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Drainage Principles and Applications

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surface and channel storage have been depleted, the depletion of the groundwater system continues. Thus the recession limb of the hydrograph of total runoff will eventually merge into the groundwater depletion curve. It is commonly assumed that the depletion of a groundwater system can be described by an exponential function; in other words, the groundwater depletion curve should produce a straight line when plotted on semi-logarithmic paper. So, the point where the recession limb of the hydrograph of total runoff merges into a straight line when plotted on semi-log paper, designates the time when both surface and channel storage have been depleted and direct runoff has come to an end (Point B in Figure 4.7B).

A simplified procedure to separate the direct runoff from the groundwater runoff is to draw a straight line between Points A and B (Figure 4.7A). The shaded area in Figure 4.7.A represents the total volume of direct runoff, which is the sum of overland flow and interflow. The time interval (A) – (B) designates the duration of direct runoff and is called the base length of the hydrograph of direct runoff.

4.4 The Curve Number Method

For drainage basins where no runoff has been measured, the Curve Number Method can be used to estimate the depth of direct runoff from the rainfall depth, given an index describing runoff response characteristics.

The Curve Number Method was originally developed by the Soil Conservation Service (Soil Conservation Service 1964; 1972) for conditions prevailing in the United States. Since then, it has been adapted to conditions in other parts of the world. Although some regional research centres have developed additional criteria, the basic concept is still widely used all over the world.

From here on, runoff means implicitly direct runoff.

4.4.1 Derivation of Empirical Relationships

When the data of accumulated rainfall and runoff for long-duration, high-intensity rainfalls over small drainage basins are plotted, they show that runoff only starts after some rainfall has accumulated, and that the curves asymptotically approach a straight line with a 45-degree slope.

The Curve Number Method is based on these two phenomena. The initial accumulation of rainfall represents interception, depression storage, and infiltration before the start of runoff and is called initial abstraction. After runoff has started, some of the additional rainfall is lost, mainly in the form of infiltration; this is called actual retention. With increasing rainfall, the actual retention also increases up to some maximum value: the potential maximum retention.

To describe these curves mathematically, SCS assumed that the ratio of actual retention to potential maximum retention was equal to the ratio of actual runoff to potential maximum runoff, the latter being rainfall minus initial abstraction. In mathematical form, this empirical relationship is

$$\frac{F}{S} = \frac{Q}{P - I_a} \tag{4.1}$$

where

F = actual retention (mm)
S = potential maximum retention (mm)
Q = accumulated runoff depth (mm)
P = accumulated rainfall depth (mm)
I_a = initial abstraction (mm)

Figure 4.8 shows the above relationship for certain values of the initial abstraction and potential maximum retention. After runoff has started, all additional rainfall becomes either runoff or actual retention (i.e. the actual retention is the difference between rainfall minus initial abstraction and runoff).

$$F = P - I_a - Q \qquad (4.2)$$

Combining Equations 4.1 and 4.2 yields

$$Q = \frac{(P - I_a)^2}{P - I_a + S} \qquad (4.3)$$

To eliminate the need to estimate the two variables I_a and S in Equation 4.3, a regression analysis was made on the basis of recorded rainfall and runoff data from small drainage basins. The data showed a large amount of scatter (Soil Conservation Service 1972). The following average relationship was found

$$I_a = 0.2\,S \qquad (4.4)$$

Combining Equations 4.3 and 4.4 yields

$$Q = \frac{(P - 0.2\,S)^2}{P + 0.8\,S} \quad \text{for } P > 0.2\,S \qquad (4.5)$$

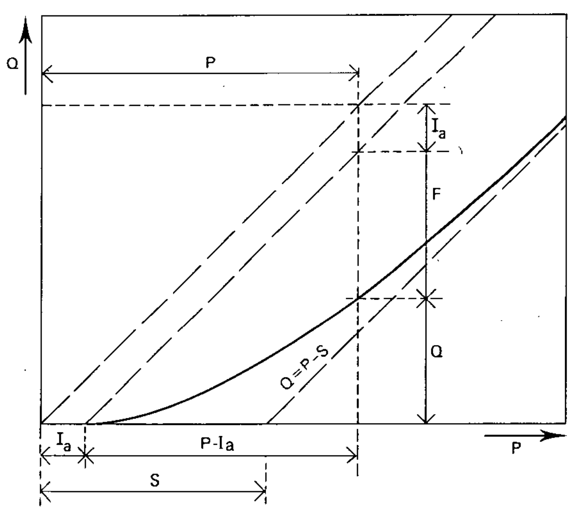

Figure 4.8 Accumulated runoff Q versus accumulated rainfall P according to the Curve Number Method

Equation 4.5 is the rainfall-runoff relationship used in the Curve Number Method. It allows the runoff depth to be estimated from rainfall depth, given the value of the potential maximum retention S. This potential maximum retention mainly represents infiltration occurring after runoff has started. This infiltration is controlled by the rate of infiltration at the soil surface, or by the rate of transmission in the soil profile, or by the water-storage capacity of the profile, whichever is the limiting factor.

The potential maximum retention S has been converted to the Curve Number CN in order to make the operations of interpolating, averaging, and weighting more nearly linear. This relationship is

$$CN = \frac{25400}{254 + S} \tag{4.6}$$

As the potential maximum retention S can theoretically vary between zero and infinity, Equation 4.6 shows that the Curve Number CN can range from one hundred to zero.

Figure 4.9 shows the graphical solution of Equation 4.5, indicating values of runoff depth Q as a function of rainfall depth P for selected values of Curve Numbers. For paved areas, for example, S will be zero and CN will be 100; all rainfall will become runoff. For highly permeable, flat-lying soils, S will go to infinity and CN will be zero; all rainfall will infiltrate and there will be no runoff. In drainage basins, the reality will be somewhere in between.

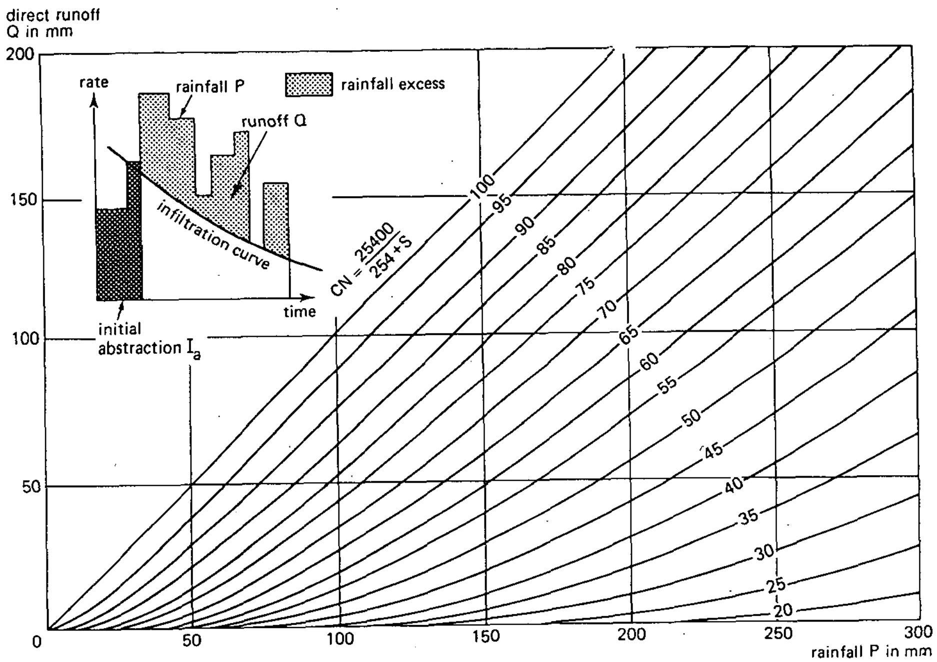

Figure 4.9 Graphical solution of Equation 4.5 showing runoff depth Q as a function of rainfall depth P and curve number CN (after Soil Conservation Service 1972)

