment $V_r = 5000 \times 10^4 \times 80.74/(1000)$

$= \mathbf{4{,}037{,}000\ m^3 = 4.037\ Mm^3}$

(b) Here AMC = Type I

Hence $$CN_I = \frac{71.45}{2.281 - (0.01281 \times 71.45)} = 52.32$$

$$S = (25400/52.32) - 254 = 231.47$$

$$Q = \frac{[125 - (0.3 \times 231.47)]^2}{125 + (0.7 \times 231.47)} = 10.75 \text{ mm}$$

Total runoff volume over the catchment $V_r = 5000 \times 10^4 \times 10.75/(1000)$

$= \mathbf{537500\ m^3 = 0.5375\ Mm^3}$

(c) From Table 5.5 AMC = Type III. Using Table 5.6-c weighted CN_{II} is calculated as below:

Land use (%)	**Total %**	**Soil Group B (60%)**			**Soil Group C (40%)**		
		%	***CN***	**Product**	**%**	***CN***	**Product**
Residential area (65% imp)	60	36	85	3060	24	90	2160
Commercial area (85% imp)	30	18	92	1656	12	94	1128
Paved roads	10	6	98	588	4	98	392
Total				**5304**			**3680**

$$\text{Weightd } CN_{II} = \frac{(5304 + 3680)}{100} = 89.8$$

$$\text{By Eq. (5.24)} \quad CN_{III} = \frac{89.8}{0.427 + (0.00573 \times 89.8)} = 95.37$$

$$S = \frac{25400}{CN} - 254 = (25400/95.37) - 254 = 12.33$$

Since the soil is not a black soil, Eq. (5.27) is used to compute the surface runoff volume.

$$Q = \frac{(P - 0.3\,S)^2}{P + 0.7\,S} \quad \text{for } P > 0.3S \text{ and}$$

$$Q = \frac{[125-(0.3\times 12.33)]^2}{125+(0.7\times 12.33)} = 110.11 \text{ mm}$$

Total runoff volume over the catchment $V_r = 5000 \times 10^4 \times 110.11/(1000)$
$= 5,505,500\ m^3 = 5.5055\ Mm^3$

CN AND C OF RATIONAL FORMULA *SCS-CN* method estimates runoff volume while the rational formula (Chapter 7, Sec. 7.2) estimates runoff rate based on the runoff coefficient *C*. *CN* and *C* of are not easily related even though they depend on the same set of parameters. For an infinite sponge *C* is 0 and *CN* is 0. Similarly for an impervious surface *C* is 1.0 and *CN* is 100. While the end points in the mapping are easily identifiable the relationship between *CN* and *C* are nonlinear. In a general sense, high *C*s are likely to be found where *CN* values are also high.

5.5 FLOW–DURATION CURVE

It is well known that the streamflow varies over a water year. One of the popular methods of studying this streamflow variability is through flow-duration curves. A flow-duration curve of a stream is a plot of discharge against the per cent of time the flow was equalled or exceeded. This curve is also known as *discharge-frequency curve*.

The streamflow data is arranged in a descending order of discharges, using class intervals if the number of individual values is very large. The data used can be daily, weekly, ten daily or monthly values. If *N* number of data points are used in this listing, the plotting position of any discharge (or class value) *Q* is

$$P_p = \frac{m}{N+1} \times 100\% \tag{5.28}$$

where *m* is the order number of the discharge (or class value), P_p = percentage probability of the flow magnitude being equalled or exceeded. The plot of the discharge *Q* against P_p is the flow duration curve (Fig. 5.8). Arithmetic scale paper, or semi-log or log-log paper is used depending upon the range of data and use of the plot. The flow duration curve represents the cumulative frequency distribution and can be considered to represent the streamflow variation of an average year. The ordinate Q_p at any percentage probability P_p represents the flow magnitude in an average year that can be expected to be equalled or exceeded P_p per cent of time and is termed as P_p % dependable flow. In a perennial river Q_{100} = 100% dependable flow is a finite value. On the other hand in an intermittent or ephemeral river the streamflow is zero for a finite part of the year and as such Q_{100} is equal to zero.

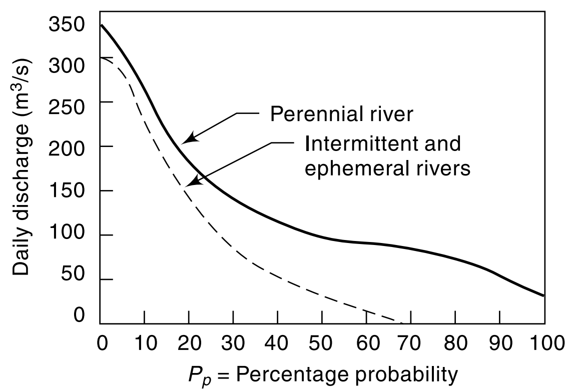

Fig. 5.8 Flow Duration Curve

The following characteristics of the flow duration curve are of interest.

- The slope of a flow duration curve depends upon the interval of data selected. For example, a daily stream flow data gives a steeper curve than a curve based on monthly data for the same stream. This is due to the smoothening of small peaks in the monthly data.
- The presence of a reservoir in a stream considerably modifies the virgin-flow duration curve depending on the nature of flow regulation. Figure 5.9 shows the typical reservoir regulation effect.
- The virgin-flow duration curve when plotted on a log probability paper plots as a straight line at least over the central region. From this property, various coefficients expressing the variability of the flow in a stream can be developed for the description and comparison of different streams.
- The chronological sequence of occurrence of the flow is masked in the flow-duration curve. A discharge of say 1000 m^3/s in a stream will have the same percentage P_p whether it has occurred in January or June. This aspect, a serious handicap, must be kept in mind while interpreting a flow-duration curve.

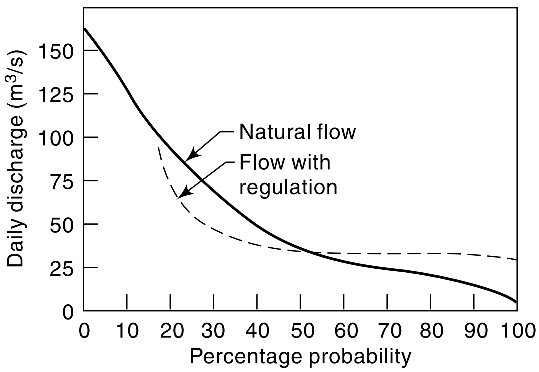

Fig. 5.9 Reservoir Regulation Effect

- The flow-duration curve plotted on a log-log paper (Fig. 5.10) is useful in comparing the flow characteristics of different streams. A steep slope of the curve indicates a stream with a highly variable discharge. On the other hand, a flat slope indicates a slow response of the catchment to the

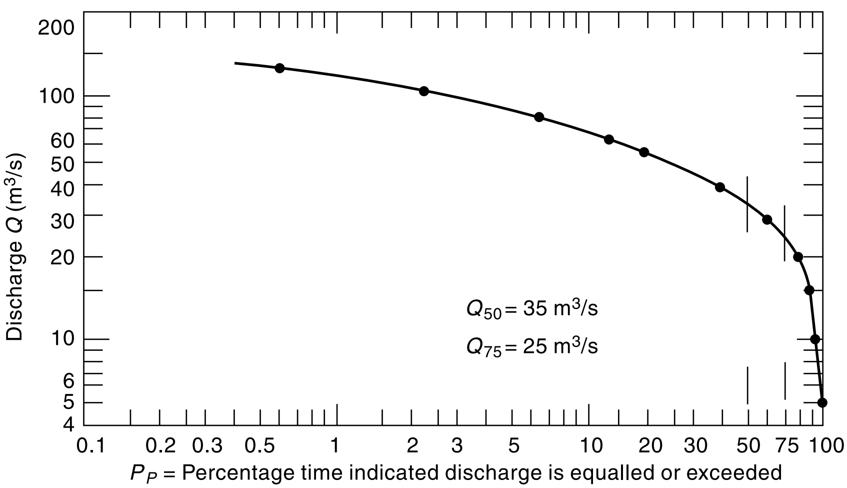

Fig. 5.10 Flow Duration Curve—Example 5.8

rainfall and also indicates small variability. At the lower end of the curve, a flat portion indicates considerable base flow. A flat curve on the upper portion is typical of river basins having large flood plains and also of rivers having large snowfall during a wet season.

Flow-duration curves find considerable use in water resources planning and development activities. Some of the important uses are:

1. In evaluating various dependable flows in the planning of water resources engineering projects
2. Evaluating the characteristics of the hydropower potential of a river
3. Designing of drainage systems
4. In flood-control studies
5. Computing the sediment load and dissolved solids load of a stream
6. Comparing the adjacent catchments with a view to extend the streamflow data.

Example 5.8 *The daily flows of a river for three consecutive years are shown in Table 5.7. For convenience the discharges are shown in class intervals and the number of days the flow belonged to the class is shown. Calculate the 50 and 75% dependable flows for the river.*

Solution: The data are arranged in descending order of class value. In Table 5.7, column 5 shows the total number of days in each class. Column 6 shows the cumulative total of column 5, i.e. the number of days the flow is equal to or greater than the class interval. This gives the value of m. The percentage probability P_p the probability of flow in the class interval being equalled or exceeded is given by Eq. (5.28),

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

Table 5.7 Calculation of Flow Duration Curve from Daily Flow Data—Example 5.8

Daily mean discharge (m^3/s)	No. of days flow in each class interval			Total of columns 2, 3, 4 1961–64	Cumulative Total m	$P_p = \left(\frac{m}{N+1}\right) \times 100\%$
	1961–62	1962–63	1963–64			
1	2	3	4	5	6	7
140–120.1	0	1	5	6	6	0.55
120–100.1	2	7	10	19	25	2.28
100–80.1	12	18	15	45	70	6.38
80–60.1	15	32	15	62	132	12.03
60–50.1	30	29	45	104	236	21.51
50–40.1	70	60	64	194	430	39.19
40–30.1	84	75	76	235	665	60.62
30–25.1	61	50	61	172	837	76.30
25–20.1	43	45	38	126	963	87.78
20–15.1	28	30	25	83	1046	95.35
15–10.1	15	18	12	45	1091	99.45
10–5.1	5	—	—	5	1096	99.91
Total	**365**	**365**	**366**	N = **1096**		

In the present case $N = 1096$. The smallest value of the discharge in each class interval is plotted against P_p on a log-log paper (Fig. 5.10). From this figure Q_{50} = 50% dependable flow = 35 m^3/s and Q_{75} = 75% dependable flow = 26 m^3/s.

