2 RAINFALL-RUNOFF ANALYSIS

2.1 Runoff analysis

Discharge is generally determined on the basis of water level recordings in combination with a stage discharge relation curve, called a rating curve. A unique relationship between water level and river discharge is usually obtained in a stretch of the river where the riverbed is stable and the flow is slow and uniform, i.e. the velocity pattern does not change in the direction of flow. Another suitable place is at a tranquil pool, just upstream of some rapids. Such a situation may also be created artificially in a stretch of the river (e.g. with non-uniform flow) by building a control structure (threshold) across the riverbed. The rating curve established at the gauging station has to be updated regularly, because scour and sedimentation of the riverbed and riverbanks may change the stage discharge relation, particularly after a flood.

The rating curve can often be represented adequately by an equation of the form:

$$Q = a\left(H - H_0\right)^b \qquad \textit{Equation 2.1}$$

where Q is the discharge in m^3/s, H is the water level in the river in m, H_0 is the water level at zero flow, and a and b are constants. The value of H_0 is determined by trial and error. The values of a and b are found by a least square fit using the measured data, or by a plot on logarithmic paper and the fit of a straight line (see Figure 2.1).

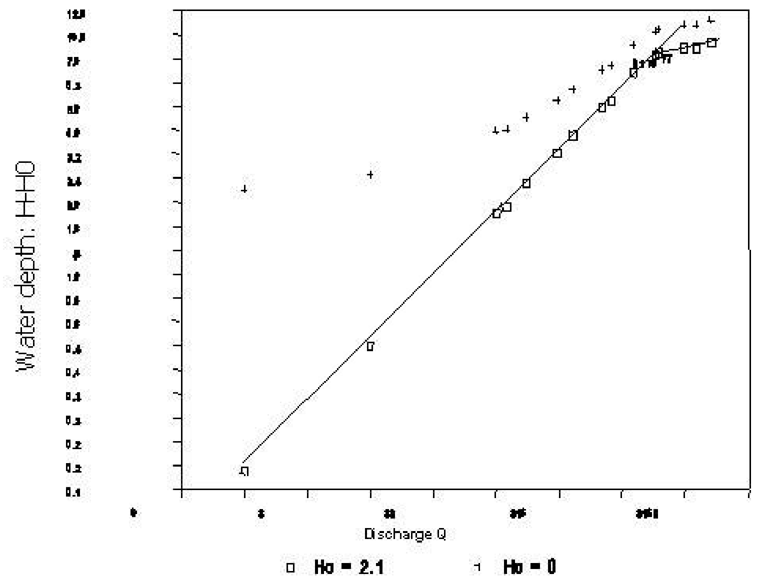

Figure 2.1: Rating curve in Limpopo river at Sicacate

Figure 2.1 shows the rating curve of the Limpopo River at Sicacate; the value of b equals 1.90. The Limpopo is an intermittent river which falls dry in the dry season and can have very high flash floods during the flood season. The station of Sicacate has a value of H_0 equal to 2.1 m. In Figure 2.1 a clear flood branch can be distinguished which is based on peak flows recorded during the floods of 1981, 1977 and 1978 in the Limpopo river. The gradient of a flood branch becomes flat as the

river enters the flood plain; a small increase in water level then results in a large increase in discharge.
To illustrate the trial and error procedure in determining the value of H_0, a plot of data with H_0=0 has been added. It can be seen that the value of H_0 particularly affects the determination of low flow.

For methods to determine water levels and flows one should refer to the lectures on Hydrometry. By using a rating curve, a time series of water levels can be transformed into runoff series.

Occurrence of floods

To the engineer, extreme floods are often the critical situation for design. Consequently, monitoring of the processes involved in the occurrence of an extreme flood (rainfall, water levels, flows) is important. However, extreme floods only occur once in a lifetime, and one is seldom adequately prepared to monitor the event effectively.