Remarks

- The Curve Number Method was developed to be used with daily rainfall data measured with non-recording rain gauges. The relationship therefore excludes time as an explicit variable (i.e. rainfall intensity is not included in the estimate of runoff depth);
- In the Curve Number Method as presented by Soil Conservation Service (1964; 1972), the initial abstraction I_a was found to be 20% of the potential maximum retention S. This value represents an average because the data plots showed a large degree of scatter. Nevertheless, various authors (Aron et al. 1977, Fogel et al. 1980, and Springer et al. 1980) have reported that the initial abstraction is less than 20% of the potential maximum retention; percentages of 15, 10, and even lower have been reported.

4.4.2 Factors Determining the Curve Number Value

The Curve Number is a dimensionless parameter indicating the runoff response characteristic of a drainage basin. In the Curve Number Method, this parameter is related to land use, land treatment, hydrological condition, hydrological soil group, and antecedent soil moisture condition in the drainage basin.

Land Use or Cover

Land use represents the surface conditions in a drainage basin and is related to the degree of cover. In the SCS method, the following categories are distinguished:

- Fallow is the agricultural land use with the highest potential for runoff because the land is kept bare;
- Row crops are field crops planted in rows far enough apart that most of the soil surface is directly exposed to rainfall;
- Small grain is planted in rows close enough that the soil surface is not directly exposed to rainfall;
- Close-seeded legumes or rotational meadow are either planted in close rows or broadcasted. This kind of cover usually protects the soil throughout the year;
- Pasture range is native grassland used for grazing, whereas meadow is grassland protected from grazing and generally mown for hay;
- Woodlands are usually small isolated groves of trees being raised for farm use.

Treatment or Practice in relation to Hydrological Condition

Land treatment applies mainly to agricultural land uses; it includes mechanical practices such as contouring or terracing, and management practices such as rotation of crops, grazing control, or burning.

Rotations are planned sequences of crops (row crops, small grain, and close-seeded legumes or rotational meadow). Hydrologically, rotations range from poor to good. Poor rotations are generally one-crop land uses (monoculture) or combinations of row crops, small grains, and fallow. Good rotations generally contain close-seeded legumes or grass.

For grazing control and burning (pasture range and woodlands), the hydrological condition is classified as poor, fair, or good.

Pasture range is classified as poor when heavily grazed and less than half the area is covered; as fair when not heavily grazed and between one-half to three-quarters of the area is covered; and as good when lightly grazed and more than three-quarters of the area is covered.

Woodlands are classified as poor when heavily grazed or regularly burned; as fair when grazed but not burned; and as good when protected from grazing.

Hydrological Soil Group

Soil properties greatly influence the amount of runoff. In the SCS method, these properties are represented by a hydrological parameter: the minimum rate of infiltration obtained for a bare soil after prolonged wetting. The influence of both the soil's surface condition (infiltration rate) and its horizon (transmission rate) are thereby included. This parameter, which indicates a soil's runoff potential, is the qualitative basis of the classification of all soils into four groups. The Hydrological Soil Groups, as defined by the SCS soil scientists, are:

Group A: Soils having high infiltration rates even when thoroughly wetted and a high rate of water transmission. Examples are deep, well to excessively drained sands or gravels.

Group B: Soils having moderate infiltration rates when thoroughly wetted and a moderate rate of water transmission. Examples are moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures.

Group C: Soils having low infiltration rates when thoroughly wetted and a low rate of water transmission. Examples are soils with a layer that impedes the downward movement of water or soils of moderately fine to fine texture.

Group D: Soils having very low infiltration rates when thoroughly wetted and a very low rate of water transmission. Examples are clay soils with a high swelling potential, soils with a permanently high watertable, soils with a clay pan or clay layer at or near the surface, or shallow soils over nearly impervious material.

Antecedent Moisture Condition

The soil moisture condition in the drainage basin before runoff occurs is another important factor influencing the final CN value. In the Curve Number Method, the soil moisture condition is classified in three Antecedent Moisture Condition (AMC) Classes:

AMC I: The soils in the drainage basin are practically dry (i.e. the soil moisture content is at wilting point).

AMC II: Average condition.

AMC III: The soils in the drainage basins are practically saturated from antecedent rainfalls (i.e. the soil moisture content is at field capacity).

These classes are based on the 5-day antecedent rainfall (i.e. the accumulated total rainfall preceding the runoff under consideration). In the original SCS method, a distinction was made between the dormant and the growing season to allow for differences in evapotranspiration.

4.4.3 Estimating the Curve Number Value

To determine the appropriate CN value, various tables can be used. Firstly, there are tables relating the value of CN to land use or cover, to treatment or practice, to hydrological condition, and to hydrological soil group. Together, these four categories are called the Hydrological Soil-Cover Complex. The relationship between the CN value and the various Hydrological Soil-Cover Complexes is usually given for average conditions, i.e. Antecedent Soil Moisture Condition Class II. Secondly, there is a conversion table for the CN value when on the basis of 5-day antecedent rainfall data the Antecedent Moisture Condition should be classified as either Class I or Class III.

Hydrological Soil-Cover Complex
For American conditions, SCS related the value of CN to various Hydrological Soil-Cover Complexes. Table 4.2 shows this relationship for average conditions (i.e. Antecedent Moisture Condition Class II). In addition to Table 4.2, Soil Conservation Service (1972) prepared similar tables for Puerto Rico, California, and Hawaii. Rawls and Richardson (1983) prepared a table quantifying the effects of conservation tillage on the value of the Curve Number. Jackson and Rawls (1981) presented a table of Curve Numbers for a range of land-cover categories that could be identified from satellite images.

All the above-mentioned tables to determine Curve Numbers have in common that slope is not one of the parameters. The reason is that in the United States, cultivated land in general has slopes of less than 5%, and this range does not influence the Curve Number to any great extent. However, under East African conditions, for example, the slopes vary much more. Five classes to qualify the slope were therefore introduced (Sprenger 1978):

I	< 1%	Flat
II	1 – 5%	Slightly sloping
III	5 – 10%	Highly sloping
IV	10 – 20%	Steep
V	> 20%	Very steep

The category land use or cover was adjusted to East African conditions and combined with the hydrological condition. Table 4.3 shows the Curve Numbers for these Hydrological Soil-Cover Complexes.