5.6 FLOW-MASS CURVE

The flow-mass curve is a plot of the cumulative discharge volume against time plotted in chronological order. The ordinate of the mass curve, V at any time t is thus

$$V = \int_{t_0}^{t} Q\, dt \tag{5.29}$$

where t_0 is the time at the beginning of the curve and Q is the discharge rate. Since the hydrograph is a plot of Q *vs* t, it is easy to see that the flow–mass curve is an integral curve (summation curve) of the hydrograph. The flow–mass curve is also known as *Rippl's mass curve* after Rippl (1882) who suggested its use first. Figure 5.9 shows a typical flow–mass curve. Note that the abscissa is chronological time in months in this figure. It can also be in days, weeks or months depending on the data being analysed. The ordinate is in units of volume in million m^3. Other units employed for ordinate include m^3/s day (cumec day), ha.m and cm over a catchment area.

The slope of the mass curve at any point represents $\dfrac{dV}{dt} = Q$ = rate of flow at that instant. If two points M and N are connected by a straight line, the slope of the line represents the average rate of flow that can be maintained between the times t_m and t_n if a reservoir of adequate storage is available. Thus the slope of the line AB joining the starting point and the last points of a mass curve represents the average discharge over the whole period of plotted record.

Calculation of Storage Volume

Consider a reservoir on the stream whose mass curve is plotted in Fig. 5.11. If it is assumed that the reservoir is full at the beginning of a dry period, i.e. when the inflow rate is less than the withdrawal (demand) rate, the maximum amount of water drawn from the storage is the cumulative difference between supply and demand volumes from the beginning of the dry season. Thus the storage required S is

$$S = \text{maximum of } (\Sigma V_D - \Sigma V_s)$$

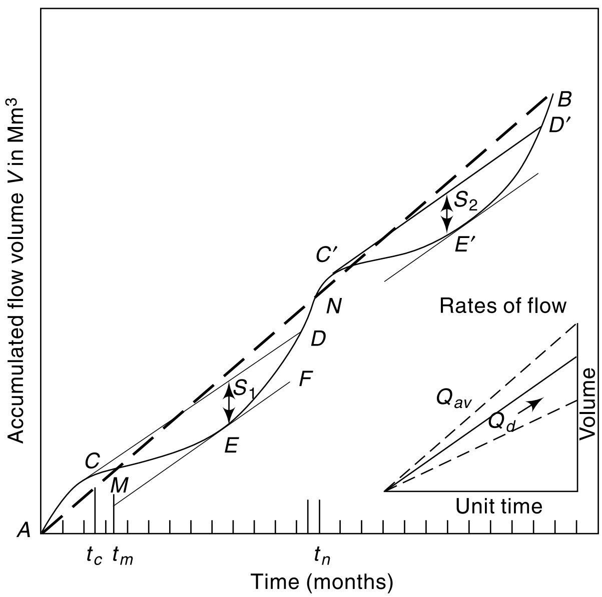

Fig. 5.11 Fow–Mass Curve

where V_D = demand volume, V_S = supply volume. The storage, S which is the maximum cumulative deficiency in any dry season is obtained as the maximum difference in the ordinate between mass curves of supply and demand. The minimum storage volume required by a reservoir is the largest of such S values over different dry periods.

Consider the line CD of slope Q_d drawn tangential to the mass curve at a high point on a ridge. This represents a constant rate of withdrawal Q_d from a reservoir and is called *demand line*. If the reservoir is full at C (at time t_c) then from point C to E the demand is larger than the supply rate as the slope of the flow–mass curve is smaller than the demand line CD. Thus the reservoir will be depleting and the lowest capacity is reached at E. The difference in the ordinates between the demand line CD and a line EF drawn parallel to it and tangential to the mass curve at E (S_1 in Fig. 5.11) represents the volume of water needed as storage to meet the demand from the time the reservoir was full. If the flow data for a large time period is available, the demand lines are drawn tangentially at various other ridges (e.g. $C' D'$ in Fig. 5.11) and the largest of the storages obtained is selected as the minimum storage required by a reservoir. Example 5.9 explains this use of the mass curve. Example 5.10 indicates, storage calculations by arithmetic calculations by adopting the mass-curve principle.

Example 5.9 *The following table gives the mean monthly flows in a river during* 1981. *Calculate the minimum storage required to maintain a demand rate of* 40 m^3/s.

Month	Jan	Fcb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec
Mean Flow (m^3/s)	60	45	35	25	15	22	50	80	105	90	80	70

Solution: From the given data the monthly flow volume and accumulated volumes and calculated as in Table 5.8. The actual number of days in the month are used in calculating of the monthly flow volume. Volumes are calculated in units of cumec. day (= 8.64 × 10^4).

Table 5.8 Calculation of Mass Curve—Example 5.9

Month	Mean flow (m^3/s)	Monthly flow volume (cumec-day)	Accumulated volume (cumec-day)
Jan	60	1860	1860
Feb	45	1260	3120
Mar	35	1085	4205
April	25	750	4955
May	15	465	5420
June	22	660	6080
July	50	1550	7630
Aug	80	2480	10,110
Sep	105	3150	13,260
Oct	90	2790	16,050
Nov	80	2400	18,450
Dec	70	2170	20,620

A mass curve of accumulated flow volume against time is plotted (Fig. 5.12). In this figure all the months are assumed to be of average duration of 30.4 days. A demand line with slope of 40 m^3/s is drawn tangential to the *hump* at the beginning of the curve; line *AB* in Fig. 5.12. A line parallel to this line is drawn tangential to the mass curve at the *valley* portion; line *A′B′*. The vertical distance S_1 between these parallel lines is the minimum storage required to maintain the demand. The value of S_1 is found to be 2100 cumec. Days = 181.4 million m^3.

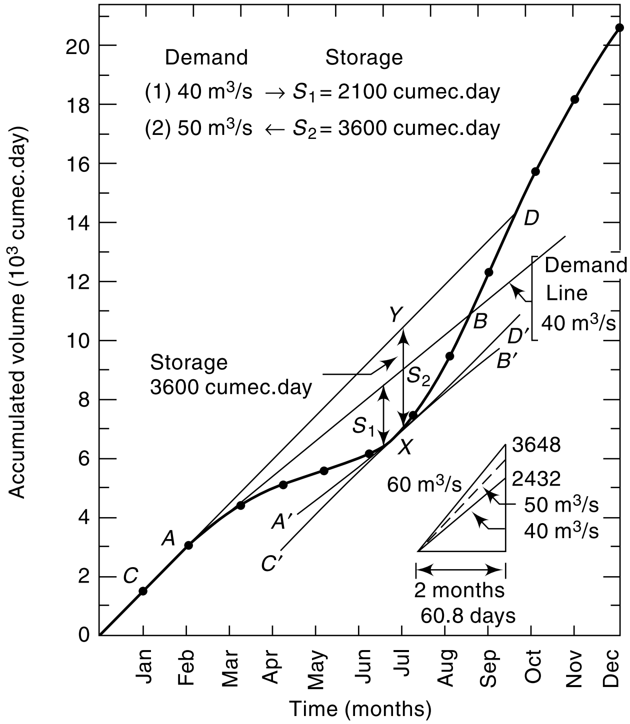

Fig. 5.12 Flow-Mass Curve—Example 5.9

Example 5.10 *Work out the Example 5.9 through arithmetic calculation without the use of mass curve. What is the maximum constant demand that can be sustained by this river?*

Table 5.9 Calculation of Storage—Example 5.9

Month	Mean inflow rate (m^3/s)	Inflow volume (cumec. day)	Demand rate (m^3/s)	Demand volume (cumec. day)	Departure [col. 3- col. 5)	Cum. excess demand volume (cumec. day)	Cum. excess inflow volume (cumec. day)
Jan	60	1860	40	1240	620		620
Feb	45	1260	40	1120	140		760
Mar	35	1085	40	1240	–155	–155	
Apr	25	750	40	1200	–450	–605	
May	15	465	40	1240	–775	–1380	
Jun	22	660	40	1200	–540	–1920	
July	50	1550	40	1240	310		310
Aug	80	2480	40	1240	1240		1550
Sept	105	3150	40	1200	1950		3500
Oct	90	2790	40	1240	1550		5050
Nov	80	2400	40	1200	1200		6250
Dec	70	2170	40	1240	930		7180
	Monthly mean =	**1718.3**					

Solution: The inflow and demand volumes of each month are calculated as in Table 5.9. Column 6 indicating the departure of the inflow volume from the demand. The negative values indicate the excess of demand over the inflow and these have to be met by the storage. Column 7 indicates the cumulative excess demand (i.e., the cumulative excess negative departures). This column indicates the depletion of storage, the first entry of negative value indicates the beginning of *dry period* and the last value the end of the dry period. Col. 8 indicates the filling up of storage and spill over (if any). Each dry period and each filling up stage is to be calculated separately as indicated in Table 5.9.

The maximum value in Col. 7 represents the minimum storage necessary to meet the demand pattern. In the present case, there is only one dry period and the storage requirement is 1920 cumec.day. Note that the difference between this value and the value of 2100 cumec.day obtained by using the mass curve is due to the curvilinear variation of inflow volumes obtained by drawing a smooth mass curve. The arithmetic calculation assumes a linear variation of the mass curve ordinates between two adjacent time units. [***Note:*** It is usual to take data pertaining to a number of N full years. When the analysis of the given data series of length N causes the first entry in Col. 7 to be a negative value and the last entry is also a negative value, then the calculation of the maximum deficit may pose some confusion. In such cases, repeating the data sequence by one more data cycle of N years in continuation with the last entry would overcome this confusion. (See Sec. 5.7, item 2.) There are many other combinations of factors that may cause confusion in interpretation of the results and as such the use of *Sequent Peak Algorithm* described in Sec. 5.7 is recommended as the foolproof method that can be used with confidence in all situations.]

Column 8 indicates the cumulative excess inflow volume from each demand withdrawal from the storage. This indicates the filling up of the reservoir and volume in excess of the provided storage (in the present case 1920 cumec.day) represent spill over. The calculation of this column is necessary to know whether the reservoir fills up after a depletion by meeting a critical demand and if so, when? In the present case the cumulative excess inflow volume will reach +1920 cumec.day in the beginning of September. The reservoir will be full after that time and will be spilling till end of February.

Average of the inflow volume per month = Annual inflow volume/12 = average monthly demand that can be sustained by this river = 1718.33 cumec.day.

Calculation of Maintainable Demand The converse problem of determining the maximum demand rate that can be maintained by a given storage volume can also be solved by using a mass curve. In this case tangents are drawn from the "ridges" of the mass curves across the next "valley" at various slopes. The demand line that requires just the given storage (u_1 v_1 in Fig. 5.13) is the proper demand that can be sustained by the reservoir in that dry period. Similar demand lines are drawn at other "valleys" in the mass curve (e.g. u_2 v_2 and the demand rates determined. The smallest of the various demand rates thus found denotes the maximum firm demand that can be sustained by the given storage. It may be noted that this problem involves a trial-and-error procedure for its solution. Example 5.10 indicates this use of the mass curve.

