LECTURE 11

DESIGN FLOOD ESTIMATION USING UNIT HYDROGRAPH APPROACH

OBJECTIVES

The objective of the lecture is to provide the necessary understanding about the design flood estimation technique based on unit hydrograph approach for any major or medium hydraulic structure located in small or moderate size catchment.

11.1 INTRODUCTION

The engineers and scientists, involved in the design of water resources structures such as storm sewers, spillways, diversion works, bridges, culverts and other flood control works, often require the design flood at a certain location in order to estimate the size and cost of those structures. 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 higher than the 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 demands 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 down stream 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. Before going into the details of procedures adopted in selecting the flood magnitude for the design of some hydraulic structures, the following definitions are first to be noted :

(*i*) *Design flood* ; Flood adopted for the design of a structure.

(*ii*) *Spillway Design flood* : Design flood used for the specific purpose of the designing the spillways of storage structures. 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 structures.

(*iii*) *Probable maximum flood* (*PMF*) *:* It is the flood discharge that may be expected from the most severe combination of critical meteorological and hydrological conditions that are reasonably possible in the region. The probable maximum flood is used in the design of projects for example, dams—where virtually complete security from potential floods is sought.

(*iv*) *Standard Project Flood* (*SPF*) *:* It is the flood discharge that may be expected from the most severe combination of meteorological and hydrological conditions that is considered reasonably characteristic of the geographic area in which the study drainage basin is located, extremely rare combinations of those conditions are not considered. The peak discharge for a standard project flood is generally about 40 to 60 per cent of that for the probable maximum flood for the same drainage basin. The standard project flood is often used where failure of the structure would have some what less disastrous effect. For example, it is used in the design of flood control facilities whose overtaxing or failure might be disastrous.

The criteria used for selecting the design flood for various hydraulic structures vary from one country to other. Table 11.1 gives a brief summary of the guidelines adopted by CWC (1969) to select design floods in India.

Table 11.1

Guidelines for selecting Design floods

Sl. No.	Structure	Recommended Design flood
1.	Spillways for major and medium projects with storage more than 60Mm^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 M m^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 upon the importance of the project.
4.	Aqueducts	
	(a) Waterway	Flood with T = 50 years
	(b) Foundations and free board	Flood with T = 100 years
5.	Project with very scanty or inadequate data	Empirical formulae

11.2 METHODS OF FLOOD ESTIMATION

The following methods are generally used for flood estimation:

(i) Rational method
(ii) Empirical method
(iii) Flood frequency method
(iv) Unit hydrograph technique and
(v) Watershed models.

The rational method, empirical method and flood frequency method are generally used for estimating the magnitude of flood peak. However, the unit hydrograph technique and watershed models can be used to estimate the design flood hydrograph in addition to the magnitude of design flood peak. The use of a particular methods depends upon (i) the desired objective (ii) the available data and (iii) the importance of the project.

The rational formula is only applicable to small size (< 50 km^2) catchments. The empirical formulae are essentially the regional formulae based on statistical correlation of the observed peak and important catchment properties. These formulae are only applicable in the region from which they were developed within the range of flood peaks used. If these formulae are applied to other areas they can at best give approximate values. The frequency analysis approach is the statistical methods to predict the flood peaks of a specified return period. The unit hydrograph method is basically a rainfall runoff relationship normally applicable to moderate size catchments with area less than 5000 sq. km. With the advent of high speed digital computers multiparameters physical watershed models have been developed by many investigators. Those models take the physical behaviour of the catchments into consideration. But the scope of such models are somewhat limited as far as design flood estimation is concerned specially in the inadequate data situation. The reason for this is that the uncertainty involved in the estimation of parameters of the models may lead to erroneous design flood estimates.

In the present lecture, the method applicable to small catchments (less than 5000 sq. km) based on unit hydrograph approach would be discussed in detail for the estimation of design flood. The unit hydrograph technique along with routing can be used to estimate the design flood for larger sized basins (greater than 5000 sq. km.) by dividing the basin into sub-basins.