Applying Murphy's Law to the occurrence of extreme floods, one could state that extreme floods occur:

- at night, when everybody is sound asleep;
- on public holidays when all offices are closed;
- after torrential rains when telephone lines are broken and radios do not work as a result of static;
- when roads are blocked by flooding and culverts have been washed out;
- when the car is being repaired, or without petrol;
- when the Director of Water Affairs is on holiday.

This implies that although an observation network may work perfectly under normal flow condition, the critical observations of extreme rainfall, peak water levels and peak discharges are generally not recorded. Here follows a short list of problems which, unfortunately, are the rule rather than the exception during a critical flood:

- the reservoir of the rain gauge overtopped; it was raining so hard that the observer was reluctant to go and empty the reservoir;
- the rain gauge was washed away by the flood;
- the pen of the recorder had no ink;
- the clockwork of the recorder had stopped;
- the housing of the water level recorder was submerged by the flood; the instrument was lost;
- the rating weir was completely destroyed by the flood;
- the bridge on which the recorder was installed was blocked by debris, overtopped and the instrument destroyed;
- while trying to measure the velocity, the current meter was caught by debris and lost.

One can therefore conclude that the routine observation network generally fails during extreme floods. Therefore attention should be paid to special flood surveys.

2.2 Flood surveys

A number of flood survey methods are presented to deal with extreme flood situations. In particular:

- discharge measurement using floats
- flood mark survey
- slope area method
- simplified slope area method

The first method is used during the flood; the latter three methods are "morning after" methods.

Floats

Contrary to what most hydrologists and hydrometrists wish to believe, floats are the most reliable and scientifically most appropriate instruments for measuring discharges during peak flows. The hydrometrist often considers floats below his professional standard and thinks incorrectly that his current meter is the most accurate instrument for determining peak discharges. Floats, well positioned, and with a resistance body at the right depth (see Figure 2.2) are best for the following reasons:

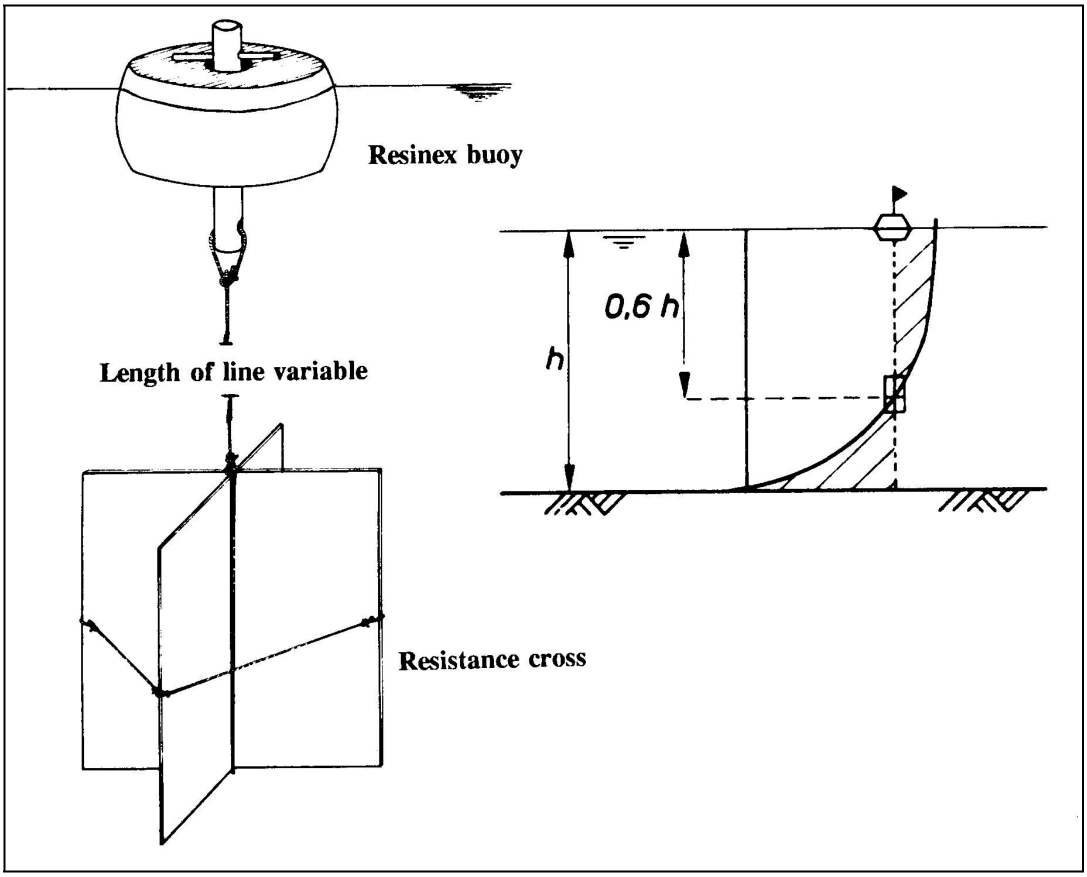

Figure 2.2: Float with resistance body, and location of the resistance body in the vertical

- floats move at the same velocity as the surrounding water (provided they are made as in Figure 2.2) and integrate the velocity in the longitudinal direction; they thus provide an accurate sample of the real mean velocity; current meters that integrate the velocity over time at a fixed position may be affected by local accelerations (e.g. due to bed forms); moreover, current meters do not always measure the point velocity accurately);
- floats are stagnant in relation to the moving water, thus the vertical position of the resistance body in the flow is correct; the vertical position of a current meter in the stream, on the other hand, is not certain; often velocities are so high that the instrument just "takes off" after touching the water surface.
- a float measurement can be carried out in a shorter period of time than a current meter measurement, which is an advantage under rapidly changing conditions;
- floats are cheap compared to current meters; it is not a disaster if one gets lost;
- at the peak of the flood, the river is full of debris; use of a current meter then is completely impossible.
- if no professional floats are available, it is easy to improvise;

If one arrives at the site unprepared, it is always possible to clock the velocity of floating debris. The larger the debris, for example trees, the better they describe the mean velocity in the vertical. In the following intermezzo, a float measurement is briefly described.

- *A straight stretch is selected of 100 m length (see* Figure 2.3*). The width is divided into approximately eleven equal distances in which ten measuring points are established. If there are constraints in time or resources, try as many points as possible. At each measuring point a float is used with a resistance body at 60% of the average depth at that point.*
- *Two measuring sections at the upstream and downstream end of the measuring reach should be marked by beacons placed on both banks in a line perpendicular to the flow. On each bank of a measuring section the two beacons should stand with sufficient distance between them to allow the observer to determine his position from a boat (if a boat is used). At night, the floats and the stacks should carry lights.*
- *A float measurement is carried out by launching a float at a particular point in the cross-section. The positioning may be done from a cable mounted across the river, or from markings on a bridge (if present) or if necessary by sextant. The float should be launched (from a bridge or from a boat) at least 10 m upstream from the cross-section where the measurement starts, so as to allow the float to adjust itself to the flow velocity. When a boat is used, the observer should stay with the float, keeping next to it. When the float enters the measuring section, the observer starts the stopwatch; he then follows the float until the measuring cross-section 100 m downstream where he stops the stopwatch, notes down the time and (if using a boat) recovers the float to return and repeat the measurement at the next observation point.*
- *The advantage of a boat moving with the float is that only one observer is needed per float and no communication problems occur. The disadvantage is that the determination of the moment in which the float passes the section is less accurate.*
- *If no boat is available, floats should be launched from a bridge, and followed along the bank. More observers could work at the same time, and one observer could clock more than one float. The advantage of this method is a greater degree of accuracy when starting and stopping the stopwatch; the disadvantage, however, is the more complicated communication system required and that the floats are lost.*

- *A very elegant alternative is to use a float attached to a string with two knots 50 m apart (the distance can be less than in the above method since the accuracy is greater). The time which elapses between the passages of the knots can be measured. The float can only be recuperated if the amount and size of debris is small.*