The following salient points in the use of the mass curve are worth noting:

- The vertical distance between two successive tangents to a mass curve at the ridges (points v_1 and u_2 in Fig. 5.13) represent the water "wasted" over the spillway.
- A demand line must intersect the mass curve if the reservoir is to refill. Nonintersection of the demand line and mass curve indicates insufficient inflow.

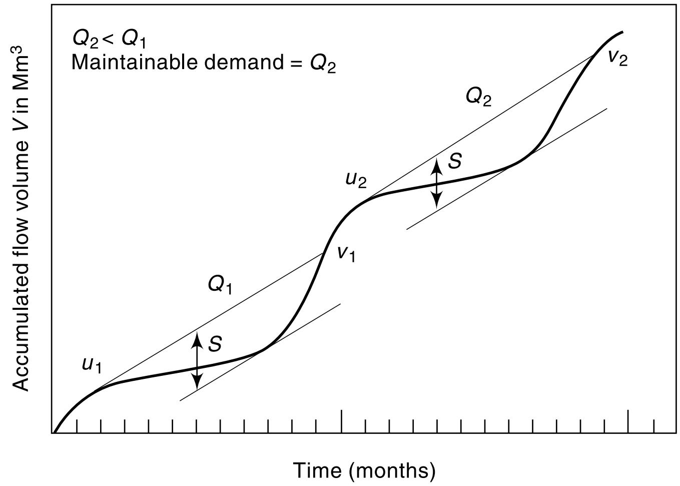

Fig. 5.13 Determination of Maintainable Demand

EXAMPLE 5.11 *Using the mass curve of Example 5.9 obtain the maximum uniform rate that can be maintained by a storage of 3600 m^3/s days.*

SOLUTION: An ordinate *XY* of magnitude 3600 Cumec-days is drawn in Fig. 5.12 at an approximate lowest position in the dip of the mass curve and a line passing through *Y* and tangential to the "hump" of the mass curve at *C* is drawn (line *CYD* in Fig. 5.12). A line parallel to *CD* at the lowest position of the mass curve is now drawn and the vertical interval between the two measured. If the original guess location of *Y* is correct, this vertical distance will be 3600 m^3/s day. If not, a new location for *Y* will have to be chosen and the above procedure repeated.

The slope of the line *CD* corresponding to the final location of *XY* is the required demand rate. In this example this rate is found to be 50 m^3/s.

VARIABLE DEMAND In the examples given above a constant demand rate was assumed to simplify the problem. In practice, it is more likely that the demand rate varies with time to meet various end uses, such as irrigation, power and water-supply needs. In such cases a mass curve of demand, also known as *variable use line* is prepared and superposed on the flow–mass curve with proper matching of time. For example, the demand for the month of February must be against the inflow for the same month. If the reservoir is full at first point of intersection of the two curves, the maximum intercept between the two curves represents the needed storage of the reservoir (Fig. 5.14). Such a plot is sometimes known as *regulation diagram* of a reservoir.

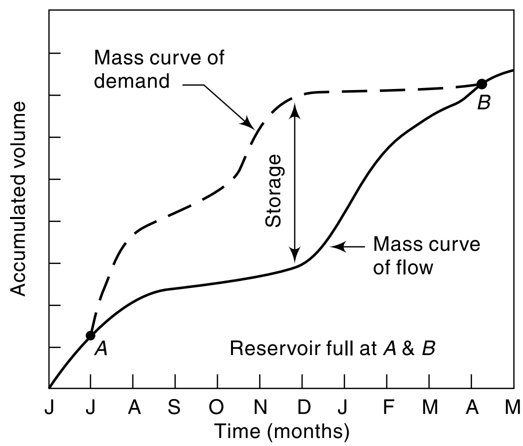

Fig. 5.14 Variable Use Line

In the analysis of problems related to the reservoirs it is necessary to account for evaporation, leakage and other losses from the reservoir. These losses in the volume of water in a known interval of time can either be included in demand rates or deducted from inflow rates. In the latter method, which is generally preferred, the mass curve may have negative slopes at some points. Example 5.12 gives an arithmetic calculation procedure for calculating storage under variable demand.

EXAMPLE 5.12 *For a proposed reservoir the following data were calculated. The prior water rights required the release of natural flow or 5 m^3/s, whichever is less. Assuming an average reservoir area of 20 km^2, estimate the storage required to meet these demands. (Assume the runoff coefficient of the area submerged by the reservoir = 0.5.)*

Month	Mean flow (m^3/s)	Demand (million m^3)	Monthly evaporation (cm)	Monthly rainfall (cm)
Jan	25	22.0	12	2
Feb	20	23.0	13	2
Mar	. 15	24.0	17	1
April	10	26.0	18	1
May	4	26.0	20	1
June	9	26.0	16	13
July	100	16.0	12	24
Aug	108	16.0	12	19
Sept	80	16.0	12	19
Oct	40	16.0	12	1
Nov	30	16.0	11	6
Dec	30	22.0	17	2

SOLUTION: Use actual number of days in a month for calculating the monthly flow and an average month of 30.4 days for prior right release.

$$\text{Prior right release} = 5 \times 30.4 \times 8.64 \times 10^4 = 13.1 \text{ Mm}^3 \text{ when } Q > 5.0 \text{ m}^3/\text{s}.$$

$$\text{Evaporation volume} = \frac{E}{100} \times 20 \times 10^6 = 0.2 \text{ E Mm}^3$$

$$\text{Rainfall volume} = \frac{P}{100} \times (1 - 0.5) \times 20 = 0.1 \text{ P Mm}^3$$

Inflow volume: $I \times$ (No. of days in the month) $\times 8.64 \times 10^4$ m^3

The calculations are shown in Table 5.6 and the required storage capacity is 64.5 Mm^3.

The mass-curve method assumes a definite sequence of events and this is its major drawback. In practice, the runoff is subject to considerable time variations and definite sequential occurrences represent only an idealized situation. The mass-curve analysis is thus adequate for small projects or preliminary studies of large storage projects. The latter ones require sophisticated methods such as *time-series analysis* of data for the final design.

5.7 SEQUENT PEAK ALGORITHM

The mass curve method of estimating the minimum storage capacity to meet a specified demand pattern, described in the previous section has a number of different forms of use in its practical application. However, the following basic assumptions are made in all the adaptations of the mass-curve method of storage analysis.

Table 5.10 Calculation of Reservoir Storage-Example 5.12

Month	Inflow volume (Mm^3)	Withdrawal: Demand (Mm^3)	Withdrawal: Prior rights (Mm^3)	Withdrawal: Evaporation (Mm^3)	Withdrawal: Rainfall (Mm^3)	Total withdrawal (3+4+5+6) (Mm^3)	Departure (Mm^3)	Cum. Excess demand (Mm^3)	Cum. Excess flow volume (Mm^3)
1	2	3	4	5	6	7	8	9	10
Jan	67.0	22.0	13.1	2.4	–0.2	37.3	+29.7	—	29.7
Feb	48.4	23.0	13.1	2.6	–0.2	38.5	+9.9	—	39.6
Mar	40.2	24.0	13.1	3.4	–0.1	40.4	–0.2	–0.2	—
Apr	25.9	26.0	13.1	3.6	–0.1	42.6	–16.7	–16.9	—
May	10.7	26.0	10.7	4.0	–0.1	40.6	–29.9	–46.8	
June	23.3	26.0	13.1	3.2	–1.3	41.0	–17.7	–64.5	
July	267.8	16.0	13.1	2.4	–2.4	29.1	+238.7	—	238.7
Aug	289.3	16.0	13.1	2.4	–1.9	29.6	259.7		498.4
Sept	207.4	16.0	13.1	2.4	–1.9	29.6	177.8		676.2
Oct	107.1	16.0	13.1	2.4	–0.1	31.4	75.7		751.9
Nov	77.8	16.0	12.1	2.2	–0.6	30.7	47.1		799.0
Dec	80.4	22.0	13.1	3.4	–0.2	38.3	42.1		841.1

- If *N* years of data are available, the inflows and demands are assumed to repeat in cyclic progression of *N* year cycles. It is to be noted that in historical data this leads to an implicit assumption that future flows will not contain a more severe drought than what has already been included in the data.
- The reservoir is assumed to be full at the beginning of a dry period. Thus, while using the mass curve method the beginning of the dry period should be noted and the minimum storage required to pass each drought period calculated. Sometimes, for example in Problem 5.7, it may be necessary to repeat the given data series of *N* years sequentially for a minimum of one cycle, i.e. for additional *N* years, to arrive at the desired minimum storage requirement.

The mass curve method is widely used for the analysis of reservoir capacity-demand problems. However, there are many variations of the basic method to facilitate graphical plotting, handling of large data, etc. A variation of the arithmetical calculation described in Examples 5.10 and 5.12 called the *sequent peak algorithm* is particularly suited for the analysis of large data with the help of a computer. This procedure, first given by Thomas (1963), is described in this section.

Let the data be available for *N* consecutive periods not necessarily of uniform length. These periods can be year, month, day or hours depending upon the problem. In the *i*th period let x_i = inflow volume and D_i = demand volume. The surplus or deficit of storage in that period is the *net-flow volume* given by

$$\text{Net-flow volume} = \text{Inflow volume} - \text{Outflow volume}$$
$$= x_i - D_i$$

In the sequent peak algorithm a mass curve of cumulative net-flow volume against chronological time is used. This curve, known as *residual mass curve* (shown typically in Fig. 5.15), will have peaks (local maximums) and troughs (local minimums).

