Chapter 7

FLOODS

7.1 INTRODUCTION

A flood is an unusually high stage in a river, normally the level at which the river overflows its banks and inundates the adjoining area. The damages caused by floods in terms of loss of life, property and economic loss due to disruption of economic activity are all too well known. Thousands of crores of rupees are spent every year in flood control and flood forecasting. The hydrograph of extreme floods and stages corresponding to flood peaks provide valuable data for purposes of hydrologic design. Further, of the various characteristics of the flood hydrograph, probably the most important and widely used parameter is the flood peak. At a given location in a stream, flood peaks vary from year to year and their magnitude constitutes a hydrologic series which enable one to assign a frequency to a given flood-peak value. In the design of practically all hydraulic structures the peak flow that can be expected with an assigned frequency (say 1 in 100 years) is of primary importance to adequately proportion the structure to accommodate its effect. The design of bridges, culvert waterways and spillways for dams and estimation of scour at a hydraulic structure are some examples wherein flood-peak values are required.

To estimate the magnitude of a flood peak the following alternative methods are available:

1. Rational method
2. Empirical method
3. Unit-hydrograph technique
4. Flood-frequency studies

The use of a particular method depends upon (i) the desired objective, (ii) the available data, and (iii) the importance of the project. Further the *rational formula* is only applicable to small-size (< 50 km^2) catchments and the unit-hydrograph method is normally restricted to moderate-size catchments with areas less than 5000 km^2.

7.2 RATIONAL METHOD

Consider a rainfall of uniform intensity and very long duration occurring over a basin. The runoff rate gradually increases from zero to a constant value as indicated in Fig. 7.1. The runoff increases as more and more flow from remote areas of the catchment reach the outlet. Designating the time taken for a drop of water from the farthest part of the catchment to reach the outlet as t_c = time of concentration, it is obvious that if the rainfall continues beyond t_c, the runoff will be constant and at the peak value. The peak value of the runoff is given by

$$Q_p = C\,A\,i; \text{ for } t \geq t_c \tag{7.1}$$

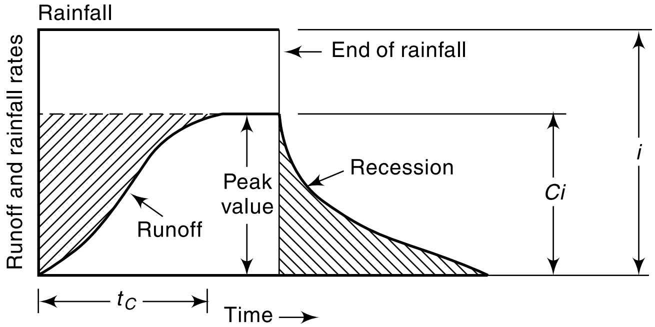

Fig. 7.1 Runoff Hydrograph due to Uniform Rainfall

where C = coefficient of runoff = (runoff/rainfall), A = area of the catchment and i = intensity of rainfall. This is the basic equation of the *rational method.* Using the commonly used units, Eq. (7.1) is written for field application as

$$Q_p = \frac{1}{3.6} C\,(i_{tc,p})\,A \tag{7.2}$$

where Q_p = peak discharge (m^3/s)

C = coefficient of runoff

$(i_{tc,p})$ = the mean intensity of precipitation (mm/h) for a duration equal to t_c and an exceedence probability P

A = drainage area in km^2

The use of this method to compute Q_p requires three parameters: t_c, $(i_{tc,p})$ and C.

Time of Concentration (t_c)

There are a number of empirical equations available for the estimation of the time of concentration. Two of these are described below.

US Practice For small drainage basins, the time of concentration is assumed to be equal to the lag time of the peak flow. Thus

$$t_c = t_p \text{ of Eq. (6.10)} = C_{tL}\left(\frac{LL_{ca}}{\sqrt{S}}\right)^n \tag{7.3}$$

where t_c = time of concentration in hours, C_{tL}, L, L_{ca}, n and S have the same meaning as in Eq. (6.10) of Chapter 6.

Kirpich Equation (1940) This is the popularly used formula relating the time of concentration of the length of travel and slope of the catchment as

$$t_c = 0.01947\, L^{0.77}\, S^{-0.385} \tag{7.4}$$

where t_c = time of concentration (minutes)

L = maximum length of travel of water (m), and

S = slope of the catchment = $\Delta H/L$ in which

ΔH = difference in elevation between the most remote point on the catchment and the outlet.

For easy use Eq. (7.4) is sometimes written as

$$t_c = 0.01947\ K_1^{0.77} \tag{7.4a}$$

where $K_1 = \sqrt{\dfrac{L^3}{\Delta H}}$

Rainfall Intensity ($i_{tc,p}$) The rainfall intensity corresponding to a duration t_c and the desired probability of exceedence P, (i.e. return period $T = 1/P$) is found from the rainfall-frequency-duration relationship for the given catchment area (Chap. 2). This will usually be a relationship of the form of Eq. (2.15), viz.

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

in which the coefficients K, a, x and n are specific to a given area. Table 2.8 (preferably in its expanded form) could be used to estimate these coefficients to a specific catchment. In USA the peak discharges for purposes of urban area drainage are calculated by using P = 0.05 to 0.1. The recommended frequencies for various types of structures used in watershed development projects in India are as below:

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

Runoff Coefficient (C)

The coefficient C represents the integrated effect of the catchment losses and hence depends upon the nature of the surface, surface slope and rainfall intensity. The effect of rainfall intensity is not considered in the available tables of values of C. Some typical values of C are indicated in Table 7.1(a & b).

Equation (7.2) assumes a homogeneous catchment surface. If however, the catchment is non-homogeneous but can be divided into distinct sub areas each having a different runoff coefficient, then the runoff from each sub area is calculated separately and merged in proper time sequence. Sometimes, a non-homogeneous catchment may have component sub areas distributed in such a complex manner that distinct sub zones cannot be separated. In such cases a weighted equivalent runoff coefficient C_e as below is used.

$$C_e = \frac{\sum_1^N C_i A_i}{A} \tag{7.5}$$

Table 7.1(a) Value of the Coefficient *C* in Eq. (7.2)

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

Table 7.1(b) Values of *C* in Rational Formula for Watersheds with Agricultural and Forest Land Covers

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

where A_i = the areal extent of the sub area *i* having a runoff coefficient C_i and N = number of sub areas in the catchment.

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

It should be noted that the word *rational* is rather a misnomer as the method involves the determination of parameters t_c and C in a subjective manner. Detailed description and the practice followed in using the rational method in various countries are given in detail in Ref. 7.

Example 7.1 (a) *An urban catchment has an area of 85 ha. The slope of the catchment is 0.006 and the maximum length of travel of water is 950 m. The maximum depth of rainfall with a 25-year return period is as below:*

Duration (min)	5	10	20	30	40	60
Depth of rainfall (mm)	17	26	40	50	57	62

If a culvert for drainage at the outlet of this area is to be designed for a return period of 25 years, estimate the required peak-flow rate, by assuming the runoff coefficient as 0.3.

Solution: The time of concentration is obtained by the Kirpich formula [Eq.(7.4)] as

$$t_c = 0.01947 \times (950)^{0.77} \times (0.006)^{-0.385} = 27.4 \text{ minutes}$$

By interpolation,
Maximum depth of rainfall for 27.4-min duration

$$= \frac{(50 - 40)}{10} \times 7.4 + 40 = 47.4 \text{ mm}$$

$$\text{Average intensity} = i_{tc,\,p} = \frac{47.4}{27.4} \times 60 = 103.8 \text{ mm/h}$$

By Eq. (7.2),

$$Q_p = \frac{0.30 \times 103.8 \times 0.85}{3.6} = 7.35 \text{ m}^3\text{/s}$$

Example 7.1 (b) *If in the urban area of Example 7.1(a), the land use of the area and the corresponding runoff coefficients are as given below, calculate the equivalent runoff coefficient.*

Land use	Area (ha)	Runoff coefficient
Roads	8	0.70
Lawn	17	0.10
Residential area	50	0.30
Industrial area	10	0.80

Solution: Equivalent runoff coefficient $C_e = \dfrac{\sum_1^N C_i A_i}{A}$

$$C_e = \frac{[(0.7 \times 8) + (0.1 \times 17) + (0.3 \times 50) + (0.8 \times 10)]}{[8 + 17 + 50 + 10]}$$

$$= \frac{30.3}{85} = 0.36$$

Example 7.2 *A 500 ha watershed has the land use/cover and corresponding runoff coefficient as given below:*

Land use/cover	Area (ha)	Runoff coefficient
Forest	250	0.10
Pasture	50	0.11
Cultivated land	200	0.30

The maximum length of travel of water in the watershed is about 3000 m and the elevation difference between the highest and outlet points of the watershed is 25 m. The maximum intensity duration frequency relationship of the watershed is given by

$$i = \frac{6.311T^{0.1523}}{(D+0.50)^{0.945}}$$

water i = intensity in cm/h, T = Return period in years and D = duration of the rainfall in hours. Estimate the (i) 25 year peak runoff from the watershed and (ii) the 25 year peak runoff if the forest cover has decreased to 50 ha and the cultivated land has encroached upon the pasture and forset lands to have a total coverage of 450 ha.

SOLUTION:

Case 1: Equivalent runoff coefficient $C_e = \dfrac{\sum_1^N C_i A_i}{A}$

$$= \frac{[(0.10\times 250)+(0.11\times 50)+(0.30\times 200)]}{500} = 0.181$$

By Eq. (7.4a) time of concentration $t_c = 0.01947\,(K_1)^{0.77}$ with $K_1 = \sqrt{\dfrac{L^3}{\Delta H}}$

Since $L = 3000$ m and $\Delta H = 25$ m $\quad K_1 = \sqrt{\dfrac{(3000)^3}{25}} = 32863$

$$t_c = 0.01947\,(32863)^{0.77} = 58.5 \text{ min} = 0.975 \text{ h}$$

Calculation of $i_{tc,p}$: Here $D = t_c = 0.975$ h. $T = 25$ years. Hence

$$i = \frac{6.311(25)^{0.1523}}{(0.975+0.50)^{0.945}} = 10.304/1.447 = 7.123 \text{ cm/h} = 71.23 \text{ mm/h}$$

Peak Flow by Eq. (7.2), $Q_p = (1/3.6)(C_e\, i\, A)$

$$= \frac{0.181\times 71.23\times (500/100)}{3.6} = 64.46 \text{ m}^3\text{/s}$$

Case 2: Here Equivalent $C = C_e = \dfrac{[(0.10\times 50)+(0.30\times 450)]}{500} = 0.28$

$i = 71.23$ mm/h and $A = 500$ ha $= 5$ (km)^2

$$Q_p = \frac{0.28\times 71.23\times 5}{3.6} = 99.72 \text{ m}^3\text{/s}$$

$i = 71.23$ mm/h and $A = 500$ ha $= 5$ km^2

$$Q_p = \frac{0.28\times 71.23\times 5}{3.6} = 99.72 \text{ m}^3\text{/s}$$

7.3 EMPIRICAL FORMULAE

The empirical formulae used for the estimation of the flood peak are essentially regional formulae based on statistical correlation of the observed peak and important catchment properties. To simplify the form of the equation, only a few of the many parameters affecting the flood peak are used. For example, almost all formulae use the catchment area as a parameter affecting the flood peak and most of them neglect the flood frequency as a parameter. In view of these, the empirical formulae are applicable only in the region from which they were developed and when applied to other areas they can at best give approximate values.