With the aid of tables such as Tables 4.2 and 4.3 and some experience, one can estimate the Curve Number for a particular drainage basin. The procedure is as follows:
- Assign a hydrological soil group to each of the soil units found in the drainage basin and prepare a hydrological soil-group map;
- Make a classification of land use, treatment, and hydrological conditions in the drainage basin according to Table 4.2 or 4.3 and prepare a land-use map;
- Delineate the main soil-cover complexes by superimposing the land-use and the soil-group maps;
- Calculate the weighted average CN value according to the areas they represent.

Table 4.2 Curve Numbers for Hydrological Soil-Cover Complexes for Antecedent Moisture Condition Class II and $I_a = 0.2\,S$ (after Soil Conservation Service 1972)

Land use or cover	Treatment or practice	Hydrological condition	Hydrological soil group			
			A	B	C	D
Fallow	Straight row	Poor	77	86	91	94
Row crops	Straight row	Poor	72	81	88	91
	Straight row	Good	67	78	85	89
	Contoured	Poor	70	79	81	88
	Contoured	Good	65	75	82	86
	Contoured/terraced	Poor	66	74	80	82
	Contoured/terraced	Good	62	71	78	81
Small grain	Straight row	Poor	65	76	84	88
	Straight row	Good	63	75	83	87
	Contoured	Poor	63	74	82	85
	Contoured	Good	61	73	81	84
	Contoured/terraced	Poor	61	72	79	82
	Contoured/terraced	Good	59	70	78	81
Close-seeded legumes or rotational meadow	Straight row	Poor	66	77	85	89
	Straight row	Good	58	72	81	85
	Contoured	Poor	64	75	83	85
	Contoured	Good	55	69	78	83
	Contoured/terraced	Poor	63	73	80	83
	Contoured/terraced	Good	51	67	76	80
Pasture range		Poor	68	79	86	89
		Fair	49	69	79	84
		Good	39	61	74	80
	Contoured	Poor	47	67	81	88
	Contoured	Fair	25	59	75	83
	Contoured	Good	6	35	70	79
Meadow (permanent)		Good	30	58	71	78
Woodlands (farm woodlots)		Poor	45	66	77	83
		Fair	36	60	73	79
		Good	25	55	70	77
Farmsteads			59	74	82	86
Roads, dirt			72	82	87	89
Roads, hard-surface			74	84	90	92

Table 4.3 Curve Numbers for Hydrological Soil-Cover Complexes for Antecedent Moisture Condition Class II and $I_a = 0.2\,S$ (after Sprenger 1978)

Land use or cover	Slopes	Hydrological soil group			
		A	B	C	D
Rice fields or mangroves or swamps	I	0	0	3	5
	II	0	5	8	10
	III	5	10	13	15
	IV		non-existent		
	V		non-existent		
Pasture or range in good hydrological condition	I	33	55	68	74
	II	39	61	74	80
	III	42	64	77	83
	IV	44	66	79	85
	V	45	67	80	86
Woods in poor hydrological condition	I	39	60	71	77
	II	45	66	77	83
	III	49	70	81	87
	IV	52	73	84	90
	V	54	75	86	92
Pasture or range in poor hydrological condition	I	63	74	81	84
	II	68	79	86	89
	III	71	82	89	92
	IV	73	84	91	94
	V	74	85	92	95

Antecedent Moisture Condition Class

By using Tables 4.2 and 4.3, we obtain a weighted average CN value for a drainage basin with average conditions (i.e. Antecedent Moisture Condition Class II). To determine which AMC Class is the most appropriate for the drainage basin under consideration, we have to use the original rainfall records. The design rainfall that was selected in the frequency analysis usually lies between two historical rainfall events. The average of the 5-day total historical rainfall preceding those two events determines

Table 4.4 Seasonal rainfall limits for AMC classes (after Soil Conservation Service 1972)

Antecedent Moisture Condition Class	5-day antecedent rainfall (mm)		
	Dormant season	Growing season	Average
1	2	3	4
I	< 13	< 36	< 23
II	13 – 28	36 – 53	23 – 40
III	> 28	> 53	> 40

the AMC Class. Table 4.4 shows the corresponding rainfall limits for each of the three AMC Classes.

Columns 2 and 3 give the values as they are used under American conditions, specified for two seasons. Column 4 gives the values under East African conditions; they are the averages of the seasonal categories of Columns 2 and 3.

When, according to Table 4.4, the AMC Class is not Class II, the Curve Number as determined from Tables 4.2 or 4.3 should be adjusted according to Table 4.5.

Remarks

Used as antecedent precipitation index in the original Curve Number Method is the 5-day antecedent rainfall. In the literature, other periods have been reported to be more representative. Hope and Schulze (1982), for example, used a 15-day antecedent period in an application of the SCS procedure in the humid east of South Africa, and Schulze (1982) found a 30-day antecedent period to yield better simulations of direct runoff in humid areas of the U.S.A., but a 5-day period to be applicable in arid zones.

4.4.4 Estimating the Depth of the Direct Runoff

Once the final CN value has been determined, the direct runoff depth can be calculated.

Table 4.5 Conversion table for Curve Numbers (CN) from Antecedent Moisture Condition Class II to AMC Class I or Class III (after Soil Conservation Service 1972)

CN AMC II	CN AMC I	CN AMC III	CN AMC II	CN AMC I	CN AMC III
100	100	100	58	38	76
98	94	99	56	36	75
96	89	99	54	34	73
94	85	98	52	32	71
92	81	97	50	31	70
90	78	96	48	29	68
88	75	95	46	27	66
86	72	94	44	25	64
84	68	93	42	24	62
82	66	92	40	22	60
80	63	91	38	21	58
78	60	90	36	19	56
76	58	89	34	18	54
74	55	88	32	16	52
72	53	86	30	15	50
70	51	85	25	12	43
68	48	84	20	9	37
66	46	82	15	6	30
64	44	81	10	4	22
62	42	79	5	2	13
60	40	78	0	0	0

This can be done in two ways:
- Graphically, by using the design rainfall depth in Figure 4.9 and reading the intercept with the final CN value;
- Numerically, by using Equation 4.6 to determine the potential maximum retention S and substituting this S value and the design rainfall depth into Equation 4.5.

Flat Areas

In flat areas, the problem is to remove a certain depth of excess surface water within an economically determined period of time. Applying the Curve Number Method for different durations of design rainfall will yield corresponding depths of direct runoff. These values in fact represent layers of stagnant water which are the basis for determining the capacity of surface drainage systems. Example 4.1 shows such an application of the Curve Number Method.

Example 4.1

Suppose we have an ungauged drainage basin of flat rangeland. The soils have a low infiltration rate and a dense grass cover. As rainfall data, we shall use the intensity-duration-frequency curves shown in Figure 4.3. For this basin, we would like to know the depth of the direct runoff with a return period of 10 years for Antecedent Moisture Condition Class II.