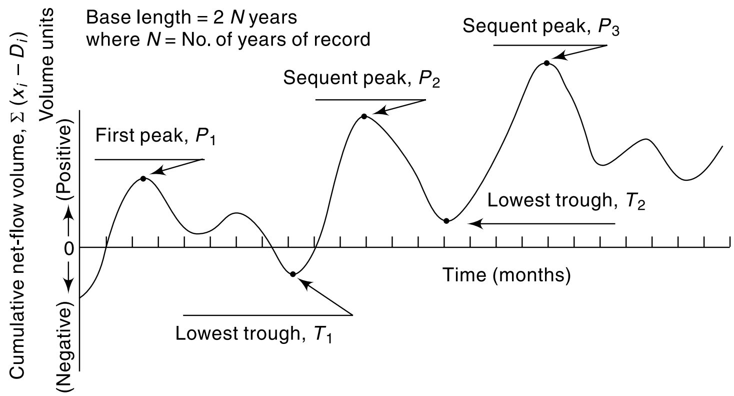

Fig. 5.15 Residual Mass Curve—Definition Sketch for Sequent Peak Algorithm

For any peak *P*, the next following peak of magnitude greater than *P*, is called a *sequent peak*. Using two cycles of *N* periods, where *N* is the number of periods of the data series, the required storage volume is calculated by the following procedure:

1. Calculate the cumulative net-flow volumes, viz.

$$\sum^{t}(x_i - D_i) \qquad \text{for } t = 1, 2, 3 \ldots, 2N$$

2. Locate the first peak P_1 and the sequent peak P_2 which is the next peak of greater magnitude than P_1 (Fig. 5.15).
3. Find the *lowest trough* T_1 between P_1 and P_2 and calculate $(P_1 - T_1)$.
4. Starting with P_2 find the next sequent peak P_3 and the lowest through T_2 and calculate $(P_2 - T_2)$.
5. Repeat the procedure for all the sequent peaks available in the 2*N* periods, i.e. determine the sequent peak P_j, the corresponding T_j and the *j*th storage $(P_j - T_J)$ for all *j* values.
6. The required reservoir storage capacity is

$$S = \text{maximum of } (P_j - T_j) \text{ values}$$

Example 5.13 *The average monthly flows into a reservoir in a period of two consecutive dry years 1981-82 and 1982-83 is given below.*

Month	Mean monthly flow (m^3/s)	Month	Mean monthly flow (m^3/s)
1981— June	20	1982— June	15
July	60	July	50
Aug	200	Aug	150
Sept	300	Sept	200
Oct	200	Oct	80
Nov	150	Nov	50
Dec	100	Dec	110
1982— Jan	80	1983—Jan	100
Feb	60	Feb	60
March	40	March	45
April	30	April	35
May	25	May	30

If a uniform discharge at 90 m^3/s is desired from this reservoir calculate the minimum storage capacity required.

SOLUTION: The data is for 2 years. As such, the sequent peak calculations are performed for 2 × 2 = 4 years. The calculations are shown in Table 5.11.

Scanning the cumulative net-flow volume values (Col. 7) from the start, the first peak P_1 is identified as having a magnitude of 12,200 cumec. day which occurs in the end of the seventh month. The sequent peak P_2 is the peak next to P_1 and of magnitude higher

Table 5.11 Sequent Peak Algorithm Calculations—Example 5.13

S.l. No.	Month	Mean inflow rate (m^3/s)	Inflow volume x_i (cumec. day)	Demand rate (m^3/s)	Demand volume D_i (cumec. day)	Net-flow volume $(x_i - D_i)$ (cumec. day)	Cumulative net-flow volume $\Sigma(x_i - D_i)$ (cumec. day)	Remark
1	June	20	600	90	2700	–2100	–2,100	I Cycle
2	July	60	1860	90	2790	–930	–3,030	
3	Aug.	200	6200	90	2790	+3410	+380	
4	Sept.	300	9000	90	2700	6300	6,680	
5	Oct.	200	6200	90	2790	3410	10,090	
6	Nov.	150	4500	90	2700	1800	11,890	
7	Dec.	100	3100	90	2790	310	12,200*	First peak
8	Jan.	80	2480	90	2790	–310	11,890	P_1
9	Feb.	60	1680	90	2520	–840	11,050	
10	March	40	1240	90	2790	–1550	9,500	
11	April	30	900	90	2700	–1800	7,700	
12	May	25	775	90	2790	–2015	5,685	
13	June	15	450	90	2700	–2250	3,435	
14	July	50	1550	90	2790	–1240	2,195	
15	Aug.	150	4650	90	2790	1860	4,055	
16	Sept.	200	6000	90	2700	3300	7,355	
17	Oct.	80	2480	90	2790	–310	7,045	
18	Nov.	50	1500	90	2700	–1200	5,845	
19	Dec.	110	3410	90	2790	620	6,465	
20	Jan.	100	3100	90	2790	310	6,775	
21	Feb.	60	1680	90	2520	–840	5,935	
22	March	45	1395	90	2790	–1395	4,540	
23	April	35	1050	90	2700	–1650	2,890	
24	May	30	930	90	2790	–1860	1,030	
25	June	20	600	90	2700	–2100	1,070	II Cycle
26	July	60	1860	90	2790	–930	–2,000*	Lowest
27	Aug.	200	6200	90	270	3410	1,410	trough T_1
28	Sept.	300	9000	90	2700	6300	7,710	between P_1
29	Oct.	200	6200	90	2790	3410	11,120	and P_2
30	Nov.	150	4500	90	2700	1800	12,920	
31	Dec.	100	3100	90	2790	310	13,230*	Sequent
32	Jan.	80	2480	90	2790	–310	12,920	Peak P_2
33	Feb.	60	1680	90	2520	–840	12,080	

(Contd.)

Table 5.11 (*Contd.*)

S.l. No.	Month	Mean inflow rate (m^3/s)	Inflow volume x_i (cumec. day)	Demand rate (m^3/s)	Demand volume D_i (cumec. day)	Net-flow volume $(x_i - D_i)$ (cumec. day)	Cumulative net-flow volume $\Sigma(x_i - D_i)$ (cumec. day)	Remark
34	March	40	1240	90	2790	–1550	10,530	
35	April	30	900	90	2700	–1800	8,730	
36	May	25	775	90	2790	–2015	6,715	
37	June	15	450	90	2700	–2250	4,465	
38	July	50	1550	90	2790	–1240	3,225	
39	Aug.	150	4650	90	2790	1860	5,085	
40	Sept.	200	6000	90	2700	3300	8,385	
41	Oct.	80	2480	90	2790	–310	8,075	
42	Nov.	50	1500	90	2700	–1200	6,875	
43	Dec.	110	3410	90	2790	620	7,495	
44	Jan.	100	3100	90	2790	310	7,805	
45	Feb.	60	1680	90	2520	–840	6,965	
46	March	45	1395	90	2790	–1395	5,570	
47	April	38	1050	90	2700	–1650	3,920	
48	May	30	930	90	2790	–1860	2,060	

(***Note:*** Since $N = 2$ years the data is run for 2 cycles of 2 years each.)

than 12,200. This P_2 is identified as having a magnitude of 13,230 cumec. day and occurs in the end of the 31st month from the start. Between P_1 and P_2 the lowest trough T_1 has a magnitude of (–2,000) cumec. day and occurs at the end of the 26th month. In the present data run for two cycles of total duration 4 years, no further sequent peak is observed.

$$P_1 - T_1 = 12{,}000 - (-2000) = 14{,}200 \text{ cumec. day}$$

Since there is no second trough,

The required minimum storage = maximum of $(P_j - T_j)$ values

$$= (P_1 - T_1) = 14{,}200 \text{ cumec. day}$$

5.8 DROUGHTS

In the previous sections of this chapter the variability of the stream flow was considered in the flow duration curve and flow mass curve. However, the extremes of the stream flow as reflected in floods and droughts need special study. They are natural disasters causing large scale human suffering and huge economic loss and considerable effort is devoted by the world over to control or mitigate the ill effects of these two hydrological extremes. The various aspects of floods are discussed in Chapters 7 and 8. The topic of drought, which is not only complex but also region specific is discussed, rather briefly, in this section. The classification of droughts and the general nature of drought studies are indicated with special reference to the Indian conditions. For further details the reader is referred to References 1, 2, 4 and 6.

Definition and Classification

Drought is a climatic anomaly characterized by deficit supply of moisture. This may result from subnormal rainfall over large regions causing below normal natural availability of water over long periods of time. Drought phenomenon is a hydrological extreme like flood and is a natural disaster. However, unlike floods the droughts are of the creeping kind; they develop in a region over a length of time and sometimes may extend to continental scale. The consequences of droughts on the agricultural production, hydropower generation and the regional economy in general is well known. Further, during droughts the quality of available water will be highly degraded resulting in serious environmental and health problems.

Many classifications of droughts are available in literature. The following classification into three categories proposed by the National Commission on Agriculture (1976) is widely adopted in the country:

- *Meteorological drought:*
  It is a situation where there is more than 25% decrease in precipitation from normal over an area.
- *Hydrological drought:*
  Meteorological drought, if prolonged, results in hydrological drought with marked depletion of surface water and groundwater. The consequences are the drying up of tanks, reservoirs, streams and rivers, cessation of springs and fall in the groundwater level.
- *Agricultural drought:*
  This occurs when the soil moisture and rainfall are inadequate during the growing season to support healthy crop growth to maturity. There will be extreme crop stress and wilt conditions.

METEOROLOGICAL DROUGHT The India Meteorological Department (IMD) has adopted the following criteria for sub-classification of meteorological droughts. A meteorological sub-division is considered to be affected by drought if it receives a total seasonal rainfall less than that of 75% of the normal value. Also, the drought is classified as *moderate* if the seasonal deficiency is between 26% and 50%. The drought is said to be *severe* if the deficiency is above 50% of the normal value. Further, a year is considered to be a *drought year* in case the area affected by moderate or severe drought either individually or collectively is more than 20% of the total area of the country.

If the drought occurs in an area with a probability $0.2 \le P \le 0.4$ the area is classified as *drought prone area*, if the probability of occurrence of drought at a place is greater than 0.4, such an area is called as *chronically drought prone area.* Further, in India the meteorological drought is in general related to the onset, breaks and withdrawal times of monsoon in the region. As such, the prediction of the occurrence of drought in a region in the country is closely related to the forecast of deficient monsoon season and its distribution. Accurate forecast of drought, unfortunately, is still not possible.

HYDROLOGICAL DROUGHT From a hydrologist's point of view drought means below average values of stream flow, contents in tanks and reservoirs, groundwater and soil moisture. Such a hydrological drought has four components:

(a) Magnitude (= amount of deficiency)
(b) Duration
(c) Severety (= cumulative amount of deficiency)
(d) Frequency of occurrence

The beginning of a drought is rather difficult to determine as drought is a creeping phenomenon. However, the end of the drought when adequate rainfall saturates the soil mass and restores the stream flow and reservoir contents to normal values is relatively easy to determine.

In the studies on hydrological drought different techniques have to be adopted for study of (i) surface water deficit, and (ii) groundwater deficit. The surface water aspect of drought studies is essentially related to the stream and the following techniques are commonly adopted:

(a) Low-flow duration curve
(b) Low-flow frequency analysis and
(c) Stream flow modelling.

Such studies are particularly important in connection with the design and operation of reservoirs, diversion of stream flow for irrigation, power and drinking water needs; and in all activities related to water quality.

AGRICULTURAL DROUGHT Deficiency of rainfall has been the principal criteria for defining agricultural drought. However, depending on whether the study is at regional level, crop level or plant level there have been a variety of definitions. Considering the various phases of growth of a crop and its corresponding water requirements, the time scale for water deficiency in agricultural drought will have to be much smaller than in hydrological drought studies. Further, these will be not only regional specific but also crop and soil specific.