11.3 COMPUTATION OF DESIGN FLOOD HYDROGRAPH BASED ON UNIT HYDROGRAPH APPROACH FOR SMALL AND MEDIUM SIZED BASINS

The unit hydrograph method of determination of design flood hydrographs for drainage basins less than a few thousand square kilometers in size (i.e. when a single unit hydrograph could be applied to the entire basin) involves the following steps :

(a) Derivation of design storm :

In estimating the critical design rainfall for a basin it is necessary to consider the size, configuration and runoff characteristics of the basin, as well as the meteorological characteristics of the major storms in the general region. In some basins particularly the smaller ones high intense storms of relatively short duration produce critical discharges, whereas in others, particularly the largers ones, storms of larger intensity and larger duration produce the most severe floods. As a general rule, metereological conditions that result in the most intense rainfall rates over small areas differ from those that cause maximum precipitations over large areas.

In estimating the design storm, estimates are made of both the total depth of storm rainfall and depths of rainfall in each of the increments of time used in computing the discharge hydrograph. The most critical sequence of these increments is obtained by trying various sequences in hydrograph computation (U.S. Army Corps of Engineers, 1959) to determine which sequence gives the highest peak discharge. Analysis of great number of storms have shown that the time distribution of rainfall intensities over an area of thousands of square kilometers may exhibit a variety of pattern in which the maximum intensities may occur at the beginning, middle or end of the storm period.

The usual variation in the spatial, areal distribution of storm rainfall in small basins may be taken into account when selecting infiltration indices and when deriving the hydrographs which reflect critical conditions. For larger drainage basins, however, the infiltration during time increments of the storm is proportional to the areas covered by rainfall intensities that exceed the soil infiltration capacity. Therefore, for larger drainage area, assumptions must be made concerning both the time and areal distribution of rainfall intensities during the design storm. It must also be remembered, too, that the infiltration capacity of a soil decreases towards some lower limit with increase in duration of the storm.

The technique of computing probable maximum precipitation for small catchments in India is stated in the following steps :

(i) A critical storm duration is selected before storm analysis begins. If the principal interest is in peak discharge, the duration of rainfall used is equal to the basin lag time, if the principal interest is in flood volume as in reservoir design the rainfall duration used is that of the largest storm experienced in the basin. However. a full analysis of all major storms experienced in the basin is recommended for possible future use.

(ii) A rainfall depth-area-duration (DAD) analysis is made using isohyet increments directly for each selected stofm.

(iii) Maximum observed rainfall depths for one or more days as obtained from non recording raingauges, are recomputed to give rainfall depths for multiples of 24 hours, using coefficients greater than unit, as obtained from a comparison of non recording raingauge records and any available nearby recording precipitation gauge. An envolving curve of depth duration data is drawn for the study basin.

(iv) Now, increments of rainfall are first aligned to match the ordinates of the design unit hydrograph (computational procedure for the design unit hydrograph is explained later in this lecture) so that the position of the maximum depth increment is matched with the maximum unit hydrograph ordinate, the position of second largest depth increment is matched with the second largest unit hydrograph ordinate and so on. The sequence of rainfall increments is then reversed to get design sequence of precipitation increments.

(b) Calculation of effective rainfall of design storm :

Generally the infiltration index and the initial losses values are derived from the available rainfall-runoff records for the severe storms in the basin using the procedure described in lecture no 5 Assuming the basin would be saturated at the time of design storm the minimum infiltration rate and initial losses values would be considered. The minimum infiltration rate and minimum initial loss, thus obtained, are used to compute the effective rainfall of design storm. For this the initial losses must be subtracted first from the rainfall increments and thereafter a uniform loss rate equal to the minimum infiltration index is applied. Unless the minimum infiltration index value has been derived from records available for a very few severe storms that have occured on wet soils much reliance can not be placed on its use in computing runoff from the design storm particularly when the design storm is the probable maximum storm.