First, we estimate the CN value for this basin. The land use is given as rangeland and the treatment practice is taken as contoured since the area is flat. Because of the dense grass cover, we select the hydrological condition 'good'. The infiltration rate of the soils is described as low and we therefore select the Hydrological Soil Group C. Using Table 4.2, we now find a CN value of 71 for AMC Class II. When we use Table 4.3, we have to define the slope category. Since we have contoured rangeland, we take slope category I. According to Table 4.3, the CN value is 68 for AMC Class II. So, a CN value of 70 seems a realistic estimate. Using Equation 4.6, we obtain for this value a potential maximum retention S of some 109 mm.

Next, we determine the appropriate rainfall data. From Figure 4.3, we can determine the depth of design rainfall as a function of its duration for the given return period of 10 years. This information is shown in Columns 1, 2, and 3 of Table 4.6.

We can now calculate the depth values of the direct runoff by substituting into Equation 4.5 the above S value and the rainfall depth data in Column 3 of Table 4.6. The data in Column 4 of Table 4.6 show the results of these calculations. These direct-runoff-depth data as a function of the duration of the design rainfall are the basis on which to determine the capacity of surface drainage systems in flat areas (as will be discussed in the Chapters 19 and 20).

Remarks

If we assume that the antecedent moisture condition in the drainage basin is not characterized as Class II but as Class III, the CN value of 70 should be adjusted according to Table 4.5. This yields an adjusted CN value of 85. The potential maximum retention S then changes to some 45 mm.

The data in Column 5 of Table 4.6 show the corresponding direct-runoff-depth data. From these data, it can be seen that changing the AMC Class from II to III

Table 4.6 Values of rainfall depth and corresponding direct runoff depth as a function of rainfall duration and AMC Class for a design return period of 10 years

Design rainfall			Direct runoff	
Duration (h)	Intensity (mm/h)	Depth (mm)	Depth (mm) AMC II	Depth (mm) AMC III
1	2	3	4	5
1	88	88	25	50
2	53	106	37	66
3	39	117	44	76
4	32	128	52	86
5	27	135	58	93
24	8.7	209	118	163
48	5.6	269	172	222
72	4.6	331	229	283

will result in direct-runoff-depth-data which are up to 100% greater. This illustrates the importance of selecting the appropriate AMC Class. The depth of direct runoff changes greatly when the CN value is adjusted to either AMC Class I or III. This is due to the discrete nature of the AMC Classes. Hawkins (1978) developed an alternative method to adjust the CN value on the basis of a simplified moisture-accounting procedure; the advantage of this method is that no sudden jumps in CN value are encountered.

Sloping Areas

In sloping areas, the problem is to accommodate the peak runoff rate at certain locations in the drainage basin. This peak runoff rate will determine the required cross-sections of main drainage canals, culverts, bridges, etc. Applying the Curve Number Method is now a first step in the calculation procedure. It gives only the depth of 'potential' direct runoff, but not how this direct runoff, following the topography and the natural drainage system, will produce peak runoff rates at certain locations. Example 4.2 shows an application of the Curve Number Method in such a situation.

Example 4.2

Suppose we have an ungauged drainage basin of highly sloping pasture land. The soils have a high infiltration rate and the hydrological condition can be characterized as poor because of heavy grazing. From Tables 4.2 and 4.3, we find a CN value of 68 and 71, respectively. So, again a CN value of 70 seems a realistic estimate.

Suppose we select from Table 4.6 a design rainfall with a duration of 3 hours. In the next section, it will be shown that, to apply the Unit Hydrograph Method, it is often necessary to split up the rainfall duration into a number of consecutive 'unit storm periods'. Suppose this unit storm period is calculated as 30 minutes. For each of these periods, the depths of direct runoff are then required for AMC Class II. The procedure to do this will now be explained.

Table 4.7 Values of rainfall depth and corresponding depth of direct runoff for a rainfall of 117 mm and a duration of 3 hours for a design return period of 10 years

Duration (h)	Rainfall (accumulated) (mm)	Direct runoff (accumulated) (mm)	Half-hour period	Direct runoff depth (mm)
1	2	3	4	5
0.0	0.0	0.0		
0.5	19.5	0.0	1	0.0
1.0	39.0	2.4	2	2.4
1.5	58.5	9.3	3	6.9
2.0	78.0	19.2	4	9.9
2.5	97.5	31.1	5	11.9
3.0	117.0	44.4	6	13.3

If no information is available on how the amount of design rainfall (117 mm) is distributed over the 3-hour period, the usual assumption is that the intensity will be uniformly distributed. This gives a rainfall intensity of 39 mm/h. Columns 1 and 2 of Table 4.7 give the accumulated rainfall amounts for 6 consecutive half-hour periods.

We can now calculate the depth values of direct runoff by substituting into Equation 4.5 the S value of 109 mm and the rainfall-depth data in Column 2 of Table 4.7. The data in Column 3 of Table 4.7 represent the accumulated direct-runoff-depth data. The direct runoff depth per half hour period can now be calculated (Columns 4 and 5 in Table 4.7).

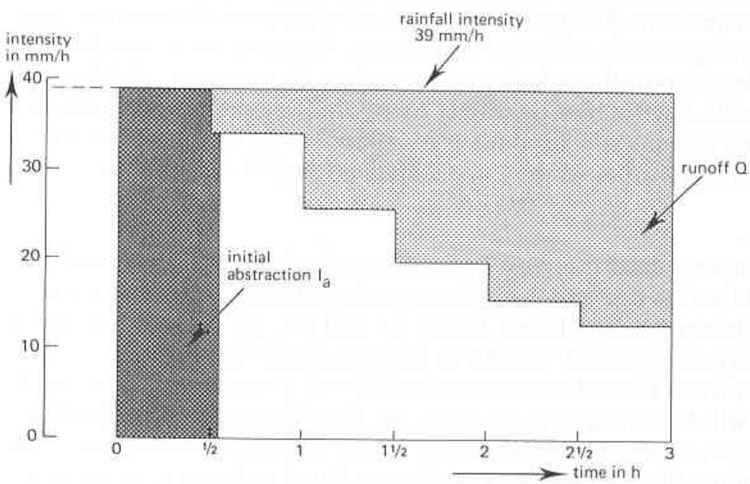

Figure 4.10 Graphical representation of the design rainfall and corresponding runoff for a selected duration of 3 hours

Figure 4.10 shows these results graphically by presenting the values of rainfall and runoff as intensities instead of depths. It can be seen from Figure 4.10 that the duration of direct runoff is shorter than the rainfall duration; the lower the CN value, the shorter the direct runoff duration will be with respect to the rainfall duration. Figure 4.10 can be compared with the inset of Figure 4.9; both were constructed in an identical manner, but the inset shows a historical rainfall with varying intensities within its duration.