FLOOD PEAK-AREA RELATIONSHIPS

By far the simplest of the empirical relationships are those which relate the flood peak to the drainage area. The maximum flood discharge Q_p from a catchment area A is given by these formulae as

$$Q_p = f(A)$$

While there are a vast number of formulae of this kind proposed for various parts of the world, only a few popular formulae used in various parts of India are given below.

DICKENS FORMULA (1865)

$$Q_p = C_D\, A^{3/4} \tag{7.6}$$

where Q_p = maximum flood discharge (m^3/s) A = catchment area (km^2)
C_D = Dickens constant with value between 6 to 30

The following are some guidelines in selecting the value of C_D:

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

For actual use the local experience will be of aid in the proper selection of C_D. Dickens formula is used in the central and northern parts of the country.

RYVES FORMULA (1884)

$$Q_p = C_R\, A^{2/3} \tag{7.7}$$

where Q_p = maximum flood discharge (m^3/s) A = catchment area (km^2)
and C_R = Ryves coefficient

This formula originally developed for the Tamil Nadu region, is in use in Tamil Nadu and parts of Karnataka and Andhra Pradesh. The values of C_R recommended by Ryves for use are:

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

INGLIS FORMULA (1930) This formula is based on flood data of catchments in Western Ghats in Maharashtra. The flood peak Q_p in m^3/s is expressed as

$$Q_p = \frac{124\,A}{\sqrt{A + 10.4}} \tag{7.8}$$

where A is the catchment area in km^2.

Equation (7.8) with small modifications in the constant in the numerator (124) is in use Maharashtra for designs in small catchments.

OTHER FORMULAE

There are many such empirical formulae developed in various parts of the world. References 3 and 5 list many such formulae suggested for use in various parts of India as well as of the world.

There are some empirical formulae which relate the peak discharge to the basin area and also include the flood frequency. Fuller's formula (1914) derived for catchments in USA is a typical one of this kind and is given by

$$Q_{Tp} = C_f A^{0.8}\,(1 + 0.8 \log T) \tag{7.9}$$

where Q_{Tp} = maximum 24-h flood with a frequency of T years in m^3/s, A = catchment area in km^2, C_f = a constant with values between 0.18 to 1.88.

ENVELOPE CURVES In regions having same climatological characteristics, if the available flood data are meagre, the enveloping curve technique can be used to develop a relationship between the maximum flood flow and drainage area. In this method the available flood peak data from a large number of catchments which do not significantly differ from each other in terms of meteorological and topographical characteristics are collected. The data are then plotted on a log-log paper as flood peak *vs* catchment area. This would result in a plot in which the data would be scattered. If an enveloping curve that would encompass all the plotted data points is drawn, it can be used to obtain maximum peak discharges for any given area. Envelop curves thus obtained are very useful in getting quick rough estimations of peak values. If equations are fitted to these enveloping curves, they provide empirical flood formulae of the type, $Q = f(A)$.

Kanwarsain and Karpov (1967) have presented enveloping curves representing the relationship between the peak-flood flow and catchment area for Indian conditions. Two curves, one for the south Indian rivers and the other for north Indian and central Indian rivers, are developed (Fig. 7.2). These two curves are based on data covering large catchment areas, in the range 10^3 to 10^6 km^2.

Based on the maximum recorded floods throughout the world, Baird and McIllwraith (1951) have correlated the maximum flood discharge Q_{mp} in m^3/s with catchment area A in km^2 as

$$Q_{mp} = \frac{3025\,A}{(278 + A)^{0.78}} \tag{7.10}$$

EXAMPLE 7.3 *Estimate the maximum flood flow for the following catchments by using an appropriate empirical formula:*

1. $A_1 = 40.5$ *km^2 for western Ghat area, Maharashtra*
2. $A_2 = 40.5$ *km^2 in Gangetic plain*
3. $A_3 = 40.5$ *km^2 in the Cauvery delta, Tamil Nadu*
4. *What is the peak discharge for $A = 40.5$ km^2 by maximum world flood experience?*

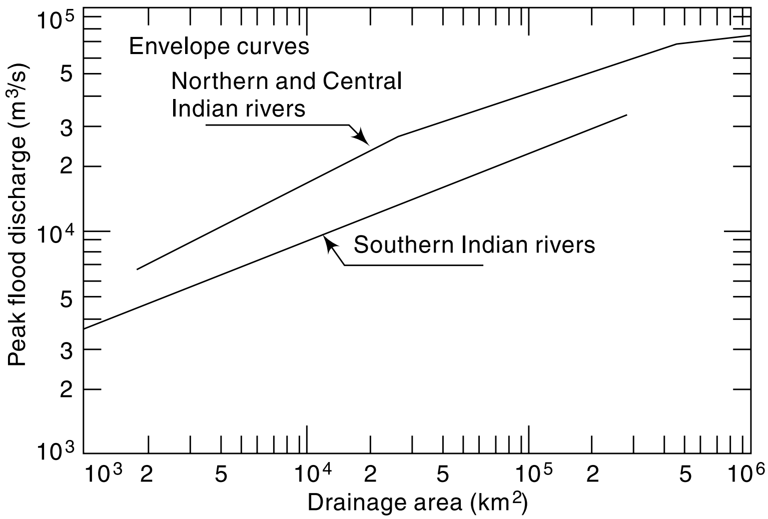

Fig. 7.2 Enveloping Curves for Indian Rivers

SOLUTION:

1. For this catchment, the Inglis formula is recommended.
   By the Inglis formula [Eq. (7.8)],
$$Q_p = \frac{124 \times 40.5}{\sqrt{40.5 + 10.4}} = 704 \text{ m}^3/\text{s}$$
2. In this case Dickens formula [Eq. (7.6)] with $C_D = 6.0$ is recommended. Hence
$$Q_p = 6.0 \times (40.5)^{0.75} = 96.3 \text{ m}^3/\text{s}$$
3. In this case Ryves formula [Eq. (7.7)] with $C_R = 6.8$ is preferred, and this gives
$$Q_p = 6.8\ (40.5)^{2/3} = 80.2 \text{ m}^3/\text{s}$$
4. By Eq. (7.10) for maximum peak discharge based on world experience,
$$Q_{mp} = \frac{3025 \times 40.5}{(278 + 40.5)^{0.78}} = 1367 \text{ m}^3/\text{s}.$$

7.4 UNIT HYDROGRAPH METHOD

The unit hydrograph technique described in the previous chapter can be used to predict the peak-flood hydrograph if the rainfall producing the flood, infiltration characteristics of the catchment and the appropriate unit hydrograph are available. For design purposes, extreme rainfall situations are used to obtain the design storm, viz. the hydrograph of the rainfall excess causing extreme floods. The known or derived unit hydrograph of the catchment is then operated upon by the design storm to generate the desired flood hydrograph. Details about this use of unit hydrograph are given in Sec. 7.12.

7.5 FLOOD FREQUENCY STUDIES

Hydrologic processes such as floods are exceedingly complex natural events. They are resultants of a number of component parameters and are therefore very difficult to model analytically. For example, the floods in a catchment depend upon the characteristics of the catchment, rainfall and antecedent conditions, each one of these factors in turn depend upon a host of constituent parameters. This makes the estimation of the flood peak a very complex problem leading to many different approaches. The empirical formulae and unit hydrograph methods presented in the previous sections are some of them. Another approach to the prediction of flood flows, and also applicable to other hydrologic processes such as rainfall etc. is the statistical method of frequency analysis.

The values of the annual maximum flood from a given catchment area for large number of successive years constitute a hydrologic data series called the *annual series*. The data are then arranged in decreasing order of magnitude and the probability P of each event being equalled to or exceeded (plotting position) is calculated by the plotting-position formula

$$P = \frac{m}{N+1} \tag{7.11}$$

where m = order number of the event and N = total number of events in the data. The recurrence interval, T (also called the *return period* or *frequency*) is calculated as

$$T = 1/P \tag{7.12}$$

The relationship between T and the probability of occurrence of various events is the same as described in Sec. 2.11. Thus, for example, the probability of occurrence of the event r times in n successive years is given by

$$P_{rn} = {}^nC_r P^r q^{n-r} = \frac{n!}{(n-r)!\,r!} P^r q^{n-r}$$

where $\quad q = 1 - P$

Consider, for example, a list of flood magnitudes of a river arranged in descending order as shown in Table 7.2. The length of the record is 50 years.

Table 7.2 Calculation of Frequency T

Order No. m	Flood magnitude Q (m^3/s)	T in years = 51/m
1	160	51.00
2	135	25.50
3	128	17.00
4	116	12.75
:	:	:
:	:	:
:	:	:
49	65	1.04
50	63	1.02

The last column shows the return period T of various flood magnitude, Q. A plot of Q *vs* T yields the probability distribution. For small return periods (i.e. for interpolation) or where limited extrapolation is required, a simple best-fitting curve through plotted points can be used as the probability distribution. A logarithmic scale for T is often advantageous. However, when larger extrapolations of T are involved, theoretical probability distributions have to be used. In frequency analysis of floods the usual problem is to predict extreme flood events. Towards this, specific extreme-value distributions are assumed and the required statistical parameters calculated from the available data. Using these the flood magnitude for a specific return period is estimated.

Chow (1951) has shown that most frequency distribution functions applicable in hydrologic studies can be expressed by the following equation known as the *general equation of hydrologic frequency analysis:*

$$x_T = \bar{x} + K\sigma \tag{7.13}$$

where x_T = value of the variate X of a random hydrologic series with a return period T, $\bar{x}$ = mean of the variate, σ = standard deviation of the variate, K = frequency factor which depends upon the return period, T and the assumed frequency distribution. Some of the commonly used frequency distribution functions for the predication of extreme flood values are

1. Gumbel's extreme-value distribution,
2. Log-Pearson Type III distribution
3. Log normal distribution.

Only the first two distributions are dealt with in this book with emphasis on application. Further details and theoretical basis of these and other methods are available in Refs. 2, 3, 7 and 8.

7.6 GUMBEL'S METHOD

This extreme value distribution was introduced by Gumbel (1941) and is commonly known as Gumbel's distribution. It is one of the most widely used probability distribution functions for extreme values in hydrologic and meteorologic studies for prediction of flood peaks, maximum rainfalls, maximum wind speed, etc.