From a study conducted by CWC (1973) on rainfall-runoff correlation, the following relationships for the estimation of uniform loss rate (ϕ-index) are envisaged for flood producing storms and soil conditions prevalent in India :

$$R = a\, I^{1.2}$$

$$\text{and } \phi = \frac{I-R}{24}$$

where R = Runoff in cm from a 24 h rainfall of intensity I cm/day and

a = a co-efficient which depends upon the soil type as given in the following table :

S. No.	Types of soil	Co-efficient a
1.	Sandy soils and sandy loam	0.17 — 0.24
2.	Coastal alluvium and silty loam	0.25 — 0 34
3.	Red soils, clayey loam, greay and brown alluvium	0 42
4.	Black cotton and clayey soils	0.41 — 0.46
5.	Hilly soils	0.46 — 0.50

In the absence of any other data, an assumption has to be made regarding the value of ϕ-index. U.S, Army Crops of Engineers has recommended to use an infiltration index of 1.0 mm/hour for the first 12 hours, 0.8 mm per hour for the subsequent 12 hours and 0.5 mm per hour thereafter. An alternative practice being used in India recommended by CWC as the use of constant loss rate of 1.0 mm per hour through out the storm. The rates adopted for a particular project are primarily influenced by the nature of soil type and cover with the basin and the purpose of the estimate.

For computing infiltration index for basin under study with inadequate records, another possibllity-could be from the values of infiltration index for adjacent gauged catchments with similar hydrologic characteristics. In case, the available data for the catchments located in the region are limited, it may be possible to estimate subjectively the runoff percentage for the design storm by assessing how much higher than the observed percentage it might be due to wetter antecedent conditions and greater rainfall amounts.

Regarding initial losses, it may be assumed to be zero during the period of design storm unless some sound evidence is there to consider some amount as initial losses.

(c) Derivation of design unit hydrograph.

Unit hydrographs for the drainage area are derived from the discharge records or by regional unit hydrograph relationships as discussed in earlier lectures. The unit hydrograph analysis technique based on Clark's approach may be preferred over the conventional techniques such as collin's method etc.

When a number of unit hydrographs have been developed for the project basin a selection has to be made to determine the unit hydrograph which will result in the best estimate of the design flood. Generally a normal unit hydrograph is developed for this purpose. Such a unit hydrograph applies to normal or average storm patterns, and is considered suitable for use in computing design floods of all magnitudes but the probable maximum flood.

In a common procedure of constructing a normal unit hydrograph of desired duration, the unit hydrograph of the same duration derived from different events are averaged using an averaging procedure described in the lecture-6. The normal unit hydrograph thus developed should be tested by reproducing observed hydrographs of major floods by applying the effective rainfall of corresponding storms.

If the normal unit hydrograph. to be used for design flood computations. is computed from a recorded flood hydrograph that represented rainfall distributions and hydraulic conditions which are not likely to be greatly different from those during the design storms, the derived normal unit hydrograph can be used without modification. However, if the unit hydrograph has been derived from a minor flood hydrograph, it may be necessary to modify the derived unit hydrograph for use with the design storm. The reason for differences between unit hydrograph derived from major and minor flood hydrographs, probably result from differences in the areal distribution of rainfall and differences in hydraulic conditions between major and minor floods. Since the data for the determination of unit hydrograph are usually limited to relatively minor flood occurrences, it is therefore necessary to increase the peak ordinate of the normal unit hydrograph to represent higher concentration of runoff. CWC recommended an increase of 25 to 50 percent in peak ordinate of the normal unit hydrograph. The magnitude of increase may be still higher if the unit hydrographs are derived from smaller floods. The increase in the peak discharges necessitates a general modification of the normal unit hydrograph to preserve the unit volume of the unit hydrograph. The modified unit hydrograph is generally assumed to apply to all unit periods of the probable maximum storm. The U.S. Army Corps Engineers practice, however, is to apply the normal unit hydrograph to all unit periods of the probable maximum storm except the largest to which the modified unit hydrograph is applied.