So, by applying the above procedure, we can specify the direct runoff for a succession of arbitrarily chosen periods within the selected duration of the design rainfall. These data are the basis on which to determine the peak runoff rate in sloping areas, as will be discussed in the next sections.

4.5 Estimating the Time Distribution of the Direct Runoff Rate

To estimate the time distribution of the direct runoff rate at a specific location in the drainage basin, we apply the Unit Hydrograph Method. For drainage basins where no runoff has been measured, the Method is based on a parametric unit hydrograph shape.

The concept of the unit hydrograph has been the subject of many papers. Unit hydrograph procedures have been developed, from graphical representations such as those presented by Sherman (1932), to generalized mathematical expressions. In the following, we shall explain the Unit Hydrograph Method on the basis of Sherman's approach.

The direct runoff discussed in the previous section as representing a depth uniformly distributed over the drainage basin is renamed 'excess rainfall' to differentiate it from the direct runoff rate that will pass a certain point in the drainage basin, which is the subject of this section.

4.5.1 Unit Hydrograph Theory

Since the physical characteristics of a basin (shape, size, slope, etc.) remain relatively constant, one can expect considerable similarity in the shape of hydrographs resulting from similar high-intensity rainfalls. This is the essence of the Sherman theory.

Sherman first introduced the unit hydrograph as the hydrograph of direct runoff resulting from 1 mm of excess rainfall generated uniformly over the basin area at a uniform rate. By comparing unit hydrographs of drainage basins with similar physical characteristics, he found that the shape of these unit hydrographs was still not similar due to differences in the duration of the excess rainfall of 1 mm.

Sherman next introduced a specified period of time for the excess rainfall and called it the 'unit storm period'. He found that for every drainage basin there is a certain unit storm period for which the shape of the hydrograph is not significantly affected by changes in the time distribution of the excess rainfall over this unit storm period.

This means that equal depths of excess rainfall with different time-intensity patterns produce hydrographs of direct runoff which are the same when the duration of this excess rainfall is equal to or shorter than the unit storm period. So, assuming a uniformly distributed time-intensity for the excess rainfall will not affect the shape of the hydrograph of direct runoff. This implies that any time-intensity pattern of excess rainfall can be represented by a succession of unit storm periods, each of which has a uniform intensity.

This unit storm period varies with characteristics of the drainage basin; in general, it can be taken as one-fourth of the time to peak (i.e. from the beginning to the peak of the hydrograph of direct runoff).

Sherman, after analyzing a great number of time-intensity graphs (hyetographs) of excess rainfall with a duration equal to or smaller than the unit storm period, concluded that the resulting hydrographs for a particular drainage basin closely fit the following properties:

- The base length of the hydrograph of direct runoff is essentially constant, regardless of the total depth of excess rainfall;
- If two high-intensity rainfalls produce different depths of excess rainfall, the rates of direct runoff at corresponding times after the beginning of each rainfall are in the same proportion to each other as the total depths of excess rainfall;
- The time distribution of direct runoff from a given excess rainfall is independent of concurrent runoff from antecedent periods of excess rainfall.

The principle involved in the first and second of these statements is known as the principle of proportionality, by which the ordinates of the hydrograph of direct runoff are proportional to the depth of excess rainfall. The third statement implies that the hydrograph of direct runoff from a drainage basin due to a given pattern of excess rainfall at whatever time it may occur, is invariable. This is known as the principle of time invariance.

These fundamental principles of proportionality and time invariance make the unit hydrograph an extremely flexible tool for developing composite hydrographs. The total hydrograph of direct runoff resulting from any pattern of excess rainfall can be built up by superimposing the unit hydrographs resulting from the separate depths of excess rainfall occurring in successive unit time periods. In this way, a unit hydrograph for a relatively short duration of excess rainfall can be used to develop composite hydrographs for high-intensity rainfalls of longer duration. Figure 4.11 shows the above principles graphically.

Suppose that the excess rainfall period can be schematized by three successive unit storm periods with, respectively, 1, 3, and 1.5 mm excess rainfall. Applying the principles of proportionality and time invariance results in three separate hydrographs for each of the amounts of excess rainfall in the individual unit storm periods, as follows:

- The first hydrograph is identical to the unit hydrograph, because the depth of excess rainfall during this period is 1 mm;
- The second hydrograph has ordinates that are three times as high as those of the unit hydrograph and starts one unit storm period later than the first hydrograph;
- The third hydrograph has ordinates that are one-and-a-half times as high as those of the unit hydrograph and starts two unit storm periods later than the first hydrograph.

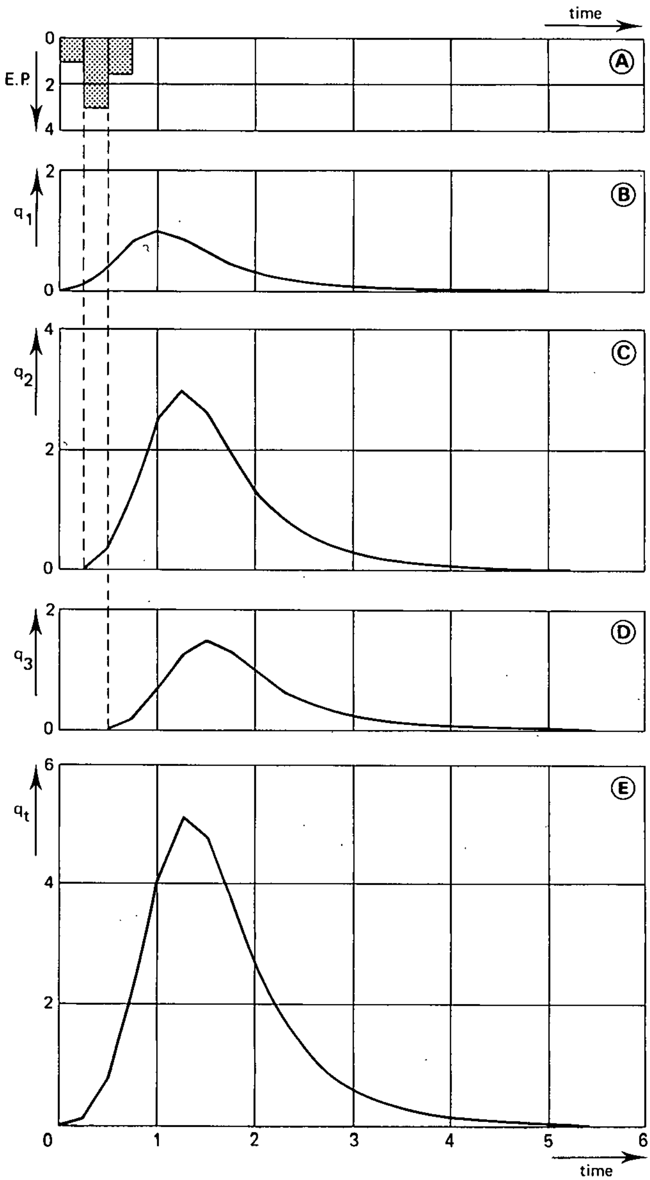

Figure 4.11 Graphical representation of the principles of proportionality, time invariance, and superposition: (A) Time intensity pattern excess rainfall (EP); (B) Hydrograph of runoff due to first unit storm period; (C) Hydrograph of runoff due to second unit storm period; (D) Hydrograph of runoff due to third unit storm period; and (E) Composite hydrograph of runoff due to the succession of the three unit storm periods

Applying the principle of superposition results in one composite hydrograph of direct runoff for the total excess rainfall period of three successive unit storm periods. Graphically, this is done by adding the ordinates of the three separate hydrographs at corresponding times.