An *aridity index* (AI) is defined as

$$AI = \frac{PET - AET}{PET} \times 100 \tag{5.30}$$

where PET = *Potential evapotranspiration* and AET = *Actual evapotranspiration*. In this AI calculation, AET is calculated according to *Thornthwite's water balance technique*, taking in to account PET, actual rainfall and field capacity of the soil. It is common to calculate AI on weekly or bi-weekly basis. AI is used as an indicator of possible moisture stress experienced by crops. The departure of AI from its corresponding normal value, known as *AI anomaly*, represents moisture shortage. Based on AI anomaly, the intensity of agricultural drought is classified as follows:

AI anomaly	Severity class
Zero or negative	Non-arid
1 –25	Mild arid
26 –50	Moderate arid
> 50	Severe arid

In addition to AI index, there are other indices such as *Palmer index* (PI) and *Moisture availability index* (MAI) which are used to characterize agricultural drought. IMD produces aridity index (AI) anomaly maps of India on a bi-weekly basis based on data from 210 stations representing different agro-climatic zones in the country. These are useful in planning and management of agricultural operations especially in the drought prone areas. Remote sensing techniques using imageries offer excellent possibilities for monitoring agricultural drought over large areas.

Drought Management

The causes of drought are essentially due to temporal and spatial aberrations in the rainfall, improper management of available water and lack of soil and water conservation. Drought management involves development of both short-term and long-term strategies. *Short-term strategies* include early warning, monitoring and assessment of droughts The *long-term strategies* aim at providing drought mitigating measures through proper soil and water conservation, irrigation scheduling and cropping patterns. Figure 5.16 shows some impacts and possible modifications of various drought components. The following is a list of possible measures for making drought prone areas less vulnerable to drought associated problems:

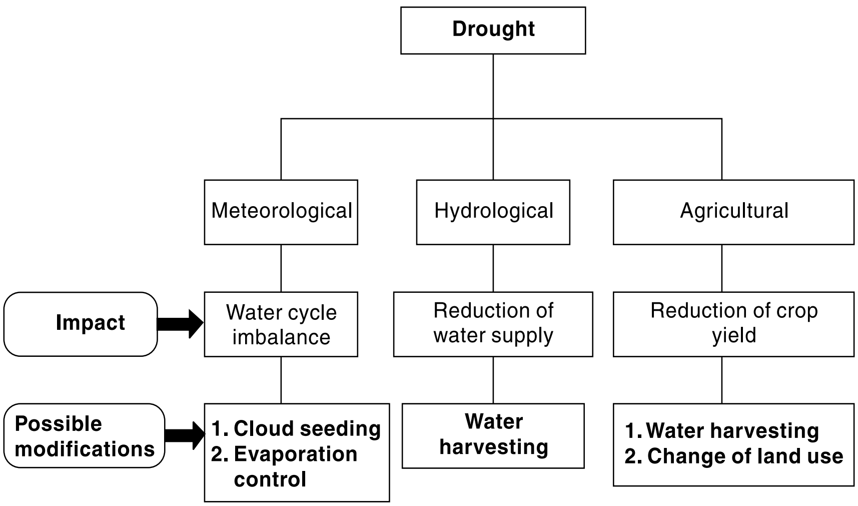

Fig. 5.16 Impact and Possible Modification of Drought Components

- Creation of water storages through appropriate water resources development
- Inter-basin transfer of surface waters from surplus water areas to drought prone areas
- Development and management of ground water potential
- Development of appropriate water harvesting practices
- In situ soil moisture conservation measures
- Economic use of water in irrigation through practices such as drip irrigation, sprinkler irrigation, etc.
- Reduction of evaporation from soil and water surfaces
- Development of afforestation, agro-forestry and agro-horticulture practices
- Development of fuelwood and fodder
- Sand dune stabilization

Drought-proofing of a region calls for integrated approach, taking into account the multi-dimensional interlinkages between various natural resources, environment and local socio-economic factors.

Salient features of water harvesting, which forms an important component in modification of drought components is described in the next sub-section.

Water Harvesting

Water harvesting is a general term to include all systems that concentrate, collect and store runoff from small catchments for later use in smaller user areas. FAO defines water harvesting as, "*Water harvesting* is defined as the process of collecting and concentrating runoff water from a runoff area into a run-on area, where the collected water is either directly applied to the cropping area and stored in the soil profile for immediate use by the crop, i.e. runoff farming, or stored in an on-farm water reservoir for future productive uses, i.e. domestic use, livestock watering, aquaculture and irrigation." The collected water can also be used for groundwater recharge and storage in the aquifer, i.e. recharge enhancement. As a general rule the catchment area from which the water is drawn is larger than the command area, where it is collected and used. The ratio of catchment, to command is inversely related to the amount and intensity of rainfall, the impermeability of soil, and the slope of the land on which it falls.

Water harvesting is essentially a traditional system used since many centuries, now being made over to meet present-day needs. Depending upon the nature of collecting surface and type of storages water harvesting is classified into several categories as mentioned in Fig. 5.17.

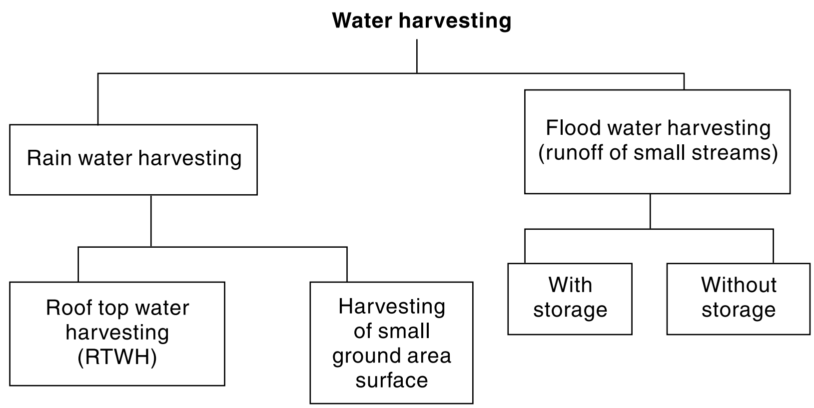

Fig. 5.17 Classification of Water Harvesting Techniques

Roof Top Water Harvesting The productive utilization of rain water falling on roof-tops of structures is known as *Roof Top Water Harvesting* (RTWH). In urban areas the roof tops are usually impervious and occupy considerable land area. Also, generally the municipal water supply is likely to be inadequate, inefficient or unreliable. In such situations, collection of runoff from roof tops of individual structures and storing them for later use has been found to be very attractive and economical proposition in many cases. Inadequacy of water availability and cost of supply has made many industries and large institutions in urban areas situated in arid and semi-arid regions to adopt RTWH systems in a big way. Factors like water quality, methods for efficient and economical collection and storage are some factors that have to be worked out in designing an efficient system to meet specific needs. The cost of adequate size storage is, generally, a constraint in economical RTWH design. In many cases, water collected from roof top is used for recharging the ground water. Characteristics of the rainfall at the place, such as intensity, duration, nature of the rainfall season, average number of rainy days, determine the design of the RTWH design.

Micro Catchment System (Within the Field) of Rainwater Harvesting In this system the catchment is a small area which is not put for any productive purpose. The catchment length is usually between 1 and 30 metres and the overland flow from this during a storm is harvested by collecting and delivering it to a small cultivated plot. The ratio of catchment to the cultivated area is usually 1 : 1 to 3 : 1 and the runoff is stored in soil profile. Normally there will be no provision for overflow. Rainwater harvesting in Micro catchments is sometimes referred to as *Within-Field Catchment System.*

Typical examples of such Rainwater harvesting in micro catchments are:

- Negarim Micro Catchments (for trees)
- Contour Bunds (for trees)
- Contour Ridges (for crops)
- Semi-Circular Bunds (for range and fodder)

Negarim micro catchment technique was originally developed in Israel; the word Negarim is derived from Hebrew word *Neger* meaning runoff. This technique consists of dividing the catchments into a large number of micro catchments in a diamond pattern along the slope. Each micro catchment is of square shape with a small earthen bunds at its boundary and an infiltration pit is provided at the lowest corner as shown in Fig. 5.18. The pit is the cultivated area and usually a tree is grown in the pit. This arrangement of micro catchments of sizes 10 m^2 to 100 m^2, has been found to be very beneficial in arid and semiarid areas where rainfall can be as low as 150 mm.

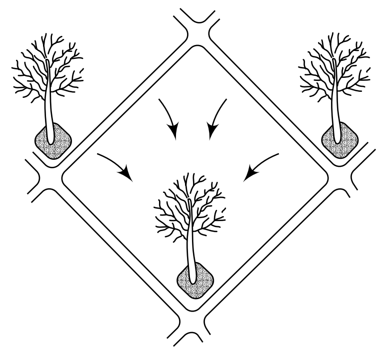

Fig. 5.18 Micro Catchment System: Negarim Micro Catchment for Trees

Macro Catchment System (Within the Field) of Rainwater Harvesting This system is designed for slightly larger catchment areas wherein overland flow and rill flow is collected behind a bund and allowed to be stored in the soil profile through infiltration. The catchment is usually 30 to 200 m long and the ratio of catchment to cultivated area is in the range 2 : 1 to 10 : 1. Typical arrangement consists of one row or two staggered rows of trapezoidal bunds with wing walls. Contour bunds made of piled up stones is also used in this system. It is usual to provide overflow arrangements for disposing of the excess runoff water. Infiltration area behind the bunds is used to grow crops.

Floodwater Farming (Floodwater Harvesting) This system is used for larger catchments and the flow in the drainage is harvested. The catchment areas are several kilometres long and the ratio of catchment to command is larger than 10 : 1. Two sub-systems mentioned below are in common use:

1. Water Harvesting using Storage Structures
2. Water Harvesting through Spreading of Water over Command

Storage Structures Systems Small storage structures are built across the drainage to store a part of the runoff. While the stored surface water would serve as a source of utilisable water to the community for some time the infiltration from this water body would provide valuable recharge to the ground water. The commonly used structures are *Check dams* and *Nalabunds*. These structures have the additional advantage of arresting erosion products from the catchment. Further, these structures prevent the deepening and widening of gullies.

The check dams usually have a masonry overflow spillway and the flanks can be of either masonry construction or of earthen embankment. They are constructed on lower order streams (up to 3) with median slopes. Generally check dams are proposed where water table fluctuations are high and the stream is influent.

Nalabunds are structures constructed across nalas (streams) for impounding runoff flow to cause a small storage. Increased water percolation and improving of soil moisture regime are its main objective. Nalabunds are of small dimension and are constructed by locally available material, usually an earthen embankment. In a Nalabund the spillway is normally a stone lined or rock cut steep channel taking off from one of the ends of the bund at appropriate level. Structures similar to a nalabund but of larger dimension are known as *percolation tanks.* Nalabunds and percolation tanks are constructed in flat reach of a stream with slopes less than 2%.