(d) Application of effective design rainfall to unit hydrograph

The critical sequence of the effective rainfall are applied to the design unit hydrograph to obtain the total design direct surface runoff hydiograph using either of the following two methods :

(i) tabular method

(ii) graphical method

(*i*) *Tabular method* : For the estimation of design direct surface runoff hydrograph from the effective design rainfall the following procedure is followed. (Ref. Table 11.1)

— Obtain the effective rainfall volumes (in mm) in each period after muliplying the effective rainfall intensity by the time unit. In Table 11.2, P_{E1}, P_{E2} and P_{E3} represent the effective rainfall (in mm) in each period and are determined by multiplying P_{e1}, P_{e2} and P_{e3} by the time unit.

- Multiply the first effective rainfall increment successively by each of the unit hydrograph ordinates. The resulting quantities represent the ordinates of the direct surface runoff hydrograph produced by the first increment of effective rainfall.
- Similarly computes the direct surface runoff hydrographs resulting due to second, third etc. increments of effective rainfall.
- Compute the total direct surface runoff from effective rainfall amounts by adding the direct surface runoff resulting due to different increments of rainfall in proper time relation being lagged successively by a time interval equal to the unit duration of the unit hydrograph.

While applying the design unit hydrograph to the design effective rainfall the following point must be noted :

Note :— If P_E 's are in mm the unit hydrograph must be for 1 mm. However, if the unit hydrograph is for 100 mm, the P'_E s must be proportions of 100 mm. For example, if unit hydrograph is for 1 mm and $P_E = 10$ mm then adjusted $P_E = 10$, Otherwise, if unit hydrograph is for 100 mm and $P_E = 10$ mm, then adjusted $P_E = 0.10$

Table 11.1 Estimation of direct surface runoff

Time Units t	Rainfall Intensity mm/hr	Loss rate ϕ mm/hr	Rainfall Excess mm/hr	Rainfall Excess mm	Surface Runoff Component Derived from Unitgraph (m^3/s) U_1	U_2	U_3	U_4	Average Surface Runoff for TimeUnit (m^3/s)
0									
	P_1	ϕ	P_{e_1}	P_{E1}	$P_{E1}U_1+$	0 +	0 +	0	$= q_1$
1									
	P_2	ϕ	P_{e_2}	P_{E2}	$P_{E2}U_1+$	0 +	+	0	$= q_2$
2									
	P_3	ϕ	P_{e_3}	P_{E3}	$P_{E3}U_1+$	$P_{E2}U_2+$	$P_{E1}U_3+$	0	$= q_3$
3									
					0	$+ P_{E3}U_2+$	$P_{E2}U_3+$	$P_{E1}U_4$	$= q_4$
4									
					0	+ 0	$+ P_{E3}U_3+$	$P_{E2}U_4$	$= q_5$
5									
					0	+ 0	+ 0	$+ P_{E3}U_4$	$= q_6$

(ii) Graphical derivation of Surface Runoff Hydrograph :

Instead of using a tabular method the hydrograph can also be determined graphically as shown in Fig. 11.1.