So, if we know the shape of the unit hydrograph, we can convert any historical or statistical rainfall into a composite hydrograph of direct runoff by using the Curve Number Method to calculate the excess rainfall depths and the Unit Hydrograph Method to calculate the direct runoff rates as a function of time.

4.5.2 Parametric Unit Hydrograph

Numerous procedures to construct a unit hydrograph for ungauged basins have been developed. In general, these procedures relate physical characteristics (parameters) of a drainage basin to geometric aspects of the unit hydrograph. Most attempts to derive these relationships were aimed at determining time to peak, peak flow, and base length of the unit hydrograph. Here, we present only one of these procedures.

The dimensionless unit hydrograph used by the Soil Conservation Service (1972) was developed by Mockus (1957). It was derived from a large number of natural unit hydrographs from drainage basins varying widely in size and geographical locations. The shape of this dimensionless unit hydrograph predetermines the time distribution of the runoff; time is expressed in units of time to peak T_p, and runoff rates are expressed in units of peak runoff rate q_p. Table 4.8 shows these time and runoff ratios numerically and Figure 4.12 (solid line) shows them graphically.

To change this dimensionless unit hydrograph into a dimensional unit hydrograph, we have to know both the time to peak T_p and the peak runoff rate q_p of the basin. To reduce this two-parameter problem to a one-parameter problem, Mockus (1957) used an equivalent triangular unit hydrograph with the same units of time and runoff as the curvilinear unit hydrograph. Figure 4.12 shows these two hydrographs; both have in common that they have identical peak runoff rates and times to peak. Since the area under the rising limb of the curvilinear unit hydrograph represents 37.5 per cent of the total area, the time base T_b of the triangular unit hydrograph equals 1/0.375 = 2.67 in order to have also the same total areas under both hydrographs, representing 1 mm of excess rainfall.

Using the equation of the area of a triangle and expressing the volumes in m^3, we obtain for the dimensional triangular unit hydrograph

$$10^6 A \times 10^{-3} Q = 1/2\,(3600 \times q_p) \times 2.67\, T_p \tag{4.7}$$

Table 4.8 Dimensionless time and runoff ratios of the SCS parametric unit hydrograph (after Soil Conservation Service 1972)

t/T_p	q_t/q_p	t/T_p	q_t/q_p	t/T_p	q_t/q_p
0	0	1.75	0.45	3.50	0.036
0.25	0.12	2.00	0.32	3.75	0.026
0.50	0.43	2.25	0.22	4.00	0.018
0.75	0.83	2.50	0.15	4.25	0.012
1.00	1.00	2.75	0.105	4.50	0.009
1.25	0.88	3.00	0.075	4.75	0.006
1.50	0.66	3.25	0.053	5.00	0.004

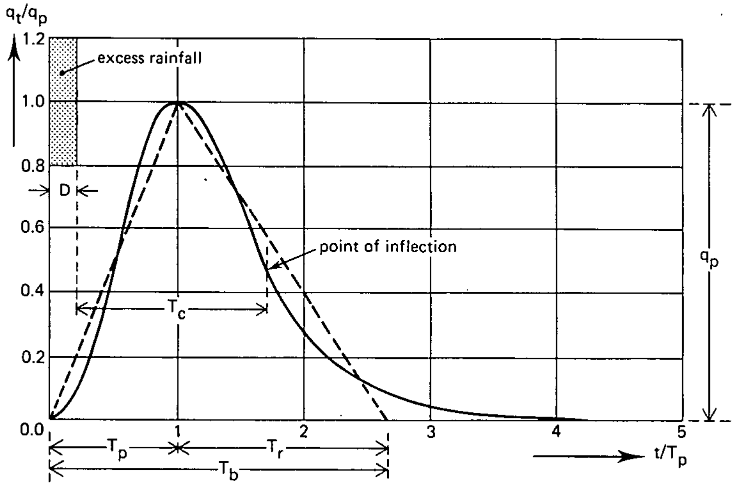

Figure 4.12 Dimensionless curvelinear unit hydrographs (solid line) and equivalent triangular unit hydrograph (dashed line) (after Soil Conservation Service 1972)

where

A = area of drainage basin (km^2)
Q = excess rainfall (mm)
q_p = peak runoff rate unit hydrograph (m^3/s)
T_p = time to peak runoff unit hydrograph (h)

Rearranging Equation 4.7 and making q_p explicit yields

$$q_p = 0.208 \frac{A\,Q}{T_p} \tag{4.8}$$

In Equation 4.8, the only unknown parameter is time to peak T_p. This can be estimated in terms of time of concentration T_c.

The time of concentration is defined as the time for runoff to travel from the hydraulically most distant point in the drainage basin to the outlet or point of interest; it is also defined as the distance between the end of excess rainfall and the inflection point in the recession limb of the dimensionless curvilinear unit hydrograph. Figure 4.12 shows that the inflection point lies at a distance of approximately 1.7 times T_p. Taking the duration of the excess rainfall equal to 0.25 times T_p (unit storm period) gives the following relationship

$$T_p = 0.7\,T_c \tag{4.9}$$

For small drainage basins of less than 15 km^2, the time to peak is regarded as being equal to the time of concentration. This relationship is based on another empirical method, the Rational Method (Chow 1964).

Quite a number of formulas exist for deriving T_c from the physical characteristics of a drainage basin. One of these empirical formulas is given by Kirpich (1940)

$$T_c = 0.02\,L^{0.77}\,S^{-0.385} \tag{4.10}$$

where

T_c = time of concentration (min)
L = maximum length of travel (m)
S = slope, equal to H/L where H is the difference in elevation between the most remote point in the basin and the outlet

The parameters to estimate the time of concentration can be derived from a topographic map. So, by estimating T_c, we can calculate the time to peak T_p and consequently the peak runoff rate q_p. Thus, a dimensional unit hydrograph for a particular basin can be derived from the dimensionless curvilinear unit hydrograph. Example 4.3 shows the calculation procedure.

Example 4.3
Suppose a drainage basin has the shape of a pear. The maximum length of travel in it is about 7600 m and the elevation difference is 25 m. Its area is 2590 ha. For this basin, we would like to know the unit hydrograph.