The irrigation tanks of south India are also sometimes termed as water harvesting structures. Tanks on local streams form a significant source of irrigation in states of Andhra Pradesh, Karnataka, Maharashtra and Tamil Nadu. These are small storage structures formed by earthen bunds to store runoff, of a small stream. The embankment, surplus weir and a sluice outlet form the essential component of a tank. The tank system in a region, which can be a group of independent tanks or a set of tanks in cascade, form an important source of surface water for domestic use, drinking water for life stock, agriculture for growing food and fodder and recharge of subsurface aquifers.

Spreading of Water In this method a diversion across the drainage would cause the runoff to flow on to the adjacent land. Appropriate bunds either of rock or of earth would cause spreading the water over the command. The spread water infiltrates into the soil and is retained as soil moisture and this is used for growing crops. Provision for overflow spillway at the diversion structure, to pass excess water onto the downstream side of the diversion structure, is an important component of the diversion structure.

General: The specific aspects related to the design of water harvesting structures depends upon the rainfall in the region, soil characteristics and terrain slope. It is usual to take up water harvesting activity at a place as a part of intergraded watershed management programme. Norms for estimating recharge from water harvesting structures are given in Sec. 9.13 of Chapter 9.

The water harvesting methods described above are particularly useful in dry land agriculture and form important draught management tool. Community participation in construction and management of water harvesting structure system is essential for economical and sustainable use of the system. Rehabilitation of old irrigation tanks through de-silting to bring it back to its original capacity is now recognized as a feasible and desirable activity in drought proofing of a region.

Droughts in India

Even though India receives a normal annual precipitation of 117 cm, the spatial and temporal variations lead to anomalies that lead to floods and droughts. Consequently droughts have been an everpresent feature of the country. While drought has remained localized in some part of the country in most of the years they have become wide spread and severe in some years. In the past four decades, wide spread and severe droughts have occurred in the years 1965–66, 1971–73, 1979–80, 1982–83, 1984–87, 1994–96, 1999–2000, 2001–02. These droughts affected the agricultural production and the economy significantly and caused immense hardship and misery to a very large population.

Since 1875 till 2004, India faced 29 drought years; the 1918 being the worst year in which about 70% of the country was affected by drought. Analysis of records since 1801 reveals that nearly equal number droughts occurred in 19th century and in 20th century and that there is a lower number of occurrences in the second quarter of both centuries.

It has been estimated that nearly one third of the area of the country (about 1 M ha) is drought prone. Most of the drought prone areas lie in the states of Rajasthan, Karnataka, Andhra Pradesh, Maharashtra, Gujarat and Orissa. Roughly 50% of the drought prone area of the country lies in Deccan plateau. Further, while Rajasthan has a return period of about 2 years for severe droughts it is about 3 years in the Deccan plateau region. It is difficult to estimate the economic losses of drought, as it is a creeping phenomenon with wide spatial coverage. However, a wide spread drought in the country would cover agricultural areas of the order of 100 lakh ha and the consequential loss due to damaged crops could be of the order of Rs 5000 crores.

5.9 SURFACE WATER RESOURCES OF INDIA

Surface Water Resources

Natural (Virgin) Flow in a river basin is reckoned as surface resource of a basin. In view of prior water resources development activities, such as construction of storage reservoirs in a basin, assessment of natural flow is a very complex activity. In most of the basins of the country, water resources have already been developed and utilized to various extents through construction of diversion structures and storage reservoirs for purposes of irrigation, drinking water supply and industrial uses. These utilizations in turn produce *return flows* of varying extent; return flow being defined as the non-consumptive part of any diversion returned back. Return flows to the stream from irrigation use in the basin are usually assumed to be 10% of the water diverted from the reservoir or diversion structure on the stream for irrigation. The return flows from diversions for domestic and industrial use is usually assumed as 80% of the use. The return flow to the stream from ground water use is commonly ignored.

The natural flow in a given period at a site is obtained through water budgeting of observed flow, upstream utilization and increase in storage, evaporation and other consumptive uses and return flows. The surface and groundwater components are generally treated separately.

Estimation of surface water resources of the country has been attempted at various times. Significant recent attempts are:

- A.N. Khosla's estimate (1949), based on empirical relationships, of total annual flow of all the river systems of the country as 1673 km^3.

- CWC (1988), on the basis of statistical analysis of available data, and on rainfall–runoff relationships where flow data was meagre or not available, estimated the total annual runoff of the river systems of India as 1881 km^3.
- The National Commission for Integrated Water Resources Development (1999) used the then available estimates and data and assessed the total surface water resources of the country as 1952.87 km^3 (say 1953 km^3).

It should be noted that the average annual natural (Virgin) flow at the terminal point of a river is generally taken as the surface water resources of the basin. But this resource is available with a probability of about 50% whereas it is customary to design *irrigation projects* with 75% dependability and *domestic water supply projects* for nearly 100% dependability. Obviously, the magnitude of water at higher values of dependability (say 75% and above) will be smaller than the average value.

The total catchment area of all the rivers in India is approximately 3.05 million km^2. This can be considered to be made up of three classes of catchments:

1. Large catchments with basin area larger than 20,000 km^2;
2. Medium catchments with area between 20,000 to 2000 km^2; and
3. Small catchments with area less than 2000 km^2.

Rao[9] has estimated that large catchments occupy nearly 85% of the country's total drainage area and produce nearly 85% of the runoff. The medium and minor catchments account for 7% and 8% of annual runoff volumes respectively. In the major river basin of the country two mighty rivers the Brahmaputra and the Ganga together constitute 71.5% of the total yield in their class and contribute 61% of the country's river flow. Further, these two rivers rank eighth and tenth respectively in the list of the world's ten largest rivers (Table 5.12). It is interesting to note that the ten rivers listed in Table 5.12 account for nearly 50% of the world's annual runoff.

Table 5.12 World's Ten Largest Rivers

Sl. No	River	Annual runoff (Billion m^3)
1.	Amazon	6307
2.	Platt	1358
3.	Congo	1245
4.	Orinoco	1000
5.	Yangtze	927
6.	Mississippi	593
7.	Yenisei	550
8.	Brahmputra	510
9.	Mekong	500
10.	Ganga	493

According to an analysis of CWC, about 80% of average annual flow in the rivers of India is carried during monsoon months. This highlights the need for creating storages for proper utilization of surface water resources of the country. Another interesting aspect of Indian rivers is that almost all the rivers flow through more than one state, highlighting the need for inter-state co-operation in the optimum development of water resources.

Utilizable Water Resources

Utilizable water resources mean the quantum of water withdrawable from its place of natural occurrence. Withdrawal of water from a river depends on topographic conditions and availability of land for the stated project. As a result of various limitations such as to topography, environmental consideration, non-availability of suitable locations and technological shortcomings, it will not be possible to utilize the entire surface water resources of the country. Further, surface water storage structures, such as reservoirs, cause considerable loss by evaporation and percolation. Also, environmental considerations preclude total utilization or diversion of surface water resources of a basin. From these considerations, it is necessary to estimate the optimum utilizable surface runoff of the country for planning purposes. Normally, the optimum utilizable surface runoff of a basin will be around 70% of the total surface runoff potential of the basin.

CWC in 1988 estimated the utilizable surface water resource of the country as 690.32 km^3. The National Commission for Integrated Water Resources Development[8] (1999) has adopted this value in preparing estimates of future water demand–supply scenarios up to the year 2050. Table 5.13 gives the basinwise distribution of utilizable surface water resource of the country.

Table 5.13 Average Flow and Utilizable Surface Water Resource of Various Basins

[Unit: km^3/Year] (Source: Ref. 8]

S. No.	River Basin	Surface water resources	Utilizable surface water resources
1.	Indus	73.31	46
2.	Ganga–Brahmaputra–Meghna Basin		
	2a Ganga sub-basin	525.02	250.0
	2b Brahmaputra sub-basin and	629.05	24.0
	2c Meghna (Barak) sub-basin	48.36	
3.	Subarnarekha	12.37	6.81
4.	Brahmani–Baitarani	28.48	18.30
5.	Mahanadi	66.88	49.99
6.	Godavari	110.54	76.30
7.	Krishna	69.81	58.00
8.	Pennar	6.86	6.32
9.	Cauvery	21.36	19.00
10.	Tapi	14.88	14.50
11.	Narmada	45.64	34.50
12.	Mahi	11.02	3.10
13.	Sabarmati	3.81	1.93
14.	West flowing rivers of Kutchch and Saurashtra	15.10	14.96
15.	West flowing rivers south of Tapi	200.94	36.21
16.	East flowing rivers between Mahanadi and Godavari	17.08	
17.	East flowing rivers between Godavari and Krishna	1.81	13.11

(*Contd.*)

(*Contd.*)

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 North of Ladakh not draining into India	0	0
22.	Rivers draining into Bangladesh	8.57	0
23.	Rivers draining into Myanmar	22.43	0
24.	Drainage areas of Andaman, Nicobar and Lakshadweep islands	0	0
	Total	**1952.87**	**690.32**

In the computation of utilizable water resources as 690 km^3 it is assumed that adequate storage facility is available for balancing the monsoon flows into an average year round availability. The minimum storage required to achieve this is estimated as 460 km^3 against the present estimated total available storage capacity of 253 km^3. If more storage capacity could be developed carry-over from years of above normal rainfall to dry years would be possible. For comparison purposes, for about the same annual runoff the USA has storage of 700 km^3.

Utilizable Dynamic Groundwater Resources The total replenish-able groundwater resources of the country (dynamic) has been estimated by CGWB as 431.89 km^3/year and the utilizable dynamic groundwater potential as 396 km^3/year (details in Chapter 9, Section 9.12).

Water Available from Return Flows Water used for a specific activity such as irrigation and domestic water supply includes consumptive and non-consumptive components. The non-consumptive component part of water use is returned back to hydrologic system either as surface flow or as addition to groundwater system or as soil moisture. However, due to economic and technological constraints and due to possibilities of diminished water quality, only a part of the return flow is recoverable for re-use. The utilizable return flow is an important component to be considered in the demand–supply analysis of utilizable water resources.

Total Water Requirement and Available Resources Scenario

Total Water Requirement for Different Uses The estimated total water requirements, estimated by NCIWRD[8], for the two scenarios and for various sectors at three future horizons are shown in Table 5.14. Irrigation would continue to have the highest water requirement (about 68% of total water requirement), followed by domestic water including drinking and bovine needs.

Evaporation In water resources evaluation studies it is common to adopt a percentage of the live capacity of a reservoir as evaporation losses. The NCIWRD has adopted a national average value of 15% of the live storage capacity of major projects and 25% of the live storage capacity of minor projects as evaporation losses in the country. The estimated evaporation losses from reservoirs are 42 km^3, 50 km^3 and 76 km^3 by the years 2010, 2025 and 2050 respectively.