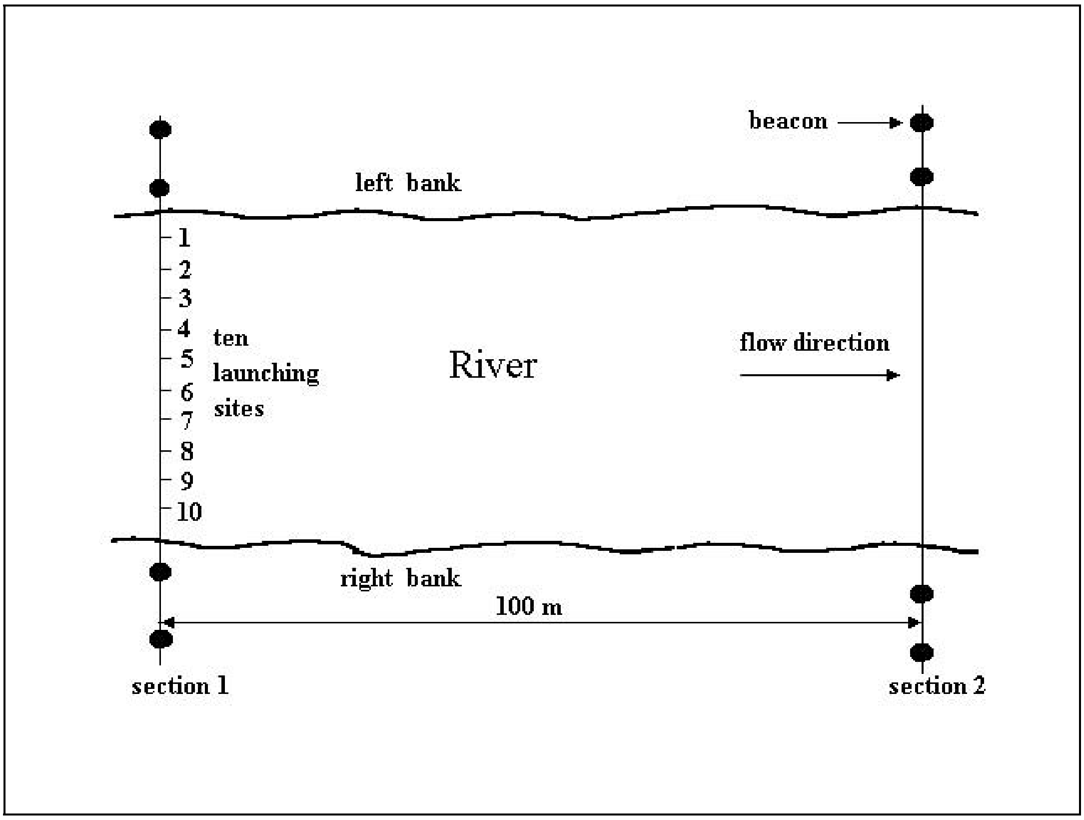

Figure 2.3: Layout of float measurement

A float measurement over a distance of 100 m has a relative error in the measurement of the velocity of 1%. A current meter does not have that degree of accuracy. Moreover, by following the flow trajectory, the velocity is correctly averaged in the longitudinal direction. The only problem remains the averaging over the cross-section. The variation over the cross-section may be substantial. The variation over the width appears to be the largest source of errors. Therefore, one should select at least ten measuring positions in the cross-section are selected.

This still leaves the problem of determining the cross-sectional area. Unless one has an echo sounder at one's disposal (and the river is navigable), this has, unfortunately, to be left to the “morning after” programme.

Flood mark survey

Immediately after the flood peak has passed one should go to the field and search for flood marks. Flood marks can be found in the colour of mud on bridges, pillars, or - in case of extreme flooding - on the walls of buildings. Also the presence of small floating debris in trees and bushes are good indications of the flood level. One should take into account, however, that bushes bend under the force of the flow and that considerable waves may occur. Both actions indicate higher flood levels than actually occurred. Flood marks on the banks, where wave action and run-up from surge are at a minimum, are generally preferable to those in bushes and trees. However, they disappear fast.