First, we calculate the time of concentration. Substituting L = 7600 m and H = 25 m into Equation 4.10 gives

$$T_c = 0.02\,(7600)^{0.77}\,(25/7600)^{-0.385} = 176\text{ min} = 2.9\text{ h}$$

Substituting this value of T_c into Equation 4.9 gives

$$T_p = 0.7 \times 2.9 = 2.0\text{ h}$$

Substituting A = 25.9 km^2, Q = 1 mm, and T_p = 2.0 h into Equation 4.8 gives

$$q_p = 0.208\,\frac{25.9 \times 1}{2.0} = 2.7\text{ m}^3/\text{s}$$

So the peak runoff rate is 2.7 m^3/s for an excess rainfall of 1 mm.

Next, we convert the SCS dimensionless curvelinear unit hydrograph into a dimensional unit hydrograph for this basin. Substituting the above values of T_p and q_p into Table 4.8 gives the runoff rates of this unit hydrograph. Table 4.9 shows these rates.

Table 4.9 shows that the unit hydrograph for this drainage basin has a time base of

Table 4.9 Dimensional time and runoff of the unit hydrograph

t (h)	q_t (m^3/s)	t (h)	q_t (m^3/s)	t (h)	q_t (m^3/s)
0	0	3.5	1.22	7.0	0.10
0.5	0.32	4.0	0.86	7.5	0.07
1.0	1.16	4.5	0.59	8.0	0.05
1.5	2.24	5.0	0.41	8.5	0.03
2.0	2.7	5.5	0.28	9.0	0.02
2.5	2.38	6.0	0.20	9.5	0.02
3.0	1.78	6.5	0.14	10.0	0.01

approximately 10 hours, a time to peak of 2 hours, and a peak runoff rate of 2.7 m^3/s.

4.5.3 Estimating Peak Runoff Rates

To obtain the hydrograph of direct runoff for a design storm, we can use the SCS dimensionless unit hydrograph in the same way as the unit hydrograph of Sherman (by the principle of superposition). Example 4.4 explains the calculation procedure.

Example 4.4
In this example, we want to know the peak runoff rate for a design rainfall with a return period of 10 years and a duration of 3 hours. We shall use the information obtained in the previous three examples.

In Example 4.3, we found the unit hydrograph for that basin by using the dimensionless curvelinear unit hydrograph. Since its time to peak is 2 hours, the unit storm period of the excess rainfall should be equal to or less than one-fourth of the time to peak. Suppose we make it equal to half an hour. We then split up the design rainfall duration of 3 hours into six consecutive unit storm periods.

In Example 4.2, we already calculated the depth of direct runoff (= excess rainfall) for each of the six half-hour periods. So we can use the data directly.

By applying the principles of Sherman's Unit Hydrograph Method, we can now calculate the composite hydrograph of direct runoff for the time-intensity pattern of excess rainfall shown in Figure 4.10. This procedure is shown numerically in Table 4.10. The composite hydrograph is plotted in Figure 4.13. As can be seen, the peak runoff rate is approximately 101 m^3/s and will occur 4 hours after the start of the design rainfall.

It should be noted that the relationships formulated for the unit hydrograph are not applicable for the composite hydrograph of direct runoff. Its time to peak will always be greater than the time to peak of the unit hydrograph. Another feature is that the total duration of excess rainfall that produces the composite hydrograph of direct runoff will always be greater than one-fourth of its time to peak.

In Example 4.1, we selected from the depth-intensity curves a design rainfall with a return period of 10 years. The total amount of this design rainfall is related to its duration as was shown in Table 4.6. This implies that the above calculation procedures should be repeated for various durations. Table 4.11 shows the results of these calculations. Only one combination of duration and amount of design rainfall will give the highest peak runoff rate for the basin.

Table 4.11 shows that the peak runoff rates increase with increasing duration of the design rainfall, up to a duration of 4 hours; this duration produces the highest peak runoff rate. For durations longer than 4 hours, the peak runoff rate will start to decrease and will continue to decrease. This phenomenon of first increasing peak runoff rates reaching a highest peak runoff rate followed by decreasing peak runoff rates will occur in all basins, but the duration that will produce the highest peak runoff rate cannot be determined beforehand. This implies that the above calculation procedure should be repeated for design rainfalls of increasing duration. Once the peak runoff rates start to decrease, one can stop the calculations.

Table 4.10 Contribution of individual hydrographs for the six consecutive unit storm periods of half an hour, yielding the total composite hydrograph of direct runoff

Unit storm period		1	2	3	4	5	6	
Excess rainfall (mm)		0	2.4	6.9	9.9	11.9	13.3	
Time	Unit	Hydrographs of unit storm period						Composite
(h)	hydrograph (m^3/s)	1	2	3	4	5	6	hydrograph (m^3/s)
0	0	0						0
0.5	0.32	0	0					0
1.0	1.16	0	0.8	0				1
1.5	2.24	0	2.8	2.2	0			5
2.0	2.70	0	5.4	8.0	3.2	0		17
2.5	2.38	0	6.5	15.5	11.5	3.8	0	37
3.0	1.78	0	5.7	18.6	22.2	13.8	4.3	65
3.5	1.22	0	4.3	16.4	26.7	26.7	15.4	90
4.0	0.86	0	2.9	12.3	23.6	32.1	29.8	101
4.5	0.59	0	2.1	8.4	17.6	28.3	35.9	92
5.0	0.41	0	1.4	5.9	12.1	21.2	31.7	72
5.5	0.28	0	1.0	4.1	8.5	14.5	23.7	52
6.0	0.20	0	0.7	2.8	5.8	10.2	16.2	36
6.5	0.14	0	0.5	1.9	4.1	7.0	11.4	25
7.0	0.10	0	0.3	1.4	2.8	4.9	7.8	17
7.5	0.07	0	0.2	1.0	2.0	3.3	5.5	12
8.0	0.05	0	0.2	0.7	1.4	2.4	3.7	8
8.5	0.03	0	0.1	0.5	1.0	1.7	2.7	6
9.0	0.02	0	0.1	0.3	0.7	1.2	1.9	4
9.5	0.02	0	0.0	0.2	0.5	0.8	1.3	3
10.0	0.01	0	0.0	0.1	0.3	0.6	0.9	2
10.5			0.0	0.1	0.2	0.4	0.7	1
11.0				0.1	0.2	0.2	0.4	1
11.5					0.1	0.2	0.3	1
12.0						0.1	0.3	0
12.5							0.1	0

Table 4.11 Peak runoff rates of the composite hydrograph of direct runoff for different durations of the design rainfall with a return period of 10 years

Design rainfall		Peak runoff rate
Duration (h)	Depth (mm)	(m^3/s)
1	88	66
2	106	93
3	117	101
4	128	108
5	135	106
24	209	53