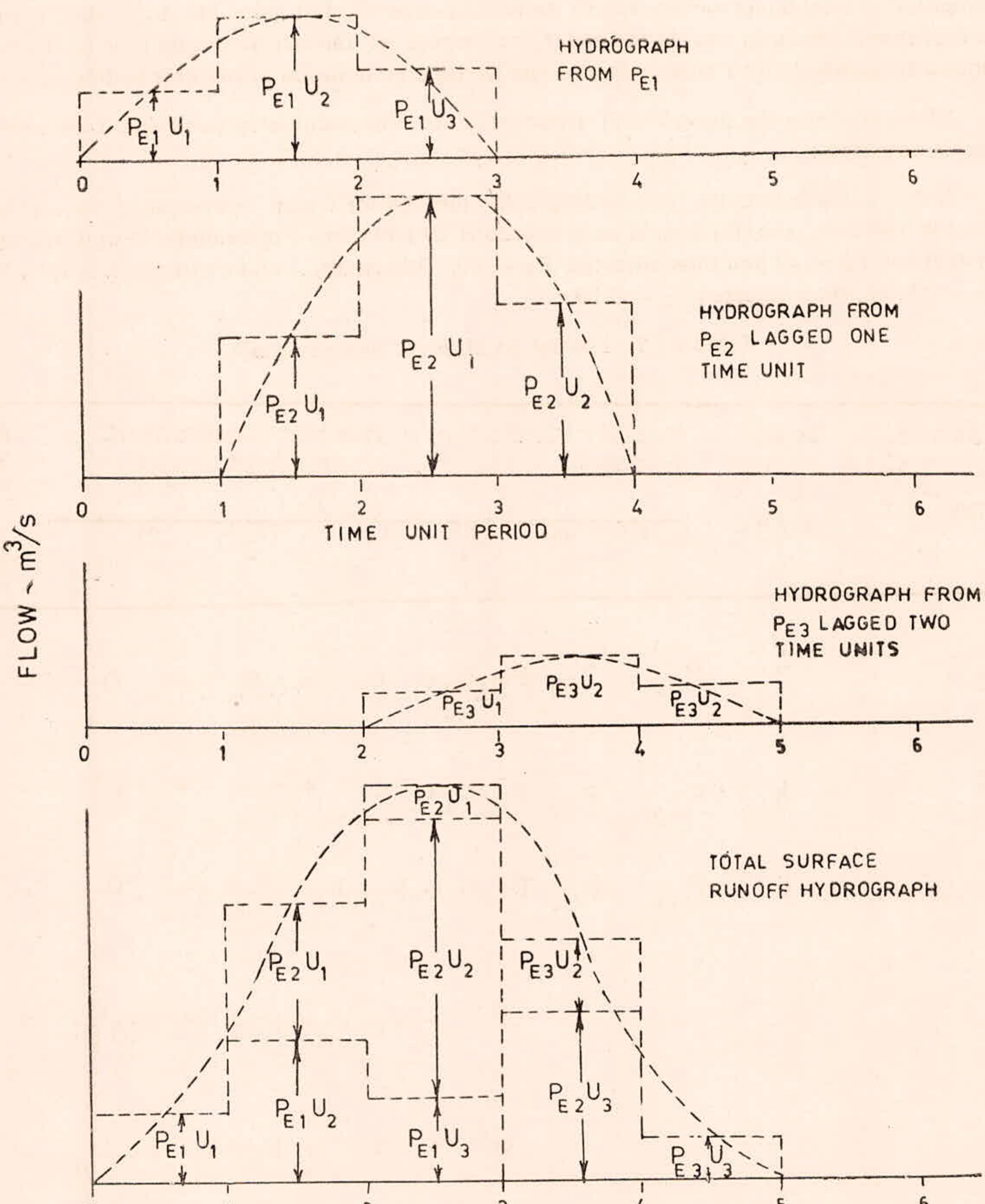

Fig. 11.1 Graphical Derivation of Surface Runoff Hydrograph

(e) Design flood hydrograph :

To obtain the design flood hydrograph of total runoff, the baseflow expected during the design storm is estimated from an analysis of flood of record in the basin and is added to the total direct surface runoff ordinates obtained in the previous step.