Gumbel defined a flood as the largest of the 365 daily flows and the annual series of flood flows constitute a series of largest values of flows. According to his theory of extreme events, the probability of occurrence of an event equal to or larger than a value x_0 is

$$P(X \geq x_0) = 1 - e^{-e^{-y}} \tag{7.14}$$

in which y is a dimensionless variable given by

$$y = \alpha(x - a) \qquad a = \bar{x} - 0.45005\,\sigma_x \qquad \alpha = 1.2825/\sigma_x$$

Thus

$$y = \frac{1.285(x - \bar{x})}{\sigma_x} + 0.577 \tag{7.15}$$

where $\bar{x}$ = mean and σ_x = standard deviation of the variate X. In practice it is the value of X for a given P that is required and as such Eq. (7.14) is transposed as

$$y_p = -\ln[-\ln(1 - P)] \tag{7.16}$$

Noting that the return period $T = 1/P$ and designating

y_T = the value of y, commonly called the reduced variate, for a given T

$$y_T = -\left[\ln.\ln\frac{T}{T-1}\right] \tag{7.17}$$

or

$$y_T = -\left[0.834 + 2.303\log\log\frac{T}{T-1}\right] \tag{7.17a}$$

Now rearranging Eq. (7.15), the value of the variate X with a return period T is

$$x_T = \bar{x} + K\,\sigma_x \tag{7.18}$$

where
$$K = \frac{(y_T - 0.577)}{1.2825} \tag{7.19}$$

Note that Eq. (7.18) is of the same form as the general equation of hydrologic-frequency analysis (Eq. (7.13)). Further, Eqs. (7.18) and (7.19) constitute the basic Gumbel's equations and are applicable to an infinite sample size (i.e. $N \to \infty$).

Since practical annual data series of extreme events such as floods, maximum rainfall depths, etc., all have finite lengths of record (Eq. (7.19)) is modified to account for finite N as given below for practical use.

Gumbel's Equation for Practical Use

Equation (7.18) giving the value of the variate X with a recurrence interval T is used as

$$x_T = \bar{x} + K\,\sigma_{n-1} \tag{7.20}$$

where σ_{n-1} = standard deviation of the sample of size $N = \sqrt{\dfrac{\Sigma(x-\bar{x})^2}{N-1}}$

K = frequency factor expressed as

$$K = \frac{y_T - \bar{y}_n}{S_n} \tag{7.21}$$

in which y_T = reduced variate, a function of T and is given by

$$y_T = -\left[\ln.\ln\frac{T}{T-1}\right] \tag{7.22}$$

or
$$y_T = -\left[0.834 + 2.303\log\log\frac{T}{T-1}\right]$$

$\bar{y}_n$ = reduced mean, a function of sample size N and is given in Table 7.3; for $N \to \infty$, $\bar{y}_n \to 0.577$

S_n = reduced standard deviation, a function of sample size N and is given in Table 7.4; for $N \to \infty$, $S_n \to 1.2825$

These equations are used under the following procedure to estimate the flood magnitude corresponding to a given return based on an annual flood series.

1. Assemble the discharge data and note the sample size N. Here the annual flood value is the variate X. Find $\bar{x}$ and σ_{n-1} for the given data.
2. Using Tables 7.3 and 7.4 determine $\bar{y}_n$ and S_n appropriate to given N.
3. Find y_T for a given T by Eq. (7.22).
4. Find K by Eq. (7.21).
5. Determine the required x_T by Eq. (7.20).

The method is illustrated in Example 7.3.

To verify whether the given data follow the assumed Gumbel's distribution, the following procedure may be adopted. The value of x_T for some return periods $T < N$ are calculated by using Gumbel's formula and plotted as x_T *vs* T on a convenient paper such as a semi-log, log-log or Gumbel probability paper. The use of Gumbel probability paper results in a straight line for x_T *vs* T plot. Gumbel's distribution has the property

Table 7.3 Reduced mean $\bar{y}_n$ in Gumbel's Extreme Value Distribution

N = sample size

N	0	1	2	3	4	5	6	7	8	9
10	0.4952	0.4996	0.5035	0.5070	0.5100	0.5128	0.5157	0.5181	0.5202	0.5220
20	0.5236	0.5252	0.5268	0.5283	0.5296	0.5309	0.5320	0.5332	0.5343	0.5353
30	0.5362	0.5371	0.5380	0.5388	0.5396	0.5402	0.5410	0.5418	0.5424	0.5430
40	0.5436	0.5442	0.5448	0.5453	0.5458	0.5463	0.5468	0.5473	0.5477	0.5481
50	0.5485	0.5489	0.5493	0.5497	0.5501	0.5504	0.5508	0.5511	0.5515	0.5518
60	0.5521	0.5524	0.5527	0.5530	0.5533	0.5535	0.5538	0.5540	0.5543	0.5545
70	0.5548	0.5550	0.5552	0.5555	0.5557	0.5559	0.5561	0.5563	0.5565	0.5567
80	0.5569	0.5570	0.5572	0.5574	0.5576	0.5578	0.5580	0.5581	0.5583	0.5585
90	0.5586	0.5587	0.5589	0.5591	0.5592	0.5593	0.5595	0.5596	0.5598	0.5599
100	0.5600									

Table 7.4 Reduced Standard Deviation S_n in Gumbel's Extreme Value Distribution

N = sample size

N	0	1	2	3	4	5	6	7	8	9
10	0.9496	0.9676	0.9833	0.9971	1.0095	1.0206	1.0316	1.0411	1.0493	1.0565
20	1.0628	1.0696	1.0754	1.0811	1.0864	1.0915	1.0961	1.1004	1.1047	1.1086
30	1.1124	1.1159	1.1193	1.1226	1.1255	1.1285	1.1313	1.1339	1.1363	1.1388
40	1.1413	1.1436	1.1458	1.1480	1.1499	1.1519	1.1538	1.1557	1.1574	1.1590
50	1.1607	1.1623	1.1638	1.1658	1.1667	1.1681	1.1696	1.1708	1.1721	1.1734
60	1.1747	1.1759	1.1770	1.1782	1.1793	1.1803	1.1814	1.1824	1.1834	1.1844
70	1.1854	1.1863	1.1873	1.1881	1.1890	1.1898	1.1906	1.1915	1.1923	1.1930
80	1.1938	1.1945	1.1953	1.1959	1.1967	1.1973	1.1980	1.1987	1.1994	1.2001
90	1.2007	1.2013	1.2020	1.2026	1.2032	1.2038	1.2044	1.2049	1.2055	1.2060
100	1.2065									

which gives $T = 2.33$ years for the average of the annual series when N is very large. Thus the value of a flood with $T = 2.33$ years is called the *mean annual flood.* In graphical plots this gives a mandatory point through which the line showing variation of x_T with T must pass. For the given data, values of return periods (plotting positions) for various recorded values, x of the variate are obtained by the relation $T = (N + 1)/m$ and plotted on the graph described above. Figure 7.3 shows a good fit of observed data with the theoretical variation line indicating the applicability of Gumbel's distribution to the given data series. By extrapolation of the straight line x_T *vs* T, values of x_T for $T > N$ can be determined easily (Example 7.3).

Gumbel Probability Paper

The Gumbel probability paper is an aid for convenient graphical representation of Gumbel's distribution. It consists of an abscissa specially marked for various convenient values of the return period T. To construct the T scale on the abscissa, first construct an arithmetic scale of y_T values, say from –2 to +7, as in Fig. 7.3. For selected values of T, say 2, 10, 50, 100, 500 and 1000, find the values of y_T by Eq. (7.22) and mark off those positions on the abscissa. The T-scale is now ready for use as shown in Fig. 7.3.

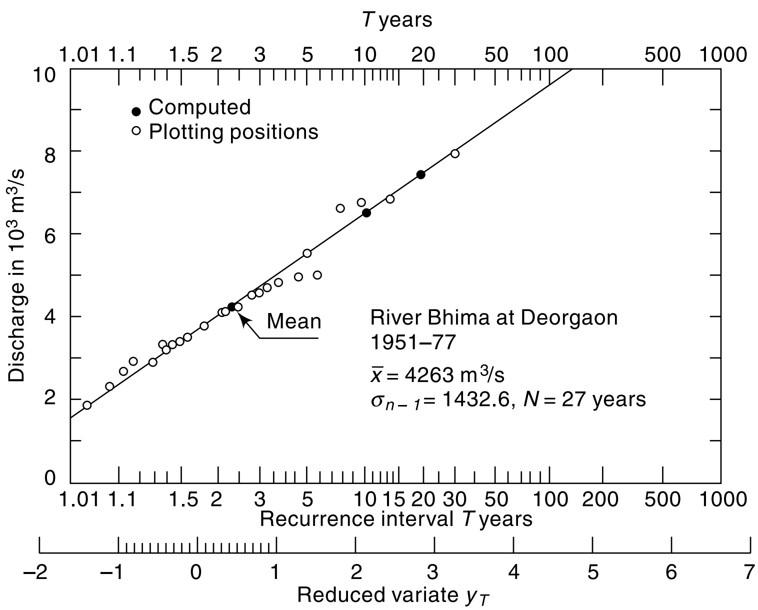

Fig. 7.3 Flood probability analysis by Gumbel's Distribution

The ordinate of a Gumbel paper on which the value of the variate, x_T (flood discharge, maximum rainfall depth, etc.) are plotted may have either an arithmetic scale or logarithmic scale. Since by Eqs (7.18) and (7.19) x_T varies linearly with y_T, a Gumbel distribution will plot as a straight line on a Gumbel probability paper. This property can be used advantageously for graphical extrapolation, wherever necessary.

Example 7.4 *Annual maximum recorded floods in the river Bhima at Deorgaon, a tributary of the river Krishna, for the period 1951 to 1977 is given below. Verify whether the Gumbel extreme-value distribution fit the recorded values. Estimate the flood discharge with recurrence interval of (i) 100 years and (ii) 150 years by graphical extrapolation.*

Year	1951	1952	1953	1954	1955	1956	1957	1958	1959
Max. flood (m^3/s)	2947	3521	2399	4124	3496	2947	5060	4903	3757
Year	1960	1961	1962	1963	1964	1965	1966	1967	1968
Max. flood (m^3/s)	4798	4290	4652	5050	6900	4366	3380	7826	3320
Year	1969	1970	1971	1972	1973	1974	1975	1976	1977
Max. flood (m^3/s)	6599	3700	4175	2988	2709	3873	4593	6761	1971

Solution: The flood discharge values are arranged in descending order and the plotting position recurrence interval T_p for each discharge is obtained as

$$T_p = \frac{N+1}{m} = \frac{28}{m}$$

where m = order number. The discharge magnitude Q are plotted against the corresponding T_p on a Gumbel extreme probability paper (Fig. 7.3).

The statistics $\bar{x}$ and an σ_{n-1} for the series are next calculated and are shown in Table 7.5. Using these the discharge x_T for some chosen recurrence interval is calculated by using Gumbel's formulae [Eqs. (7.22), (7.21) and (7.20)].