Demand and Available Water Resources The summary of NCIWRD[8] (1999) study relating to national level assessment of demand and available water

Table 5.14 Water Requirement for Different Uses

(Unit: Cubic Kilometer) [Source: Ref. 8]

Sl No.	Uses	Year 2010			Year 2025			Year 2050		
		Low	High	%	Low	High	%	Low	High	%
Surface Water										
1.	Irrigation	330	339	48	325	366	43	375	463	39
2.	Domestic	23	24	3	30	36	5	48	65	6
3.	Industries	26	26	4	47	47	6	57	57	5
4.	Power	14	15	2	25	26	3	50	56	5
5.	Inland Navigation	7	7	1	10	10	1	15	15	1
6.	Environment (Ecology)	5	5	1	10	10	1	20	20	2
7.	Evaporation Losses	42	42	6	50	50	6	76	76	6
	Total	**447**	**458**	**65**	**497**	**545**	**65**	**641**	**752**	**64**
Ground Water										
1.	Irrigation	213	218	31	236	245	29	253	344	29
2.	Domestic	19	19	2	25	26	3	42	46	4
3.	Indusrtries	11	11	1	20	20	2	24	24	2
4.	Power	4	4	1	6	7	1	13	14	1
	Total	**247**	**252**	**35**	**287**	**298**	**35**	**332**	**428**	**36**
	Grand Total	**694**	**710**	**100**	**784**	**843**	**100**	**973**	**1180**	**100**

resources is given in Table 5.15. The utilizable return flow is an important component to be considered in the demand–supply analysis of utilizable water resources. Estimated utilizable return flows of the country in surface and groundwater mode for different time horizons are shown in Table 5.15. It may be noted that the return flow contributes to an extent of nearly 20–25% in reducing the demand.

Table 5.15 Utilizable Water, Requirements and Return Flow

(Quantity in Cubic Kilometre) [Source: Ref. 8]

Sl. No.	Particulars	Year 2010		Year 2025		Year 2050	
		Low Demand	High Demand	Low Demand	High Demand	Low Demand	High Demand
1	**Utilizable Water**						
	Surface Water	690	690	690	690	690	690
	Ground water	396	396	396	396	396	396
	Augmentation from canal Irrigation	90	90	90	90	90	90
	Total	**996**	**996**	**996**	**996**	**996**	**996**
	Total Water						

(*Contd.*)

(*Contd.*)

2	**Requirement**						
	Surface Water	447	458	497	545	641	752
	Ground Water	247	252	287	298	332	428
	Total	**694**	**710**	**784**	**843**	**973**	**1180**
3	**Return Flow**						
	Surface Water	52	52	70	74	91	104
	Ground Water	144	148	127	141	122	155
	Total	**196**	**200**	**197**	**215**	**213**	**259**
4	**Residual Utilizable water**						
	Surface Water	295	284	263	219	140	42
	Ground Water	203	202	146	149	96	33
	Total	**498**	**486**	**409**	**463**	**236**	**75**

While the table is self-explanatory, the following significant aspects may be noted:

(a) The available water resources of the country are adequate to meet the low demand scenario up to year 2050. However, at high demand scenario it barely meets the demand.

(b) Need for utmost efficiency in management of every aspect of water use, conservation of water resources and reducing the water demand to low demand scenario are highlighted.

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Revision Questions

5.1 List the factors affecting the seasonal and annual runoff (Yield) of a catchment. Describe briefly the interactions of factors listed by you.

5.2 With the help of typical hydrographs describe the salient features of (i) Perennial, (ii) intermittent, and (iii) ephemeral steams.

5.3 Explain briefly:
(a) Water year (b) Natural (Virgin) flow

5.4 What is meant by 75% dependable yield of a catchment? Indicate a procedure to estimate the same by using annual runoff volume time series.

5.5 Describe briefly the *SCS-CN* method of estimation yield of a catchment through use of daily rainfall record.

5.6 Indicate a procedure to estimate the annual yield of a catchment by using Strange's tables.

5.7 Explain clearly the procedure for calculating 75% dependable yield of a basin at a flow gauging station. List the essential data series required for this analysis.

5.8 Distinguish between yield and surface water resources potential of a basin having substantial water resources development for meeting irrigation, domestic and industrial needs within the basin.

5.9 What is watershed simulation? Explain briefly the various stages in the simulation study.

5.10 What is a flow-duration curve? What information can be gathered from a study of the flow duration curve of a stream at a site?

5.11 Sketch a typical flow mass curve and explain how it could be used for the determination of
(a) the minimum storage needed to meet a constant demand
(b) the maximum constant maintainable demand from a given storage.

5.12 Describe the use of flow mass curve to estimate the storage requirement of a reservoir to meet a specific demand pattern. What are the limitations of flow mass curve?

5.13 What is a residual mass curve? Explain the sequent peak algorithm for the calculation of minimum storage required to meet a demand.

5.14 What is a hydrological drought? What are its components and their possible effects?

5.15 List the measures that can be adopted to lessen the effects of drought in a region.

5.16 Describe briefly the surface water resources of India.

PROBLEMS

5.1 Long-term observations at a streamflow-measuring station at the outlet of a catchment in a mountainous area gives a mean annual discharge of 65 m^3/s. An isohyetal map for the annual rainfall over the catchment gives the following areas closed by isohyets and the divide of the catchment:

Isohyet (cm)	Area (km^2)	Isohyet (cm)	Area (km^2)
140–135	50	120–115	600
135–130	300	115–110	400
130–125	450	110–105	200
125–120	700		

Calculate (a) the mean annual depth of rainfall over the catchment,
(b) the runoff coefficient.

5.2 A small stream with a catchment area of 70 km^2 was gauged at a location some distance downstream of a reservoir. The data of the mean monthly gauged flow, rainfall and upstream diversion are given. The regenerated flow reaching the stream upstream of the gauging station can be assumed to be constant at a value of 0.20 Mm^3/month. Obtain the rainfall runoff relation for this stream. What virgin flow can be expected for a monthly rainfall value or 15.5 cm?

Month	Monthly rainfall (cm)	Gauged monthly flow (Mm^3)	Upstream utilization (Mm^3)
1.	5.2	1.09	0.60
2.	8.6	2.27	0.70
3.	7.1	1.95	0.70
4.	9.2	2.80	0.70
5.	11.0	3.25	0.70
6.	1.2	0.28	0.30
7.	10.5	2.90	0.70
8.	11.5	2.98	0.70
9.	14.0	3.80	0.70
10.	3.7	0.84	0.30
11.	1.6	0.28	0.30
12.	3.0	0.40	0.30

5.3 The following table shows the observed annual rainfall and the corresponding annual runoff for a small catchment. Develop the rainfall–runoff correlation equation for this catchment and find the correlation coefficient. What annual runoff can be expected from this catchment for an annual rainfall of 100 cm?

Year	1964	1965	1966	1967	1968	1969
Annual Rainfall (cm)	90.5	111.0	38.7	129.5	145.5	99.8
Annual Runoff (cm)	30.1	50.2	5.3	61.5	74.8	39.9
Year	1970	1971	1972	1973	1974	1975
Annual Rainfall (cm)	147.6	50.9	120.2	90.3	65.2	75.9
Annual Runoff (cm)	64.7	6.5	46.1	36.2	24.6	20.0

5.4 Flow measurement of river Netravati at Bantwal (catchment area = 3184 km^2) yielded the following annual flow volumes:

Year	Observed annual flow (Mm^3)	Year	Observed annual flow (Mm^3)
1970–71	15925	1980–81	16585
1971–72	14813	1981–82	14649
1972–73	11726	1982–83	10662
1973–74	11818	1983–84	11555
1974–75	12617	1984–85	10821
1975–76	15704	1985–86	9466
1976–77	8334	1986–87	9732
1977–78	12864		
1978–79	16195		
1979–80	10392		

The withdrawal upstream of the gauging station [for meeting irrigation, drinking water and industrial needs are 91 Mm^3 in 1970–71 and is found to increase linearly at a rate of 2 Mm^3/year. The annual evaporation loses from water bodies on the river can be assumed to be 4 Mm^3. Estimate the 75% dependable yield at Bantwal. If the catchment area at the mouth of the river is 3222 km^2, estimate the average yield for the whole basin.

5.5 The mean monthly rainfall and temperature of a catchment near Bangalore are given below. Estimate the annual runoff volume and the corresponding runoff coefficient by using Khosla's runoff formula.

Month	Jan	Feb	Mar	Apr	May	Jun	July	Aug	Sep	Oct	Nov	Dec
Temp (°C)	24	27	32	33	31	26	24	24	23	21	20	21
Rainfall (mm)	7	9	11	45	107	71	111	137	164	153	61	13

5.6 An irrigation tank has a catchment of 900 ha. Estimate, by using Strange's method, the monthly and total runoff volumes into the tank due to following monthly rainfall values.

Month	July	Aug	Sept	Oct
Monthly Rainfall (mm)	210	180	69	215

5.7 For a 500 ha watershed in South India with predominantly non-black cotton soil, the CN_{II} has been estimated as 68. (a) If the total rainfall in the past five days is 25 cm and the season is dormant season, estimate the runoff volume due to 80 mm of rainfall in a day? (b) What would be the runoff volume if the rainfall in the past five days were 35 mm?

5.8 Estimate the values of CN_I, CN_{II} and CN_{III} for a catchment with the following land use:

Land use	Soil group C (%)	Soil group D (%)	Total % area
Cultivated land (Paddy)	30	45	75
Scrub forest	6	4	10
Waste land	9	6	15

5.9 A 400 ha watershed has predominantly black cotton soil and its CN_{II} value is estimated as 73. Estimate the runoff volume due to two consecutive days of rainfall as follows:

Day	Day 1	Day 2
Rainfall (mm)	65	80

The AMC can be assumed to be Type III.

5.10 Compute the runoff volume due to a rainfall of 15 cm in a day on a 550 ha watershed. The hydrological soil groups are 50% of group C and 50% of group D, randomly distributed in the watershed. The land use is 55% cultivated with good quality bunding and 45% waste land. Assume antecedent moisture condition of Type-III and use standard *SCS-CN* equations.

5.11 A watershed having an area 680 ha has a CN_{III} value of 77. Estimate the runoff volume due to 3 days of rainfall as below:

Day	Day 1	Day 2	Day 3
Rainfall (mm)	30	50	13

Assume the AMC at Day 1 to be of Type III. Use standard *SCS-CN* equations.