The first action to be taken is to paint the observed flood marks on walls and trees, where possible accompanied by the date of occurrence of the flood. A record of flood marks on the wall of a solid structure is an important future source of information.

Try to get as many reliable flood marks as possible along the river. Also in areas which at the time are not yet developed. A good survey of flood marks of an extreme flood is an invaluable asset for the planning of future projects.

Sometimes flood marks are difficult to find, principally because one is too late and rains have cleared the colouring, or winds have cleared the debris from the trees. A good method then is to install a levelling instrument at the suspected flood level and to look through the instrument towards different objects. If the instrument is indeed at the approximate flood mark position, then the accumulation of sometimes insignificant marks may help to verify the flood level.

If one is really too late to find back any traces of the flood, one should gather information from people living in the area. Needless to say that such information is much less reliable.

Finally, where possible, take photographs of flood marks, or of people indicating a flood mark.

Slope area method

For a good slope area computation one should look for a fairly straight stable clean channel, without pools, rapids, islands or sharp curves. No bridges or other obstructions should be downstream of the reach.

The reach to determine the cross-sectional area should be about ten times the width of the river; one should survey approximately five to ten cross-sections. The flood mark survey should be over a long enough distance to determine the water level slope accurately, taking into account the error of reading. This will often amount to a distance of several kilometres.

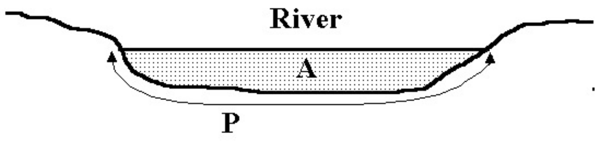

Figure 2.4: Cross-section of a river

The hydraulic calculations are based on Chézy's formula:

$$Q = CA\sqrt{R}\sqrt{S} = CK\sqrt{S} \qquad \textit{Equation 2.2}$$

where Q is the discharge in m^3/s
C is Chézy's coefficient of smoothness in $\sqrt{m}/s$
A is the cross-sectional area in m^2
R is the hydraulic radius in m
S is the longitudinal slope
K is the geometric conveyance in $m^{2.5}$

In Equation 2.2, the discharge Q, the Chézy coefficient C and the slope S are considered constant over the reach. However, there are sometimes strong variations in the geometry of the channel along the reach. To eliminate these variations the average value of K is determined of the surveyed cross-sections:

$$K = \frac{\sum\left(A_i\sqrt{R_i}\right)}{N} \qquad \textit{Equation 2.3}$$

where N is the number of sections surveyed and the index i indicates a certain cross-section.

In the above method the greatest difficulty lies in the determination of channel roughness. It is very difficult for even an experienced surveyor to arrive at an objective value. As long as people carry out the survey, the result obtained will always be subjective.

Simplified slope area method

A remarkable method developed by H.C. Riggs (1976) may be used to overcome the difficulty of subjectivity. Riggs postulates that in alluvial streams slope and roughness are related. In more popular terms this means that the river adjusts its roughness (through bed forms) to the slope, or that the river adjusts its slope (through meandering) to the roughness. Experiments have shown that there indeed is a relation between roughness and slope, although the relation shows considerable scatter.

However, Riggs showed that the error in the relation between slope and roughness is less than the error made by experienced surveyors in estimating the roughness.

The equation of the simplified method reads:

$$\log Q = 0.188 + 1.33 \log A + 0.05 \log S - 0.056 (\log S)^2 \qquad \textit{Equation 2.4}$$

In this equation the value of *A* should be representative for the reach under study. The cross-sectional area *A* should be determined on the basis of five to ten cross-sections:

$$A = \frac{\sum A_i}{N} \qquad \textit{Equation 2.5}$$

where *N* is the number of sections surveyed. The equation was tested in 64 rivers in the USA with discharges ranging from 2 m^3/s to 2500 m^3/s, and with Chézy coefficients ranging from 14 to 65 $m^{0.5}/s$. The method is extremely useful, also as a check on the previously mentioned slope area method.