Table 7.5 Calculation of T_p for Observed Data—*Example* 7.4

Order number m	Flood discharge x (m^3/s)	T_p (years)	Order number m	Flood discharge x (m^3/s)	T_p (years)
1	7826	28.00	15	3873	1.87
2	6900	14.00	16	3757	1.75
3	6761	9.33	17	3700	1.65
4	6599	7.00	18	3521	1.56
5	5060	5.60	19	3496	1.47
6	5050	4.67	20	3380	1.40
7	4903	4.00	21	3320	1.33
8	4798	3.50	22	2988	1.27
9	4652	3.11	23	2947	—
10	4593	2.80	24	2947	1.17
11	4366	2.55	25	2709	1.12
12	4290	2.33	26	2399	1.08
13	4175	2.15	27	1971	1.04
14	4124	2.00			

$N = 27$ years, $\bar{x} = 4263\ m^3/s$, $\sigma_{n-1} = 1432.6\ m^3/s$

From Tables 7.3 and 7.4, for $N = 27$, $y_n = 0.5332$ and $S_n = 1.1004$.

Choosing $T = 10$ years, by Eq. (7.22),

$$y_T = -[\ln \times \ln (10/9)] = 2.25037$$

$$K = \frac{2.25307 - 0.5332}{1.1004} = 1.56$$

$$\bar{x}_T = 4263 + (1.56 \times 1432.6) = 6499 \text{ m}^3\text{/s}$$

Similarly, values of x_T are calculated for two more T values as shown below.

T years	**x_T [obtained by Eq. (7.20)] (m³/s)**
5.0	5522
10.0	6499
20.0	7436

These values are shown in Fig. 7.3. It is seen that due to the property of the Gumbel's extreme probability paper these points lie on a straight line. A straight line is drawn through these points. It is seen that the observed data fit well with the theoretical Gumbel's extreme-value distribution.

[***Note:*** In view of the linear relationship of the theoretical x_T and T on a Gumbel probability paper it is enough if only two values of T and the corresponding x_T are calculated. However, if Gumbel's probability paper is not available, a semi-log plot with log scale for T will have to be used and a large set of (x_T, T) values are needed to identify the theoretical curve.]

By extrapolation of the theoretical x_T *vs* T relationship, from Fig. 7.3,

At $T = 100$ years, $x_T = 9600 \text{ m}^3\text{/s}$

At $T = 150$ years, $x_T = 10{,}700 \text{ m}^3\text{/s}$

[By using Eqs (7.20) to (7.22), $x_{100} = 9558 \text{ m}^3\text{/s}$ and $x_{150} = 10{,}088 \text{ m}^3\text{/s}$.]

EXAMPLE 7.5 *Flood-frequency computations for the river Chambal at Gandhisagar dam, by using Gumbel's method, yielded the following results:*

Return period T (years)	**Peak flood (m³/s)**
50	40,809
100	46,300

Estimate the flood magnitude in this river with a return period of 500 years.

SOLUTION: By Eq. (7.20),

$$x_{100} = \bar{x} + K_{100}\, \sigma_{n-1} \qquad x_{50} = \bar{x} + K_{50}\, \sigma_{n-1}$$

$$(K_{100} - K_{50})\sigma_{n-1} = x_{100} - x_{50} = 46300 - 40809 = 5491$$

But
$$K_T = \frac{y_T}{S_n} - \frac{\bar{y}_n}{S_n}$$

where S_n and $\bar{y}_n$ are constants for the given data series.

$\therefore$
$$(y_{100} - y_{50})\frac{\sigma_{n-1}}{S_n} = 5491$$

By Eq. (7.22)

$$y_{100} = -[\ln \times \ln(100/99)] = 4.60015$$

$$y_{50} = -[\ln \times \ln(50/99)] = 3.90194$$

$$\frac{\sigma_{n-1}}{S_n} = \frac{5491}{(4.60015 - 3.90194)} = 7864$$

For $T = 500$ years, by Eq. (7.22),

$$y_{500} = -[\ln \times \ln (500/499)] = 6.21361$$

$$(y_{500} - y_{100}) \frac{\sigma_{n-1}}{S_n} = x_{500} - x_{100}$$

$$(6.21361 - 4.60015) \times 7864 = x_{500} - 46300$$

$$x_{500} = 58988, \text{ say } 59,000 \text{ m}^3/\text{s}$$

Example 7.6 *The mean annual flood of a river is 600 m^3/s and the standard deviation of the annual flood time series is 150 m^3/s. What is the probability of a flood of magnitude 1000 m^3/s occurring in the river within next 5 years? Use Gumbel's method and assume the sample size to be very large.*

Solution: $\bar{x} = 600$ m^3/s and $\sigma_{n-1} = 150$ m^3/s $\qquad x_T = \bar{x} + K\sigma_{n-1}$

$$1000 = 600 + K(150)$$

$$K = 2.6667 = \frac{y_T - 0.577}{1.2825}$$

Hence $y_T = 3.9970$

Also, $y_T = 3.9970 = -\left[\ln \cdot \ln \frac{T}{T-1}\right]$

$\therefore \quad \frac{T}{T-1} = 1.01854$

$$T = 54.9 \text{ years, say } 55 \text{ years}$$

Probability of occurrence of a flood of magnitude 1000 m^3/s $= p = 1/55 = 0.0182$

The probability of a flood of magnitude 1000 m^3/s occurring at least once in 5 years =

$$p_1 = 1 - (1 - p)^5 = 1 - (0.9818)^5 = 0.0877 = 11.4\%$$

Confidence Limits

Since the value of the variate for a given return period, x_T determined by Gumbel's method can have errors due to the limited sample data used, an estimate of the confidence limits of the estimate is desirable. The confidence interval indicates the limits about the calculated value between which the true value can be said to lie with a specific probability based on sampling errors only.

For a confidence probability c, the confidence interval of the variate x_T is bounded by values x_1 and x_2 given by[6]

$$x_{1/2} = x_T \pm f(c)\, S_e \tag{7.23}$$

where $f(c)$ = function of the confidence probability c determined by using the table of normal variates as

c in per cent	50	68	80	90	95	99
$f(c)$	0.674	1.00	1.282	1.645	1.96	2.58

$$S_e = \text{probable error} = b \frac{\sigma_{n-1}}{\sqrt{N}} \tag{7.23a}$$

$$b = \sqrt{1 + 1.3K + 1.1K^2}$$

K = frequency factor given by Eq. (7.21)

σ_{n-1} = standard deviation of the sample

N = sample size.

It is seen that for a given sample and T, 80% confidence limits are twice as large as the 50% limits and 95% limits are thrice as large as 50% limits.

Example 7.7 *Data covering a period of 92 years for the river Ganga at Raiwala yielded the mean and standard derivation of the annual flood series as 6437 and 2951 m^3/s respectively. Using Gumbel's method estimate the flood discharge with a return period of 500 years. What are the (a) 95% and (b) 80% confidence limits for this estimate.*

Solution: From Table 7.3 for $N = 92$ years, $\bar{y}_n = 0.5589$ and $S_n = 1.2020$ from Table 7.4.

$$Y_{500} = -[\ln \times \ln (500/499)] = 6.21361$$

$$K_{500} = \frac{6.21361 - 0.5589}{1.2020} = 4.7044$$

$$x_{500} = 6437 + 4.7044 \times 2951 = 20320 \text{ m}^3/\text{s}$$

From Eq. (7.33a)

$$b = \sqrt{1 + 1.3(4.7044) + 1.1(4.7044)^2} = 5.61$$

$$S_e = \text{probable error} = 5.61 \times \frac{2951}{\sqrt{92}} = 1726$$

(a) For 95% confidence probability $f(c)$ = 1.96 and by Eq. (7.23)

$x_{1/2} = 20320 \pm (1.96 \times 1726)$ $\quad x_1 = 23703$ m^3/s and $x_2 = 16937$ m^3/s

Thus estimated discharge of 20320 m^3/s has a 95% probability of lying between 23700 and 16940 m^3/s

(b) For 80% confidence probability, $f(c)$ = 1.282 and by Eq. (7.23)

$x_{1/2} = 20320 \pm (1.282 \times 1726)$ $\quad x_1 = 22533$ m^3/s and $x_2 = 18107$ m^3/s

The estimated discharge of 20320 m^3/s has a 80% probability of lying between 22530 and 18110 m^3/s.

For the data of Example 7.7, the values of x_T for different values of T are calculated and shown plotted on a Gumbel probability paper in Fig. 7.4. This variation is marked as "fitted line" in the figure. Also shown in this plot are the 95 and 80% confidence limits for various values of T. It is seen that as the confidence probability increases, the confidence interval also increases. Further, an increase in the return period T causes the confidence band to spread. Theoretical work by Alexeev (1961) has shown that for Gumbel's distribution the coefficient of skew $C_s \to 1.14$ for very low values of N. Thus the Gumbel's distribution will give erroneous results if the sample has a value of C_s very much different from 1.14.

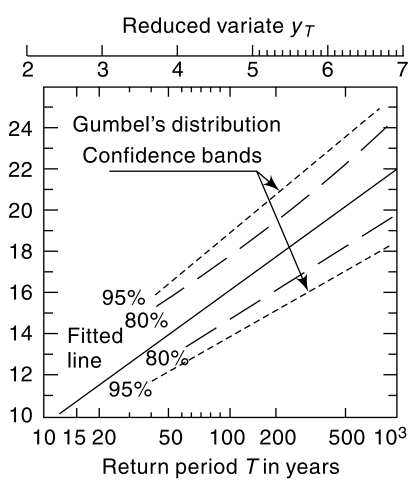

Fig. 7.4 Confidence Bands for Gumbels Distribution—Example 7.7

7.7 LOG-PEARSON TYPE III DISTRIBUTION

This distribution is extensively used in USA for projects sponsored by the US Government. In this the variate is first transformed into logarithmic form (base 10) and the transformed data is then analysed. If X is the variate of a random hydrologic series, then the series of Z variates where

$$z = \log x \tag{7.24}$$

are first obtained. For this Z series, for any recurrence interval T, Eq. (7.13) gives

$$z_T = \bar{z} + K_z\,\sigma_z \tag{7.25}$$

where K_z = a frequency factor which is a function of recurrence interval T and the coefficient of skew C_s,

σ_z = standard deviation of the Z variate sample

$$= \sqrt{\Sigma(z-\bar{z})^2/(N-1)} \tag{7.25a}$$

and C_s = coefficient of skew of variate Z

$$= \frac{N\,\Sigma(z-\bar{z})^3}{(N-1)(N-2)(\sigma_z)^3} \tag{7.25b}$$

$\bar{z}$ = mean of the z values

N = sample size = number of years of record

The variations of $K_z = f(C_s, T)$ is given in Table 7.6.

After finding z_T by Eq. (7.25), the corresponding value of x_T is obtained by Eq. (7.24) as

$$x_T = \text{antilog}\,(z_T) \tag{7.26}$$

Sometimes, the coefficient of skew C_s, is adjusted to account for the size of the sample by using the following relation proposed by Hazen (1930).

$$\hat{C}_s = C_s\left(\frac{1+8.5}{N}\right) \tag{7.27}$$

where $\hat{C}_s$ = adjusted coefficient of skew. However, the standard procedure for use of log-Pearson Type III distribution adopted by U.S. Water Resources Council does not include this adjustment for skew.

When the skew is zero, i.e. $C_s = 0$, the log-Pearson Type III distribution reduces to *log normal distribution.* The log-normal distribution plots as a straight line on logarithmic probability paper.