5.12 A watershed has the following land use:

(a) 400 ha of row crop with poor hydrologic condition and

(b) 100 ha of good pasture land

The soil is of hydrologic soil group B. Estimate the runoff volume for the watershed under antecedent moisture category III when 2 days of consecutive rainfall of 100 mm and 90 mm occur. Use standard *SCS-CN* equations.

5.13 (a) Compute the runoff from a 2000 ha watershed due to 15 cm rainfall in a day. The watershed has 35% group B soil, 40% group C soil and 25% group D soil. The land use is 80% residential that is 65% impervious and 20% paved roads. Assume AMC II conditions.

(b) If the land were pasture land in poor condition prior to the development, what would have been the runoff volume under the same rainfall? What is the percentage increase in runoff volume due to urbanization?

[***Note:*** Use standard *SCS-CN* equations.]

5.14 Discharges in a river are considered in 10 class intervals. Three consecutive years of data of the discharge in the river are given below. Draw the flow-duration curve for the river and determine the 75% dependable flow.

Discharge range (m^3/s)	< 6	6.0–9.9	10 14.9	15–24.9	25–39	40–99	100–149	–150 249	250–349	>350
No. of occurrences	20	137	183	232	169	137	121	60	30	6

5.15 The average monthly inflow into a reservoir in a dry year is given below:

Month	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**	**Jan**	**Feb**	**Mar**	**Apr**	**May**
Mean monthly flow (m^3/s)	20	60	200	300	200	150	100	80	60	40	30	25

If a uniform discharge at 90 m^3/s is desired from this reservoir what minimum storage capacity is required?

(*Hints:* Assume the next year to have similar flows as the present year.)

5.16 For the data given in Prob. 5.15, plot the flow mass curve and find:

(a) The minimum storage required to sustain a uniform demand of 70 m^3/s;

(b) If the reservoir capacity is 7500 cumec-day, estimate the maximum uniform rate of withdrawal possible from this reservoir.

5.17 The following table gives the monthly inflow and contemplated demand from a proposed reservoir. Estimate the minimum storage that is necessary to meet the demand

Month	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**July**	**Aug**	**Sept**	**Oct**	**Nov**	**Dec**
Monthly inflow (Mm^3)	50	40	30	25	20	30	200	225	150	90	70	60
Monthly demand (Mm^3)	70	75	80	85	130	120	25	25	40	45	50	60

5.18 For the reservoir in Prob. 5.17 the mean monthly evaporation and rainfall are given below.

Month	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**July**	**Aug**	**Sept**	**Oct**	**Nov**	**Dee**
Evaporation (cm)	6	8	13	17	22	22	14	11	13	12	7	5
Rainfall (cm)	1	0	0	0	0	19	43	39	22	6	2	1

If the average reservoir area can be assumed to be 30 km^2, estimate the change in the storage requirement necessitated by this additional data. Assume the runoff coefficient of the area flooded by the reservoir as equal to 0.4.

5.19 Following is the stream flow record of a stream and covers a critical 2 year period. What is the minimum size of the reservoir required on this stream to provide a constant downstream flow of 0.07 cumecs? Use Sequent Peak Algorithm.

Month (1st Year)	Monthly Discharge (ha.m)	Month (2nd Year)	Monthly Discharge (ha.m)
Jan	57.4	Jan	10.2
Feb	65.5	Feb	30.8
March	28.6	March	43.1
April	32.8	April	53.1
May	36.9	May	38.9
June	24.6	June	28.9
July	10.2	July	16.4
Aug	2.1	Aug	12.3
Sept	2.1	Sept	12.3
Oct	2.1	Oct	4.1
Nov	4.1	Nov	8.2
Dec	8.2	Dec	2.1

5.20 Solve Problem 5.18 using Sequent Peak Algorithm method.

5.21 An unregulated stream provides the following volumes through each successive 4-day period over a 40-day duration at a possible reservoir site. What would be the reservoir capacity needed to ensure maintaining the average flow over these 40 days, if the reservoir is full to start with? What is the average flow? What would be the approximate quantity of water wasted in spillage in this case?

Day	0	4	8	12	16	20	24	28	32	36	40
Runoff volume (Mm^3)	0	9.6	5.4	2.3	3.5	2.3	2.2	1.4	6.4	12.4	10.9

5.22 A reservoir is located in a region where the normal annual precipitation is 160 cm and the normal annual US class A pan evaporation is 200 cm. The average area of reservoir water surface is 75 km^2. If under conditions of 35% of the rainfall on the land occupied by the reservoir runoff into the stream, estimate the net annual increase or decrease in the stream flow as result of the reservoir. Assume evaporation pan coefficient = 0.70.

Objective Questions

5.1 A mean annual runoff of 1 m^3/s from a catchment of area 31.54 km^2 represents an effective rainfall of

(a) 100 cm (b) 1.0 cm (e) 100 mm (d) 3.17 cm

5.2 Direct runoff is made up of

(a) Surface runoff, prompt interflow and channel precipitation
(b) Surface runoff, infiltration and evapotranspiration
(c) Overland flow and infiltration
(d) Rainfall and evaporation

5.3 A hydrograph is a plot of
(a) rainfall intensity against time (b) stream discharge against time
(c) cumulative rainfall against time (d) cumulative runoff against time

5.4 The term *base flow* denotes
(a) delayed groundwater flow reaching a stream
(b) delayed groundwater and snowmelt reaching a stream
(c) delayed groundwater and interflow
(d) the annual minimum flow in a stream

5.5 *Virgin flow* is
(a) the flow in the river downstream of a gauging station
(b) the flow in the river upstream of a gauging station
(c) the flow unaffected by works of man
(d) the flow that would exist in the stream if there were no abstractions to the precipitation

5.6 The water year in India starts from the first day of
(a) January (b) April (c) June (d) September

5.7 An ephemeral stream
(a) is one which always carries some flow
(b) does not have any base flow contribution
(c) is one which has limited contribution of groundwater in wet season
(d) is one which carries only snow-melt water.

5.8 An intermittent stream
(a) has water table above the stream bed throughout the year
(b) has only flash flows in response to storms
(c) has flows in the stream during wet season due to contribution of groundwater.
(d) does not have any contribution of ground water at any time

5.9 Khosla's formula for monthly runoff R_m due to a monthly rainfall P_m is $R_m = P_m - L_m$ where L_m is
(a) a constant
(b) monthly loss and depends on the mean monthly catchment temperature
(c) a monthly loss coefficient depending on the antecedent precipitation index
(d) a monthly loss depending on the infiltration characteristics of the catchment

5.10 The flow-duration curve is a plot of
(a) accumulated flow against time
(b) discharge against time in chronological order
(c) the base flow against the percentage of times the flow is exceeded
(d) the stream discharge against the percentage of times the flow is equalled or exceeded.

5.11 In a flow–mass curve study the demand line drawn from a ridge in the curve did not interest the mass curve again. This represents that
(a) the reservoir was not full at the beginning
(b) the storage was not adequate
(c) the demand cannot be met by the inflow as the reservoir will not refill
(d) the reservoir is wasting water by spill.

5.12 If in a flow–mass curve, a demand line drawn tangent to the lowest point in a valley of the curve does not intersect the mass curve at an earlier time period, it represents that
(a) the storage is inadequate
(b) the reservoir will not be full at the start of the dry period
(c) the reservoir is full at the beginning of the dry period
(d) the reservoir is wasting later by spill.

5.13 The flow-mass curve is an integral curve of

(a) the hydrograph (b) the hyetograph

(c) the flow duration curve (d) the S-curve.

5.14 The total rainfall in a catchment of area 1200 km^2 during a 6-h storm is 16 cm while the surface runoff due to the storm is 1.2×10^8 m^3. The ϕ index is

(a) 0.1 cm/h (b) 1.0 cm/h

(c) 0.2 cm/h (d) cannot be estimated with the given data.

5.15 In India, a meteorological subdivision is considered to be affected by moderate drought if it receives a total seasonal rainfall which is

(a) less than 25% of normal value

(b) between 25% and 49% of normal value

(c) between 50% and 74% of normal value

(d) between 75% and 99% of normal value

5.16 An area is classified as a *drought prone area* if the probability P of occurrence of a drought is

(a) $0.4 < P \leq 1.0$ (b) $0.2 \leq P \leq 0.40$

(c) $0.1 \leq P < 0.20$ (d) $0.0 < P < 0.20$

5.17 In the standard SCS-CN method of modelling runoff due to daily rainfall, the maximum daily rainfall that would not produce runoff in a watershed with $CN = 50$ is about

(a) 65 mm (b) 35 mm (c) 50 mm (d) 25 mm

5.18 In the standard *SCS-CN* method, if $CN = 73$ the runoff volume for a one day rainfall of 100 mm is about

(a) 38 mm (b) 2 mm (c) 56 mm (d) 81 mm

Chapter
6

HYDROGRAPHS

6.1 INTRODUCTION

While long-term runoff concerned with the estimation of yield was discussed in the previous chapter, the present chapter examines in detail the short-term runoff phenomenon. The storm hydrograph is the focal point of the present chapter.

Consider a concentrated storm producing a fairly uniform rainfall of duration, *D* over a catchment. After the initial losses and infiltration losses are met, the rainfall excess reaches the stream through overland and channel flows. In the process of translation a certain amount of storage is built up in the overland and channel-flow phases. This storage gradually depletes after the cessation of the rainfall. Thus there is a time lag between the occurrence of rainfall in the basin and the time when that water passes the gauging station at the basin outlet. The runoff measured at the stream-gauging station will give a typical hydrograph as shown in Fig. 6.1. The duration of the rainfall is also marked in this figure to indicate the time lag in the rainfall and runoff. The hydrograph of this kind which results due to an isolated storm is typically single-peaked skew distribution of discharge and is known variously as *storm hydrograph*, *flood hydrograph* or simply *hydrograph.* It 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*.

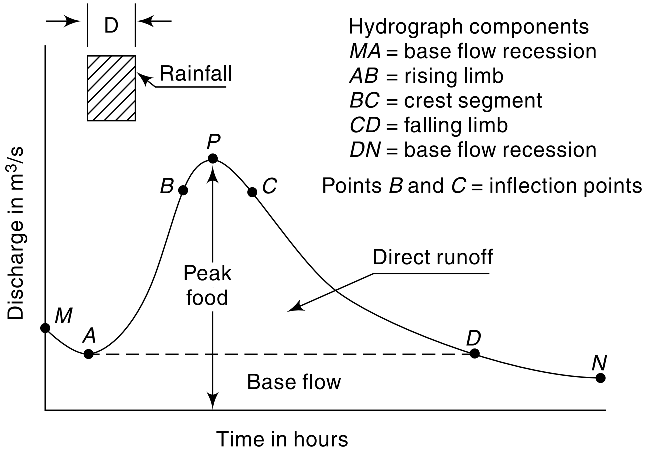

Fig. 6.1 Elements of a Flood Hydrograph