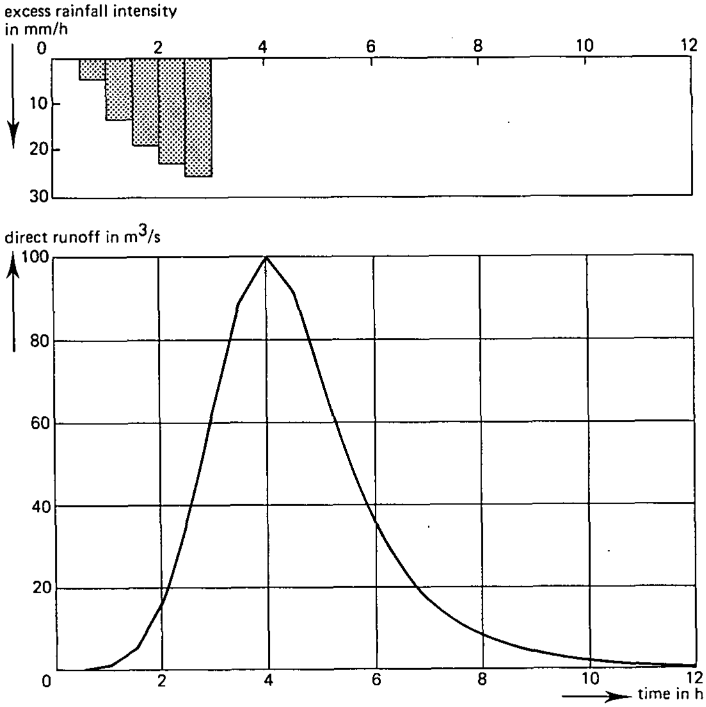

Figure 4.13 Time-intensity pattern of excess rainfall and corresponding composite hydrograph of direct runoff for a return period of 10 years

4.6 Summary of the Calculation Procedure

The calculation procedure discussed in the previous sections is based on the situation where no runoff records are available and a design peak runoff rate has to be estimated from rainfall-runoff relations. This calculation procedure can be summarized in the following steps:

1 Select a design frequency. The process of selecting such a frequency (or return period) is not discussed in this chapter; it involves a decision that is fundamental to the designer's intention and to the criteria for the satisfactory performance and safety of the works under consideration. In drainage works, the design return period usually ranges from 5 to 25 years.

2 From depth-duration-frequency curves or intensity-duration-frequency curves available for rainfall data and representative for the drainage basin under consideration, select the curve with the frequency that corresponds to the design return period selected in Step 1.

3 From the curve selected in Step 2, read the total depths or intensities of rainfall for various durations. Convert intensity data, if available, to depth data. Steps 2 and 3 are illustrated in Example 4.1 of Section 4.4.4. Select one duration with a corresponding total depth of rainfall; this is called the design rainfall.

4 Calculate the time to peak of the unit hydrograph for the drainage basin under

consideration, using empirical relationships as were formulated in Equations 4.9 and 4.10. Step 4 is illustrated in Example 4.3 of Section 4.5.2.

5 Split up the duration of design rainfall as selected in Step 3 into a number of consecutive unit storm periods. This unit storm period should be equal to or less than one-fourth of the time to peak as calculated in Step 4.

6 Determine the Curve Number value for the drainage basin under consideration, using Tables 4.2 and/or 4.3. Adjust this CN value, if necessary according to AMC Class I or III, using Tables 4.4 and 4.5. Step 6 is illustrated in Example 4.1 of Section 4.4.4.

7 Calculate the depths of excess rainfall (= direct runoff), using the Curve Number Method for design rainfall depths of accumulated unit storm periods as determined in Step 5 and the CN value as determined in Step 6. For each of the successive unit storm periods, calculate the contribution of excess rainfall depth. Steps 5, 6, and 7 are illustrated in Example 4.2 of Section 4.4.4.

8 Calculate the peak runoff rate of the unit hydrograph for the drainage basin under consideration, using the empirical relationship as was formulated in Equation 4.8.

9 Calculate the ordinates of the dimensional unit hydrograph, using the dimensionless ratios as given in Table 4.8, and time to peak and peak runoff rate values as calculated in Steps 4 and 8, respectively. Steps 8 and 9 are illustrated in Example 4.3 of Section 4.5.2.

10 Calculate the ordinates of the individual hydrographs of direct runoff for each of the unit storm periods, using the ordinates of the unit hydrograph as calculated in Step 9 and the corresponding excess rainfall depths as calculated in Step 7.

11 Calculate the ordinates of the total composite hydrograph of direct runoff by adding the ordinates of the individual hydrographs of direct runoff as calculated in Step 10. The ordinates of these individual hydrographs are lagged in time one unit storm period with respect to each other.

12 Determine the highest value from the ordinates of the total composite hydrograph as calculated in Step 11. This represents the peak runoff rate for a design rainfall with a duration as selected in Step 3.

13 Select durations of design rainfall different from the initial one as selected in Step 3 and read the corresponding total depths of rainfall as determined in Step 3. Repeat Steps 4 to 12. This will yield a set of peak runoff rates. The highest value represents the design peak runoff rate for the drainage basin under consideration. Steps 10 to 13 are illustrated in Example 4.4 of Section 4.5.3.

Remark

The contribution of groundwater runoff is not included in this procedure to estimate the design peak runoff rate. Because the calculation procedure is based on the assumption that no runoff has been measured, this groundwater runoff cannot be determined.

4.7 Concluding Remarks

The availability of depth-duration-frequency or intensity-duration frequency curves as mentioned in Step 2 of the calculation procedure is essential for small drainage basins. High-intensity rainfalls of short duration (i.e. a few hours) will then produce the highest peak runoff rates. For drainage basins of less than 1300 km^2, hourly rainfall data are required. It should be noted that this maximum size should be treated as an indication, not as an absolute value.

The above also implies that applying the calculation procedure only on the basis of daily rainfall frequency data will consistently underestimate the peak runoff rate, unless the size of the drainage basin is large. Large in this respect means at least 2500 km^2.

The reliability of the estimate of the design peak runoff rate depends largely on a proper estimate of the final CN value and the time to peak of the dimensional unit hydrograph.

With regard to the CN value, it can be stated that both its determination from the characteristics of a drainage basin and the selection of the proper Antecedent Moisture Condition Class are crucial. Errors in the latter can result in peak runoff rates up to 100% in error.

With regard to the time to peak of the dimensional unit hydrograph, it can be stated that it is derived from the time of concentration. Because the use of different formulas for deriving the time of concentration results in a wide range of values, and because the relationship between time of concentration and time to peak also varies, the design peak runoff rate with respect to an incorrect value of the time to peak of the unit hydrograph can be more than 100% in error.

The calculation procedure presented will therefore gain substantially in reliability when the above two parameters can be determined from field observations. One should therefore measure at least one, but preferably more flood hydrographs with concurrent rainfall in the drainage basin.

The procedure to determine the CN value for each observed flood hydrograph can be summarized as follows. By hydrograph separation, the area under the thus derived hydrograph of direct runoff can be calculated. This area represents the volume of direct runoff and can be converted to a depth value by dividing it by the area of the drainage basin. Substituting this latter value and the observed concurrent rainfall into the Curve Number equation will yield the potential maximum retention and finally the corresponding Curve Number.

The procedure to determine the time to peak of the unit hydrograph and, with that, its actual shape involves an inverse application of the unit hydrograph theory. Anyone wanting more information on this subject is referred to the literature (Chow et al 1988).

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