Table 7.6 $K_z = F(C_s, T)$ for Use in Log-Pearson Type III Distribution

Coefficient of skew, C_s	Recurrence interval T in years						
	2	10	25	50	100	200	1000
3.0	–0.396	1.180	2.278	3.152	4.051	4.970	7.250
2.5	–0.360	1.250	2.262	3.048	3.845	4.652	6.600
2.2	–0.330	1.284	2.240	2.970	3.705	4.444	6.200
2.0	–0.307	1.302	2.219	2.912	3.605	4.298	5.910
1.8	–0.282	1.318	2.193	2.848	3.499	4.147	5.660
1.6	–0.254	1.329	2.163	2.780	3.388	3.990	5.390
1.4	–0.225	1.337	2.128	2.706	3.271	3.828	5.110

(*Contd.*)

(*Contd.*)

1.2	–0.195	1.340	2.087	2.626	3.149	3.661	4.820
1.0	–0.164	1.340	2.043	2.542	3.022	3.489	4.540
0.9	–0.148	1.339	2.018	2.498	2.957	3.401	4.395
0.8	–0.132	1.336	1.998	2.453	2.891	3.312	4.250
0.7	–0.116	1.333	1.967	2.407	2.824	3.223	4.105
0.6	–0.099	1.328	1.939	2.359	2.755	3.132	3.960
0.5	–0.083	1.323	1.910	2.311	2.686	3.041	3.815
0.4	–0.066	1.317	1.880	2.261	2.615	2.949	3.670
0.3	–0.050	1.309	1.849	2.211	2.544	2.856	3.525
0.2	–0.033	1.301	1.818	2.159	2.472	2.763	3.380
0.1	–0.017	1.292	1.785	2.107	2.400	2.670	3.235
0.0	**0.000**	**1.282**	**1.751**	**2.054**	**2.326**	**2.576**	**3.090**
–0.1	0.017	1.270	1.716	2.000	2.252	2.482	2.950
–0.2	0.033	1.258	1.680	1.945	2.178	2.388	2.810
–0.3	0.050	1.245	1.643	1.890	2.104	2.294	2.675
–0.4	0.066	1.231	1.606	1.834	2.029	2.201	2.540
–0.5	0.083	1.216	1.567	1.777	1.955	2.108	2.400
–0.6	0.099	1.200	1.528	1.720	1.880	2.016	2.275
–0.7	0.116	1.183	1.488	1.663	1.806	1.926	2.150
–0.8	0.132	1.166	1.448	1.606	1.733	1.837	2.035
–0.9	0.148	1.147	1.407	1.549	1.660	1.749	1.910
–1.0	0.164	1.128	1.366	1.492	1.588	1.664	1.880
–1.4	0.225	1.041	1.198	1.270	1.318	1.351	1.465
–1.8	0.282	0.945	1.035	1.069	1.087	1.097	1.130
–2.2	0.330	0.844	0.888	0.900	0.905	0.907	0.910
–3.0	0.396	0.660	0.666	0.666	0.667	0.667	0.668

[*Note:* C_s = 0 corresponds to log-normal distribution]

EXAMPLE 7.8 *For the annual flood series data given in Example 7.4, estimate the flood discharge for a return period of (a) 100 years (b) 200 years and (c) 1000 years by using log-Pearson Type III distribution.*

SOLUTION: The variate $z = \log x$ is first calculated for all the discharges (Table 7.7). Then the statistics $\bar{Z}$, σ_z and C_s are calculated from Table 7.7 to obtain

Table 7.7 Variate Z—*Example* 7.8

Year	Flood x (m^3/s)	$z = \log x$	Year	Flood x (m^3/s)	$z = \log x$
1951	2947	3.4694	1965	4366	3.6401
1952	3521	3.5467	1966	3380	3.5289
1953	2399	3.3800	1967	7826	3.8935
1954	4124	3.6153	1968	3320	3.5211
1955	3496	3.5436	1969	6599	3.8195
1956	2947	3.4694	1970	3700	3.5682

(*Contd.*)

(*Contd.*)

1957	5060	3.7042	1971	4175	3.6207
1958	4903	3.6905	1972	2988	3.4754
1959	3751	3.5748	1973	2709	3.4328
1960	4798	3.6811	1974	3873	3.5880
1961	4290	3.6325	1975	4593	3.6621
1962	4652	3.6676	1976	6761	3.8300
1963	5050	3.7033	1977	1971	3.2947
1964	6900	3.8388			

$\sigma_z = 0.1427 \qquad C_s = \dfrac{27 \times 0.0030}{(26)\,(25)\,(0.1427)^3}$

$\bar{Z} = 3.6071 \qquad C_s = 0.043$

The flood discharge for a given T is calculated as below. Here, values of K_z for given T and $C_s = 0.043$ are read from Table 7.6.

	$\bar{Z}$ = 3.6071	σ_z = 0.1427	C_s = 0.043	
T (years)	**K_z (from Table 7.6) (for C_s = 0.043)**	**$K_z \sigma_z$**	**$Z_T = \bar{Z} + K_z\sigma_z$**	**x_T = antilog z_T (m³/s)**
100	2.358	0.3365	3.9436	8782
200	2.616	0.3733	3.9804	9559
1000	3.152	0.4498	4.0569	11400

EXAMPLE 7.9 *For the annual flood series data analyzed in Example 7.8 estimate the flood discharge for a return period of (a) 100 years, (b) 200 years, and (c) 1000 years by using log-normal distribution. Compare the results with those of Example 7.8.*

SOLUTION: Log-normal distribution is a special case of log-Pearson type III distribution with $C_s = 0$. Thus in this case C_s is taken as zero. The other statistics are $\bar{z} = 3.6071$ and $\sigma_z = 0.1427$ as calculated in Example 7.8.

The value of K for a given return period T and $C_s = 0$ is read from Table 7.6. The estimation of the required flood discharge is done as shown below.

	$\bar{z}$ = 3.6071	σ_z = 0.1427	C_s = 0	
T (years)	**K_z (from Table 7.6)**	**$K_z\, \sigma_z$**	**$Z_T = \bar{Z} + K_z\, \sigma_z$**	**x_T = antilog z_T (m³/s)**
100	2.326	0.3319	3.9390	8690
200	2.576	0.3676	3.9747	9434
1000	3.090	0.4409	4.0480	11170

On comparing the estimated x_T with the corresponding values in Example 7.8, it is seen that the inclusion of the positive coefficient of skew ($C_s = 0.047$) in log-Pearson type III method gives higher values than those obtained by the log-normal distribution method. However, as the value of C_s is small, the difference in the corresponding values of x_T by the two methods is not appreciable.

[***Note:*** If the coefficient of skew is negative, the log-Pearson type III method gives consistently lower values than those obtained by the log-normal distribution method.]

7.8 PARTIAL DURATION SERIES

In the annual hydrologic data series of floods, only one maximum value of flood per year is selected as the data point. It is likely that in some catchments there are more than one independent floods in a year and many of these may be of appreciably high magnitude. To enable all the large flood peaks to be considered for analysis, a flood magnitude larger than an arbitrary selected base value are included in the analysis. Such a data series is called *partial-duration series.*

In using the partial-duration series, it is necessary to establish that all events considered are independent. Hence the partial-duration series is adopted mostly for rainfall analysis where the conditions of independency of events are easy to establish. Its use in flood studies is rather rare. The recurrence interval of an event obtained by annual series (T_A) and by the partial duration series (T_p) are related by

$$T_P = \frac{1}{\ln T_A - \ln (T_A - 1)} \tag{7.28}$$

From this it can be seen that the difference between T_A and T_P is significant for $T_A < 10$ years and that for $T_A > 20$, the difference is negligibly small.

7.9 REGIONAL FLOOD FREQUENCY ANALYSIS

When the available data at a catchment is too short to conduct frequency analysis, a regional analysis is adopted. In this a hydrologically homogeneous region from the statistical point of view is considered. Available long time data from neighbouring catchments are tested for homogeneity and a group of stations satisfying the test are identified. This group of stations constitutes a region and all the station data of this region are pooled and analysed as a group to find the frequency characteristics of the region. The mean annual flood Q_{ma}, which corres-ponds to a recurrence interval of 2.33 years is used for nondimensionalising the results. The variation of Q_{ma} with drainage area and the variation of Q_T/Q_{ma} with T where Q_T is the discharge for any T are the basic plots prepared in this analysis. Details of the method are available in Ref. 2.

7.10 DATA FOR FREQUENCY STUDIES

The flood-frequency analysis described in the previous sections is a direct means of estimating the desired flood based upon the available flood flow data of the catchment. The results of the frequency analysis depend upon the length of data. The minimum number of years of record required to obtain satisfactory estimates depends upon the variability of data and hence on the physical and climatological characteristics of the basin. Generally a minimum of 30 years of data is considered as essential. Smaller lengths of records are also used when it is unavoidable. However, frequency analysis should not be adopted if the length of records is less than 10 years.

In the frequency analysis of time series, such as of annual floods, annual yields and of precipitation, some times one comes across very long (say of the order of 100 years) times series. In such cases it is necessary to test the series for *Homogeneity* to ascertain that there is no significant difference in the causative hydrological processes over the span of the time series. A time series is called time-homogeneous (also known as *stationary*) if identical events under consideration in the series are likely to occur at all times. Departure from time homogeneity is reflected either in trend or periodicity or persistence of the variable over time. Potential non-homogeneity region, (if any), could be detected by (i) mass curve or (ii) by moving mean of the variable. Statistical tests like *F-test* for equality of variances and *t-test* for significance of differences of means are adopted to identify non- homogeneous region/s in the series. Only the contiguous homogeneous region of the series covering the recent past is to be adopted for frequency analysis. However, it is prudent to test all time series, whether long or short, for time-homogeneity before proceeding with the frequency analysis. Thus the cardinal rule with the data of time series would be that the data should be reliable and homogeneous.

Flood frequency studies are most reliable in climates that are uniform from year to year. In such cases a relatively short record gives a reliable picture of the frequency distribution.

7.11 DESIGN FLOOD

In the design of hydraulic structures it is not practical from economic considerations to provide for the safety of the structure and the system at the maximum possible flood in the catchment. Small structures such as culverts and storm drainages can be designed for less severe floods as the consequences of a higher than design flood may not be very serious. It can cause temporary inconvenience like the disruption of traffic and very rarely severe property damage and loss of life. On the other hand, storage structures such as dams demand greater attention to the magnitude of floods used in the design. The failure of these structures causes large loss of life and great property damage on the downstream of the structure. From this it is apparent that the type, importance of the structure and economic development of the surrounding area dictate the design criteria for choosing the flood magnitude. This section highlights the procedures adopted in selecting the flood magnitude for the design of some hydraulic structures.

The following definitions are first noted.

Design Flood Flood adopted for the design of a structure.

Spillway Design Flood Design flood used for the specific purpose of designing the spillway of a storage structure. This term is frequently used to denote the maximum discharge that can be passed in a hydraulic structure without any damage or serious threat to the stability of the structure.

Standard Project Flood (SPF) The flood that would result from a severe combination of meteorological and hydrological factors that are reasonably applicable to the region. Extremely rare combinations of factors are excluded.