When computing design flood hydrographs for ungauged areas the estimated base flow rates of similar gaued basins expressed in cumecs per sq. kilometre of the catchment area are used as a guide to estimate the design base flow. However, a study conducted for baseflow for small catchments revealed that baseflow during flood season varies from 0.05 cumec/sq km to 0.44 cumec/sq km depending upon the meteorological zones in which the basins are located. The values given below were considered reasonable :

	Region	Baseflow (cumec/sq km)
1.	For Luni, Chambal, Sone, Punjab plain, Gangetic plains, upper Narmada & Tapi basins, upper Godavari, Krishna, Cauvery, J & K, Kumaon Hills.	0.05
2.	Betwa, Mahi, Sabarmati, Lower-Narmada, and Tapi basins, Lower Godavari, Indrawati Basin, East Coast, Terai.	0.11
3.	Mahanadi basin, west coast.	0.22
4.	Brahmputra basin	0.44

The following example illustrates the computational procedure for design storm and design flood.

Example 11.1 : The ordinates of cumulative rainfall from a severe storm in a catchment is given Ordinates of a 6 hour unit hydrograph applicable to the catchment are also given. Develop a design storm to estimate the design flood for catchment. Taking initial loss = 1.2 cm and ϕ index as 0.15 cm/hour, estimate the resulting design flood hydrograph. Assuming the base flow to be 300 m^3/s and the unit hydrograph principle is applicable for the catchment.

Time (hour)	Commulative Rainfall (cm)	6-hour unit hydrograph ordinate (m^3/s)
6	16.5	30
12	24.5	190
18	30.0	540
24	34.2	700
30	37.2	590
36	39.3	330
42	40.8	200
48	42.0	140
54		100
60		75
66		56
72		40
78		22
84		12
90		4
96		0

Solution :—The computational steps are :

(i) Enter the time intervals in column (1) of Table 11.2

(ii) Enter the cummulative rainfall (cm) in column (2) of Table 11.2

(iii) Compute 6 hour incremental rainfall and enter them in column (3) of Table 11.2

(iv) Enter the given ordinates of 6 hour unit hydrograph in column (4) of Table 11.2

(v) Arrange the maximum rainfall increment against the maximum unit hydrograph ordinate, the second highest rainfall increment against the second largest unit hydrograph and so on as shown in column (5) of Table 11.2

(vi) Reverse the squence of rainfall increments obtained from step (v) in such a way that the last item becomes first and the first item, becomes last. This sequence of rainfall increment is called as design sequence of rainfall increments. Enter those values in column (6) of Table 11.2

(vii) Enter the initial loss and infiltration loss (=0.9 cm.) in column (7) of Table 11.2

(viii) Compute the rainfall excess of design storm after subtracting the infiltration loss from the design sequence of rainfall increments. Enter those values in column (8) of Table 11.2

(ix) Enter the time from start of effective rainfall in hours in column (1) of Table 11.3

(x) Enter the 6-hour unit hydrograph ordinates in column (2) of Table 11.3

(xi) Enter the excess rainfall for design storm obtained at step (viii) in column (3) of Table 11.3

(xii) Compute the direct surface runoff in cumces due to different excess rainfall and enter them in column (4), (5), (6), (7), (8), (9) and (10) of Table 11.3 respectively.

(xiii) Compute the total direct surface runoff ordinates at each time interval after adding the corresponding values of column (4), (5), (6), (7), (8), (9) and (10). Enter these values in column (11) of Table 11.3

(xiv) Enter the base flow values in column (12) of Table 11.3

(xv) Add the values of column (11) and column (12) and enter them in column (13) of Table 11.3. These values are the design flood hydrograph ordinates.

Table 11.2 Calculation of Design Storm

Time (hours)	Cummulative rainfall (cm)	6-hour incremental rainfall (cm)	Ordinate of 6-hour Unit hydrograph (m^3/s)	First arrangement of rainfall increment (cm)	Design sequence of rainfall increment (cm)	Infiltration loss (cm)	Rainfall excess of design storm (cm)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
6	16.5	16 5	30	0	1.2	1.2	0
12	24.5	8.0	190	2.1	1.5	0.9	0.6
18	30.0	5.5	540	5.5	3.0	0.9	2.1
24	34.2	4.2	700	16.5	4.2	0.9	3.3
30	37.2	3.0	590	8.0	8.0	0.9	7.1
36	39.3	2.1	330	4.2	16.5	0.9	15.6
42	40.8	1.5	200	3.0	5.5	0.9	4.6
48	42.0	1.2	140	1.5	2.1	0.9	1.2
54			100	1.2	0		
60			75				
66			56				
72			40				
78			22				
84			12				
90			4				
96							