Probable Maximum Flood (PMF) The extreme flood that is physically possible in a region as a result of severemost combinations, including rare combinations of meteorological and hydrological factors.

The PMF is used in situations where the failure of the structure would result in loss of life and catastrophic damage and as such complete security from potential floods is sought. On the other hand, SPF is often used where the failure of a structure would cause less severe damages. Typically, the SPF is about 40 to 60% of the PMF for the same drainage basin. The criteria used for selecting the design flood for various

Table 7.8 Guidelines for Selecting Design Floods (CWC, India)[1]

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

hydraulic structures vary from one country to another. Table 7.8 gives a brief summary of the guidelines adopted by CWC India, to select design floods.

The Indian Standard Guidelines for Design of Floods for Dams

"IS : 11223—1985 : Guidelines for fixing spillway capacity" (Ref. 4) is currently used in India for selection of design floods for dams. In these guidelines, dams are classified according to size by using the hydraulic head and the gross storage behind the dam. The hydraulic head is defined as the difference between the maximum water level on the upstream and the normal annual average flood level on the downstream. The classification is shown in Table 7.9(a). The overall size classifications for dams would be greater of that indicated by either of the two parameters. For example, a dam with a gross storage of 5 Mm^3 and hydraulic head of 15 m would be classified as *Intermediate* size dam.

Table 7.9(a) Size Classification of Dams

Class	Gross storage (Mm^3)	Hydraulic head (m)
Small	0.5 to 10.0	7.5 to 12.0
Intermediate	10.0 to 60.0	12.0 to 30.0
Large	> 60.0	> 30.0

The inflow design flood (IDF) for safety of the dam is taken for each class of dam as given in Table 7.9(b).

Table 7.9(b) Inflow Design Flood for Dams

Size/Class (based on Table 7.9(a))	Inflow design flood for safety
Small	100-year flood
Intermediate	Standard project flood (SPF)
Large	Probable Maximum flood (PMF)

7.12 DESIGN STORM

To estimate the design flood for a project by the use of a unit hydrograph, one needs the design storm. This can be the storm-producing probable maximum precipitation (PMP) for deriving PMF or a standard project storm (SPS) for SPF calculations. The computations are performed by experienced hydrometeorologists by using meteorological data. Various methods ranging from highly sophisticated hydrometeorological methods to simple analysis of past rainfall data are in use depending on the availability of reliable relevant data and expertise.

The following is a brief outline of a procedure followed in India:

- The duration of the critical rainfall is first selected. This will be the basin lag if the flood peak is of interest. If the flood volume is of prime interest, the duration of the longest storm experienced in the basin is selected.
- Past major storms in the region which conceivably could have occurred in the basin under study are selected. DAD analysis is performed and the enveloping curve representing maximum depth–duration relation for the study basin obtained.
- Rainfall depths for convenient time intervals (e.g. 6 h) are scaled from the enveloping curve. These increments are to be arranged to get a critical sequence which produces the maximum flood peak when applied to the relevant unit hydrograph of the basin.

  The critical sequence of rainfall increments can be obtained by trial and error. Alternatively, increments of precipitation are first arranged in a table of relevant unit hydrograph ordinates, such that (i) the maximum rainfall increment is against the maximum unit hydrograph ordinate, (ii) the second highest rainfall increment is against the second largest unit hydrograph ordinate, and so on, and (iii) the sequence of rainfall increments arranged above is now reversed, with the last item first and first item last. The new sequence gives the design storm (Example 7.8).
- The design storm is then combined with hydrologic abstractions most conducive to high runoff, viz. low initial loss and lowest infiltration rate to get the hyetograph of rainfall excess to operate upon the unit hydrograph.

  Further details about the above procedure and other methods for computing design storms are available in Ref. 7. Reference 1 gives details of the estimation of the design flood peak by unit hydrographs for small drainage basins of areas from 25–500 km^2.

Example 7.10 *The ordinates of cumulative rainfall from the enveloping maximum depth–duration curve for a basin are given below. Also given are the ordinates of a 6-h unit hydrograph. Design the critical sequence of rainfall excesses by taking the φ index to be 0.15 cm/h.*

Solution: The critical storm and rainfall excesses are calculated in a tabular form in Table 7.10.

Time from start (h)	0	6	12	18	24	30	36	42	48	54	60
Cumulative rainfall (cm)	0	15	24.1	30	34	37	39	40.5	41.3		
6-h UH ordinate (m^3/s)	0	20	54	98	126	146	154	152	138	122	106
Time from start (h)	66	72	78	84	90	96	102	108	114	129	132
6-h UH ordinate (m^3/s)	92	79	64	52	40	30	20	14	10	6	0

Table 7.10 Calculation of Critical Storm—*Example* 7.10

Time (h)	Cumulative rainfall (cm)	6-h incremental rainfall (cm)	Ordinate of 6-h UH (m^3/s)	First arrangement of rainfall increment	Design sequence of rainfall increment	Infiltration loss (cm)	Rainfall excess of design storm (cm)
1	2	3	4	5	6	7	8
0	0		0		0	0	0
6	15.0	15.0	20		1.5	0.9	0.6
12	24.1	9.1	54		2.0	0.9	1.1
18	30.0	5.9	98	0.8	4.0	0.9	3.1
24	34.0	4.0	126	3.0	9.1	0.9	8.2
30	37.0	3.0	146	5.9	15.0	0.9	14.1
36	39.0	2.0	154	15.0	5.9	0.9	5.0
42	40.5	1.5	152	9.1	3.0	0.9	2.1
48	41.3	0.8	138	4.0	0.8	0.9	0
54			122	2.0			
60			106	1.5			
66			92				
72			79				
78			64				
84			52				
90			40				
96			30				
102			20				
108			14				
114			10				
120			6				
132			0				

1. (Column 6 is reversed sequence of column 5)
2. Infiltration loss = 0.15 cm/h = 0.9 cm/6 h

7.13 RISK, RELIABILITY AND SAFETY FACTOR

RISK AND RELIABILITY

The designer of a hydraulic structure always faces a nagging doubt about the risk of failure of his structure. This is because the estimation of the hydrologic design values (such as the design flood discharge and the river stage during the design flood) involve a natural or inbuilt uncertainty and as such a hydrological risk of failure. As an example, consider a weir with an expected life of 50 years and designed for a flood magnitude of return period T = 100 years. This weir may fail if a flood magnitude greater than the design flood occurs within the life period (50 years) of the weir.

The probability of occurrence of an event ($x \geq x_T$) at least once over a period of n successive years is called the risk, $\bar{R}$. Thus the risk is given by

$\bar{R}$ = 1 – (probability of non-occurrence of the event $x \geq x_T$ in n years)

$$\bar{R} = 1 - (1 - P)^n$$

$$= 1 - \left(1 - \frac{1}{T}\right)^n \tag{7.29}$$

where P = probability $P(x \geq x_T) = \frac{1}{T}$

T = return period

The reliability R_e, is defined as

$$R_e = 1 - \bar{R} = \left(1 - \frac{1}{T}\right)^n \tag{7.30}$$

It can be seen that the return period for which a structure should be designed depends upon the acceptable level of risk. In practice, the acceptable risk is governed by economic and policy considerations.

SAFETY FACTOR

In addition to the hydrological uncertainty, as mentioned above, a water resource development project will have many other uncertainties. These may arise out of structural, constructional, operational and environmental causes as well as from non-technological considerations such as economic, sociological and political causes. As such, any water resource development project will have a safety factor for a given hydrological parameter M as defined below.

$$\text{Safety factor (for the parameter } M) = (SF)_m = \frac{\text{Actual value of the parameter } M \text{ adopted in the design of the project}}{\text{Value of the parameter } M \text{ obtained from hydrological considerations only}}$$

$$= \frac{C_{am}}{C_{hm}} \tag{7.31}$$

The parameter M includes such items as flood discharge magnitude, maximum river stage, reservoir capacity and free board. The difference ($C_{am} - C_{hm}$) is known as *safety margin.*

The concepts of risk, reliability and safety factor form the building blocks of the emerging field of reliability based design.

Example 7.11 *A bridge has an expected life of 25 years and is designed for a flood magnitude of return period 100 years. (a) What is the risk of this hydrologic design? (b) If a 10% risk is acceptable, what return period will have to be adopted?*

Solution:

(a) The risk $\bar{R} = 1 - \left(1 - \frac{1}{T}\right)^n$

Here $n = 25$ years and $T = 100$ years

$$\bar{R} = 1 - \left(1 - \frac{1}{100}\right)^{25} = 0.222$$

Hence the inbuilt risk in this design is 22.2%

(b) If $\bar{R} = 10\% = 0.10$ $\qquad 0.10 = 1 - \left(1 - \frac{1}{T}\right)^{25}$

$$\left(1 - \frac{1}{T}\right)^{25} = 0.90 \quad \text{and} \quad T = 238 \text{ years} = \text{say } 240 \text{ years.}$$

Hence to get 10% acceptable risk, the bridge will have to be designed for a flood of return period $T = 240$ years.

Example 7.12 *Analysis of annual flood series of a river yielded a sample mean of 1000 m³/s and standard deviation of 500 m³/s. Estimate the design flood of a structure on this river to provide 90% assurance that the structure will not fail in the next 50 years. Use Gumbel's method and assume the sample size to be very large.*

Solution: $\bar{x} = 1000 \text{ m}^3\text{/s}$ and $\sigma_{n-1} = 500 \text{ m}^3\text{/s}$

Reliability $\qquad R_e = 0.90 = \left(1 - \frac{1}{T}\right)^{50}$

$$1 - \frac{1}{T} = (0.90)^{1/50} = 0.997895$$

$$T = 475 \text{ years} \qquad x_T = \bar{x} + K\sigma_{n-1} \qquad K = \frac{y_T - 0.577}{1.2825}$$

Also, $\qquad y_T = -\left[\ln \cdot \ln \frac{475}{(475-1)}\right] = 6.16226$

$$K = \frac{6.16226 - 0.577}{1.2825} = 4.355$$

$$x_T = 1000 + (4.355) \times 500 = 3177 \text{ m}^3\text{/s}$$

Example 7.13 *Annual flood data of the river Narmada at Garudeshwar covering the period 1948 to 1979 yielded for the annual flood discharges a mean of 29,600 m³/s and a standard deviation of 14,860 m³/s. For a proposed bridge on this river near this site it is decided to have an acceptable risk of 10% in its expected life of 50 years. (a) Estimate the flood discharge by Gumbel's method for use in the design of this structure (b) If the actual flood value adopted in the design is 125,000 m³/s what are the safety factor and safety margin relating to maximum flood discharge?*

Solution: Risk $\bar{R} = 0.10$

Life period of the structure $n = 50$ years

Hence $\bar{R} = 0.10 = 1 - \left(1 - \frac{1}{T}\right)^{50}$

$$\left(1 - \frac{1}{T}\right) = (1 - 0.10)^{1/50} = 0.997895$$

$$T = 475 \text{ years}$$

Gumbel's method is now used to estimate the flood magnitude for this return period of $T = 475$ years.

Record length N = 1948 to 1979 = 32 years

From Tables 7.3 and 7.4, $\bar{y}_n = 0.5380$ and $S_n = 1.1193$

$$y_T = -\left[\ln.\ln\frac{T}{T-1}\right] = -\left[\ln.\ln\frac{475}{(475-1)}\right] = 6.16226$$

$$K = \frac{y_T - \bar{y}_n}{S_n} = \frac{(6.16226 - 0.5380)}{1.1193} = 5.0248$$

$$x_T = \bar{x}_T + K\,\sigma_{n-1}$$

$$= 29600 + (5.0248 \times 14860) = 104268$$

say = 105,000 m^3/s = hydrological design flood magnitude

Actual flood magnitude adopted in the project is = 125,000 m^3/s

Safety factor = $(SF)_{flood}$ = 125,000/105,000 = 1.19

Safety margin for flood magnitude = 125,000 – 105,000 = 20,000 m^3/s

References

1. Central Water Commission, India, "Estimation of design flood peak", Flood Estimation Directorate, *Report No.* 1/73, New Delhi, 1973.
2. Chow. V.T., *Handbook of Applied Hydrology*, McGraw-Hill, New York, NY, 1964.
3. Gray, D.M., *Principles of Hydrology,* Water Inf. Centre, Huntington, NY, 1970.
4. Indian Bureau of Standards, "*Guidelines for fixing Spillway Capacity*", IS: 11223 – 1985.
5. Khushalani, K.B. and M. Khushalani, *Irrigation Practice and Design.* Vol. 1, Oxford & IBH, New Delhi, 1971.
6. Nemec, J., *Engineering Hydrology,* Tata McGraw-Hill, New Delhi, 1973.
7. Sokolov, A.A., S.E. Rantz, and M. Roche, *Flood Computation— Methods Compiled from World Experience,* The UNESCO Press, Paris, 1976.
8. UNO/WMO, "Assessment of magnitude and frequency of flood flows", United Nations Pub., *Water Resources Series No. 30*, 1967.

Revision Questions

7.1 Explain the rational method of computing the peak discharge of a small catchment. Where is this method commonly used and what are its merits and demerits?

7.2 Discuss the factors affecting the runoff coefficient C in rational formula.

7.3 What do you understand by time of concentration of a catchment? Describe briefly methods of estimation of the time of concentration.

7.4 What is the importance of time of concentration of a catchment in the estimation of flood by rational formula?

7.5 Annual flood series having N consecutive entries are available for a catchment. Describe a procedure to verify whether the data follow Gumbel's distribution.

7.6 Write a brief note on frequency factor and its estimation in Gumbel's method.

7.7 If the annual flood series data for a catchment are available for *N* consecutive years, explain a procedure to determine a flood discharge with a return period of *T*, (where $T > N$), by using
(a) Log-Pearson type III distribution, and (b) Log-normal distribution.

7.8 What are the limitations of flood frequency studies?

7.9 Explain briefly the following terms:
(a) Design flood (b) Standard project flood
(c) Probable maximum flood (d) Design storm

7.10 What are the recommended design floods for
(a) Spillways of dams (b) Terrace outlets and vegetated waterways
(c) Field diversions (d) Permanent barrages
(e) Waterway for aqueducts

7.11 Explain briefly the following terms:
(a) Risk (b) Reliability (c) Safety margin

Problems

7.1 A catchment of area 120 ha has a time of concentration of 30 min and runoff coefficient of 0.3. If a storm of duration 45 min results in 3.0 cm of rain over the catchment estimate the resulting peak flow rate.

7.2 Information on the 50-year storm is given below.

Duration (minutes)	15	30	45	60	180
Rainfall (mm)	40	60	75	100	120

A culvert has to drain 200 ha of land with a maximum length of travel of 1.25 km. The general slope of the catchment is 0.001 and its runoff coefficient is 0.20. Estimate the peak flow by the rational method for designing the culvert for a 50-year flood.

7.3 A basin is divided by 1-h isochrones into four sub-areas of size 200, 250, 350 and 170 hectares from the upstream end of the outlet respectively. A rainfall event of 5-h duration with intensities of 1.7 cm/h for the first 2 h and 1.25 cm/h for the next 3 h occurs uniformly over the basin. Assuming a constant runoff coefficient of 0.5, estimate the peak rate of runoff.
(***Note:*** An *isochrone* is a line on the catchment map joining points having equal time of travel of surface runoff. See Sec. 8.8.)

7.4 An urban catchment of area 3.0 km^2 consists of 52% of paved areas, 20% parks, 18% multi-unit residential area. The remaining land use can be classified as light industrial area. The catchment is essentially flat and has sandy soil. If the time of concentration is 50 minutes, estimate the peak flow due to a design storm of depth 85 mm in 50 minutes.

7.5 In estimating the peak discharge of a river at a location *X* the catchment area was divided into four parts *A*, *B*, *C* and *D*. The time of concentration and area for different parts are as follows

Part	Time of Concentration	Area (in ha)
A	One Hour	600
B	Two Hours	750
C	Three Hours	1000
D	Four Hours	1200

Records of a rain storm lasting for four hours as observed and the runoff factors during different hours are as follows:

Time (in hours)		Rainfall (mm)	Runoff factor
From	To		
0	1	25.0	0.50
1	2	50.0	0.70
2	3	50.0	0.80
3	4	23.5	0.85

Calculate the maximum flow to be expected at X in m^3/s assuming a constant base flow of 42.5 m^3/s.

7.6 A catchment area has a time of concentration of 20 minutes and an area of 20 ha. Estimate the peak discharge corresponding to return period of 25 yrs. Assume a runoff coefficient of 0.25. The intensity-duration-frequency for the storm in the area can be expressed by $i = KT^x/(D + a)^n$, where i = intensity in cm/h, T = retum period in years, and D = duration of storm in hours, with coefficients $K = 6.93$, $x = 0.189$, $a = 0.50$, $n = 0.878$.

7.7 A 100 ha watershed has the following characteristics

i. Maximum length of travel of water in the catchment = 3500 m
ii. Difference in elevation between the most remote point on the catchment and the outlet = 65 m
iii. Land use/cover details:

Land use/cover	Area (ha)	Runoff coefficient
Forest	30	0.25
Pasture	10	0.16
Cuiltivated land	60	0.40

The maximum intensity – duration – frequency relationship for the watershed is given by

$$i = \frac{3.97\,T^{0.165}}{(D+0.15)^{0.733}}$$

where i = intensity in cm/h, T = Return period in years and D = duration of rainfall in hours. Estimate the 25-year peak runoff from the watershed that can be expected at the outlet of the watershed.

7.8 A rectangular paved area 150 m × 450 m has a longitudinal drain along one of its longer edges. The time of concentration for the area is estimated to be 30 minutes and consists of 25 minutes for over land flow across the pavement to the drain and 5 minutes for the maximum time from the upstream end of the drain to the outlet at the other end.

(a) Construct the isochrones at 5 minutes interval for this area.
(b) A rainfall of 7 cm/h occurs on this plot for D minutes and stops abruptly. Assuming a runoff coefficient of 0.8 sketch idealized outflow hydrographs for D = 5 and 40 minutes.

7.9 A rectangular parking lot is 150 m wide and 300 m long. The time of overland flow across the pavement to the longitudinal gutter along the centre is 20 minutes and the estimated total time of concentration to the downstream end of the gutter is 25 minutes. The coefficient of runoff is 0.92. If a rainfall of intensity 6 cm/h falls on the lot for 10 minutes and stops abruptly determine the peak rate of flow.

7.10 A flood of 4000 m^3/s in a certain river has a return period of 40 years. (a) What is its probability of exceedence? (b) What is the probability that a flood of 4000 m^3/s or greater magnitude may occur in the next 20 years? (c) What is the probability of occurrence of a flood of magnitude less than 4000 m^3/s?

7.11 Complete the following:

(a) Probability of a 10 year flood occurring at least once in the next 5 years is ________

(b) Probability that a flood of magnitude equal to or greater than the 20 year flood will not occur in the next 20 years is ________

(c) Probability of a flood equal to or greater than a 50 year flood occurring next year is ________

(d) Probability of a flood equal to or greater than a 50 year flood occurring three times in the next 10 years is ________

(e) Probability of a flood equal to or greater than a 50 year flood occurring at least once in next 50 years is ________

7.12 A table showing the variation of the frequency factor K in the Gumbel's extreme value distribution with the sample size N and return period T is often given in books. The following is an incomplete listing of K for T = 1000 years. Complete the table.

Sample size, N	25	30	35	40	45	50	55	60	65	70
Value of K (T, N) for T = 1000 years	5.842	5.727	—	5.576	—	5.478	—	—	—	5.359

7.13 The following table gives the observed annual flood values in the River Bhagirathi at Tehri. Estimate the flood peaks with return periods of 50, 100 and 1000 years by using:
(a) Gumbel's extreme value distribution, (b) log-Pearson type III distribution, and
(c) log-normal distribution

Year	1963	1964	1965	1966	1967	1968	1969
Flood discharge m^3/s	3210	4000	1250	3300	2480	1780	1860
Year	1970	1971	1972	1973	1974	1975	
Flood discharge m^3/s	4130	3110	2320	2480	3405	1820	

7.14 A hydraulic structure on a stream has been designed for a discharge of 350 m^3/s. If the available flood data on the stream is for 20 years and the mean and standard deviation for annual flood series are 121 and 60 m^3/s respectively, calculate the return period for the design flood by using Gumbel's method.

7.15 In a frequency analysis of rainfall based on 15 years of data of 10 minutes storm, the following values were obtained:
Arithmetic mean of data = 1.65 cm
Standard deviation = 0.45 cm
Using Gumbel's extremal distribution, find the recurrence interval of a storm of 10 minutes duration and depth equal to 3.0 cm. Assume the sample size to be very large.

7.16 For a data of maximum-recorded annual floods of a river the mean and the standard deviation are 4200 m^3/s and 1705 m^3/s respectively. Using Gumbel's extreme value distribution, estimate the return period of a design flood of 9500 m^3/s. Assume an infinite sample size.

7.17 The flood data of a river was analysed for the prediction of extreme values by Log-Pearson Type III distribution. Using the variate $z = \log Q$, where Q = flood discharge in the river, it was found that $\bar{z} = 2.510$, $\sigma_z = 0.162$ and coefficient of skew $C_s = 0.70$. (a) Estimate the flood discharges with return periods of 50, 100, 200 and 1000 years in this river. (b) What would be the corresponding flood discharge if log-normal distribution was used?

7.18 The frequency analysis of flood data of a river by using Log Pearson Type III distribution yielded the following data:
Coefficient of Skewness = 0.4

Return Period (T) (in yrs)	**Peak Flood (m^3/s)**
50	10,000
200	15,000

Given the following data regarding the variation of the frequency factor K with the return period T for C_s = 0.4, estimate the flood magnitude in the river with a return period of 1000 yrs.

Return Period (T)	:	50	200	1000
Frequency Factor (K)	:	2.261	2.949	3.670

7.19 A river has 40 years of annual flood flow record. The discharge values are in m^3/s. The logarithms to base 10 of these discharge values show a mean value of 3.2736, standard deviation of 0.3037 and a coefficient of skewness of 0.07. Calculate the 50 year return period annual flood discharge by,

(a) Log-normal distribution and

(b) Log-Pearson type III distribution.