Table 11.3 Computation of Design Flood Hydrograph

Time from start of effect rain fall hours	6 hour unit hydrograph ordinates (m^3/s)	Effective rainfall increments for six hour period (cm)	Direct runoff in m^3/s due to effective rainfall increments of							Total direct runoff in (m^3/s)	Base flow in m^3/s	Design flood hydrograph ordinates (m^3/s)
			0.6 cm	2.1 cm	3.3 cm	7.1 cm	15.6 cm	4.6 cm	1.2 cm			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
6	30	0.6	18.0	-	-	-	-	-	-	18.0	300	318.0
12	190	2.1	114.0	63.0	-	-	-	-	-	177.0	300	477.0
18	540	3.3	324.0	399.0	99.0	-	-	-	-	822.0	300	1122.0
24	700	7.1	420.0	1134.0	627.0	213.0	-	-	-	2394.0	300	2694.0
30	590	15.6	354.0	1470.0	1782.0	1349.0	468.0	-	-	5423.0	300	5723.0
36	330	4.6	198.0	1239.0	2310.0	3834.0	2694.0	138.0	-	10683.0	300	10983.0
42	200	1.2	120.0	693.0	2947.0	4970.0	8424.0	874.0	36.0	17064.0	300	17364.0
48	140		84.0	420.0	1089.0	4189.0	10920.0	2484.0	228.0	19414.0	300	19714.0
54	100		60.0	294.0	660.0	2343.0	9204.0	3220.0	648.0	16429.0	300	16729.0
60	75		45.0	210.0	462.0	1420.0	5148.0	2714.0	840.0	10839.0	300	11139.0
66	55		33.6	157.5	330.0	994.0	3120.0	1518.0	708.0	6861.1	300	7161.1

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	7(11)	(12)	(13)
72	40		24.0	117.6	247.5	710.1	2184.0	920.0	396.0	4599.1	300	4899.1
78	22		13.2	84.0	184,8	532.5	1560.0	644.0	240.0	3258.5	300	3558.5
84	12		7.2	46.2	132.0	397.6	1170.0	460.0	168.0	2381.0	300	2681.0
90	4		2.4	25.2	72.6	284.0	873.6	345.0	120.0	1722.8	300	2022.8
96	0		0	8.4	39.6	156.2	624.0	251.6	90.0	1175.8	300	1475.8
102				0	13.2	85.2	334.2	184.0	67.2	683.8	300	983.8
108					0	28.4	187.2	101.2	48.0	364.8	300	664.8
114						0	62.4	55.2	26.4	144.0	300	444.0
120							0	18.4	14.4	32.8	300	332.8
126								0	4.8	4.8	300	304.8
132									0	0	300	300.0

BIBLIOGRAPHY

1. Chow, V.T. (1964), 'Runof, Section 14, Hand Book of Applied Hydrology, Mc. Graw Hill Book Co., IOC, New York.

2. C.W.C. (1969) 'Flood Estimation by Unit Hydrography Method', Chapter 5, CWC Recommended Procedure.

3. C.W.C. (1973) 'Estimation of design flood peak' Flood Estimation Directorate Report No. 1/73, New Delhi.

4. U.S. Army Corps of Engineers (1959), 'Flood Hydrograph Analysis and Applications', Engineering Manual 1110-2-1405, U.S. Army, Washington, D.C.

5. U.S Army Corps of Engineers (1965), 'Hydraulic Design of Spillways', Engineering Manual 1110-2-1603, U.S. Army, Washington, D.C.