7.20 The following data give flood-data statistics of two rivers in UP:

S. No.	River	Length of records (years)	Mean annual flood (m^3/s)	σ_{n-1}
1	Ganga at Raiwala	92	6437	2951
2	Yamuna at Tajewala	54	5627	3360

(a) Estimate the 100 and 1000 year floods for these two rivers by using Gumbel's method.

(b) What are the 95% confidential intervals for the predicted values?

7.21 For a river, the estimated flood peaks for two return periods by the use of Gumbel's method are as follows:

Return Period (years)	Peak flood (m^3/s)
100	435
50	395

What flood discharge in this river will have a return period of 1000 years?

7.22 Using 30 years data and Gumbel's method the flood magnitudes, for return periods of 100 and 50 years for a river are found to be 1200 and 1060 m^3/s respectively.

(a) Determine the mean and standard deviation of the data used, and

(b) Estimate the magnitude of a flood with a return period of 500 years.

7.23 The ordinates of a mass curve of rainfall from a severe storm in a catchment is given. Ordinates of a 12-h unit hydrograph applicable to the catchment are also given. Using the given mass curve, develop a design storm to estimate the design flood for the catchment. Taking the ϕ index as 0.15 cm/h, estimate the resulting flood hydrograph. Assume the base flow to be 50 m^3/s.

Time (h)	0	12	24	36	48	60	72	84	96	108	120	132
Cumulative rainfall (cm)	0	10.2	30.5	34.0	36.0							
12-h UH ordinate (m^3/s)	0	32	96	130	126	98	75	50	30	15	7	0

7.24 A 6-hour unit hydrograph is in the form of a triangle with a peak of 50 m^3/s at 24 hours from start. The base is 54 hours. The ordinates of a mass curve of rainfall from a severe storm in the catchment is as below:

Time (h)	0	6	12	18	24
Cumulative Rainfall (cm)	0	5	12	15	17.6

Using this data, develop a design storm and estimate the design flood for the catchment. Assume ϕ index = 0.10 cm/h and the base flow = 20 m^3/s.

7.25 A water resources project has an expected life of 20 years. (a) For an acceptable risk of 5% against the design flood, what design return period is to be adopted? (b) If the above return period is adopted and the life of the structure can be enhanced to 50 years, what is the new risk value?

7.26 A factory is proposed to be located on the edge of the 50 year flood plain of a river. If the design life of the factory is 25 years, what is the reliability that it will not be flooded during its design life?

7.27 A spillway has a design life of 20 years. Estimate the required return period of a flood if the acceptable risk of failure of the spillway is 10% (a) in any year, and (b) over its design life.

7.28 Show that if the life of a project n has a very large value, the risk of failure is 0.632 when the design period is equal to the life of the project, n.

(*Hint:* Show that $\left(1-\frac{1}{n}\right)^n = e^{-1}$ for large values of n)

7.29 The regression analysis of a 30 year flood data at a point on a river yielded sample mean of 1200 m^3/s and standard deviation of 650 m^3/s. For what discharge would you design the structure to provide 95% assurance that the structure would not fail in the next 50 years? Use Gumbel's method. The value of the mean and standard deviation of the reduced variate for $N = 30$ are 0.53622 and 1.11238 respectively.

7.30 Analysis of the annual flood peak data of river Damodar at Rhondia, covering a period of 21 years yielded a mean of 8520 m^3/s and a standard deviation of 3900 m^3/s. A proposed water control project on this river near this location is to have an expected life of 40 years. Policy decision of the project allows an acceptable reliability of 85%.

(a) Using Gumbel's method recommend the flood discharge for this project.

(b) If a safety factor for flood magnitude of 1.3 is desired, what discharge is to be adopted? What would be the corresponding safety margin?

Objective Questions

7.1 A culvert is designed for a peak flow Q_p on the basis of the rational formula. If a storm of the same intensity as used in the design but of duration twice larger occurs the resulting peak discharge will be

(a) Q_p (b) $2\,Q_p$ (c) $Q_p/2$ (d) Q_p^2

7.2 A watershed of area 90 ha has a runoff coefficient of 0.4. A storm of duration larger than the time of concentration of the watershed and of intensity 4.5 cm/h creates a peak discharge of

(a) 11.3 m^3/s (b) 0.45 m^3/s (c) 450 m^3/s (d) 4.5 m^3/s

7.3 A rectangular parking lot, with direction of overland flow parallel to the larger side, has a time of concentration of 25 minutes. For the purpose of design of drainage, four rainfall patterns as below are to be considered.

A = 35 mm/h for 15 minutes, B = 45 mm/h for 10 minutes,
C = 10 mm/h for 60 minutes, D = 15 mm/h for 25 minutes,

The greatest peak rate of runoff is expected in the storm

(a) A (b) B (c) C (d) D

7.4 For an annual flood series arranged in decreasing order of magnitude, the return period for a magnitude listed at position m in a total of N entries, by Weibull formula is

(a) m/N (b) $m/(N+1)$ (c) $(N+1)/m$ (d) $N/(m+1)$.

7.5 The probability that a hundred year flood may not occur at all during the 50 year life of a project is

(a) 0.395 (b) 0.001 (c) 0.605 (d) 0.133

7.6 The probability of a flood, equal to or greater than 1000 year flood, occurring next year is

(a) 0.0001 (b) 0.001 (c) 0.386 (d) 0.632

7.7 The probability of a flood equal to or greater than 50 year flood, occurring at least one in next 50 years is
(a) 0.02 (b) 0.636 (c) 0.364 (d) 1.0

7.8 The general equation for hydrological frequency analysis states that x_T = value of a variate with a return period of T years is given by x_T =
(a) $\bar{x} - K\sigma$ (b) $\bar{x}/K\sigma$ (c) $K\sigma$ (d) $\bar{x} + K\sigma$

7.9 For a return period of 100 years the Gumbel's reduced variate y_T is
(a) 0.0001 (b) 0.001 (c) 0.386 (d) 0.632

7.10 An annual flood series contains 100 years of flood data. For a return period of 200 years the Gumbel's reduced variate can be taken as
(a) 5.296 (b) –4.600 (c) 1.2835 (d) 0.517

7.11 To estimate the flood magnitude with a return period of T years by the Log–Pearson Type III method, the following data pertaining to annual flood series is sufficient
(a) Mean, standard deviation and coefficient of skew of discharge data
(b) Mean and standard deviation of the log of discharge and the number of years of data
(c) Mean, standard deviation and coefficient of skew of log of discharge data
(d) Mean and standard deviation of the log of discharges

7.12 If the recurrence interval of an event is T_A in annual series and T_p in partial duration series, then
(a) T_A is always smaller than T_p
(b) Difference between T_A and T_p is negligible for $T_A < 5$ years
(c) Difference between T_A and T_p is negligible for $T_A > 10$ years
(d) Difference between T_A and T_p is not negligible till $T_A > 100$ years

7.13 The term mean annual flood denotes
(a) Mean floods in partial-duration series
(b) Mean of annual flood flow series
(c) A flood with a recurrence interval of 2.33 years
(d) A flood with a recurrence interval of N/2 years, where N = number of years of record.

7.14 The use of unit hydrographs for estimating floods is generally limited to catchments of size less than
(a) 5000 km^2 (b) 500 km^2 (c) 10^6 km^2 (d) 5000 ha

7.15 The probable maximum flood is
(a) The standard project flood of an extremely large river
(b) A flood adopted in the design of all kinds of spillways
(c) An extremely large but physically possible flood in the region
(d) The maximum possible flood that can occur anywhere in the country

7.16 The standard project flood is
(a) Smaller than probable maximum flood in the region
(b) The same as the design flood used for all small hydraulic structures
(c) Larger than the probable maximum flood by a factor implying factor of safety
(d) The same as the probable maximum flood

7.17 A hydraulic structure has been designed for a 50 year flood. The probability that exactly one flood of the design capacity will occur in the 75 year life of the structure is
(a) 0.02 (b) 0.220 (c) 0.336 (d) 0.780

7.18 The return period that a designer must use in the estimation of a flood for a hydraulic structure, if he is willing to accept 20% risk that a flood of that or higher magnitude will occur in the next 10 years is
(a) 95 years (b) 75 years (c) 45 years (d) 25 years

7.19 A hydraulic structure with a life of 30 years is designed for a 30 year flood. The risk of failure of the structure during its life is
(a) 0.033 (b) 0.638 (c) 0.362 (d) 1.00

7.20 A bridge is designed for a 50 year flood. The probability that only one flood of the design capacity or higher will occur in the 75 years life of the bridge is
(a) 0.020 (b) 0.220 (c) 0.786 (d) 0.336

Chapter 8

Flood Routing

8.1 INTRODUCTION

The flood hydrograph discussed in Chap. 6 is in fact a wave. The stage and discharge hydrographs represent the passage of waves of the river depth and discharge respectively. As this wave moves down the river, the shape of the wave gets modified due to various factors, such as channel storage, resistance, lateral addition or withdrawal of flows, etc. When a flood wave passes through a reservoir, its peak is attenuated and the time base is enlarged due to the effect of storage. Flood waves passing down a river have their peaks attenuated due to friction if there is no lateral inflow. The addition of lateral inflows can cause a reduction of attenuation or even amplification of a flood wave. The study of the basic aspects of these changes in a flood wave passing through a channel system forms the subject matter of this chapter.

Flood routing is the technique of determining the flood hydrograph at a section of a river by utilizing the data of flood flow at one or more upstream sections. The hydrologic analysis of problems such as flood forecasting, flood protection, reservoir design and spillway design invariably include flood routing. In these applications two broad categories of routing can be recognised. These are:

1. Reservoir routing, and
2. Channel routing.

In *Reservoir routing* the effect of a flood wave entering a reservoir is studied. Knowing the volume-elevation characteristic of the reservoir and the outflow-elevation relationship for the spillways and other outlet structures in the reservoir, the effect of a flood wave entering the reservoir is studied to predict the variations of reservoir elevation and outflow discharge with time. This form of reservoir routing is essential (i) in the design of the capacity of spillways and other reservoir outlet structures, and (ii) in the location and sizing of the capacity of reservoirs to meet specific requirements.

In *Channel routing* the change in the shape of a hydrograph as it travels down a channel is studied. By considering a channel reach and an input hydrograph at the upstream end, this form of routing aims to predict the flood hydrograph at various sections of the reach. Information on the flood-peak attenuation and the duration of high-water levels obtained by channel routing is of utmost importance in flood-forecasting operations and flood-protection works.

A variety of routing methods are available and they can be broadly classified into two categories as: (i) hydrologic routing, and (ii) hydraulic routing. Hydrologic-routing methods employ essentially the equation of continuity. Hydraulic methods, on the other hand, employ the continuity equation together with the equation of motion of unsteady flow. The basic differential equations used in the hydraulic routing, known as St. Venant equations afford a better description of unsteady flow than hydrologic methods.