2.3 Hydrograph analysis

A hydrograph is the graphical representation of the instantaneous discharge of a stream plotted with time (see Figure 2.5). It includes the integrated contributions from surface runoff, groundwater seepage, drainage and channel precipitation. The shape of a hydrograph of a single storm occurring over the drainage area follows a general pattern. This pattern shows a period of rise that culminates in a peak, followed by a period of decreasing discharge (called recession) which may, or may not, decrease to zero discharge, depending on the amount of groundwater flow.

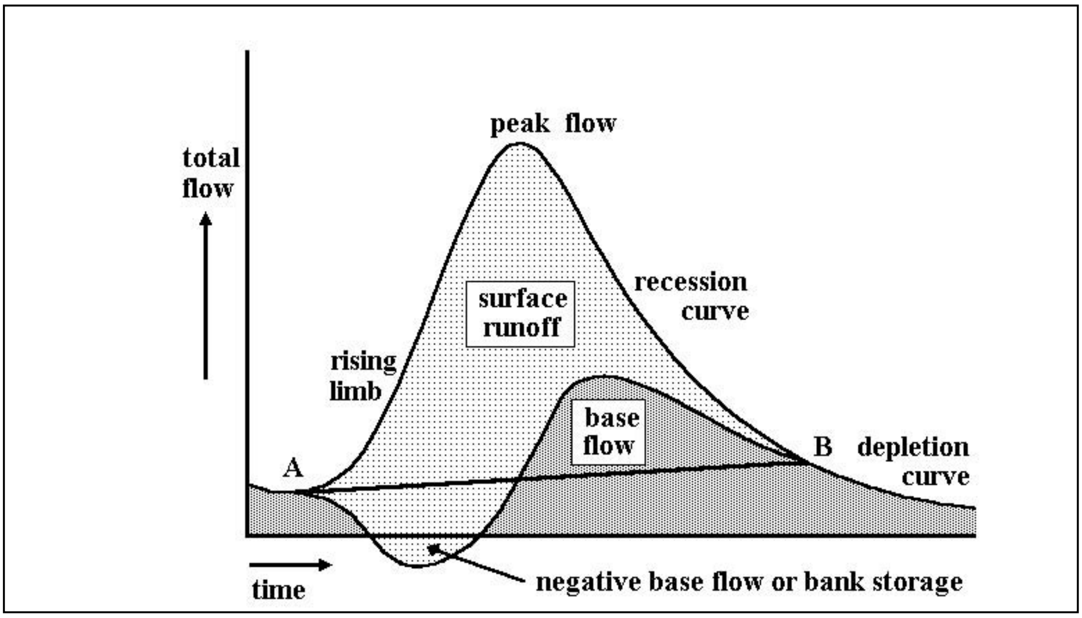

Figure 2.5: Components of hydrograph

The hydrograph has two main components, a broad band near the time axis representing base flow contributed from groundwater, and the remaining area above the base flow, the surface runoff, which is produced by the storm.

At the beginning of the rainfall, the river discharge is low and a period of time elapses before the river begins to rise. During this period the rainfall is intercepted by vegetation or soaks into the ground to make up the soil moisture deficit. The length of the delay before the river rises depends on the wetness of the catchment before the storm and on the intensity of the rainfall itself. When the rainfall has made up the catchment deficits and when surfaces and soils are saturated, the rains begin to contribute to the stream flow. In the classical definition of storm runoff, the proportion of the rainfall that finds its way directly to the river is known as effective rainfall; the rest is either intercepted or infiltrated into the soil. In a more modern view of effective rainfall, there also is an important component of subsurface runoff to storm flow. In that definition, the effective rainfall is the part of the rainfall that contributes to fast runoff. The remainder either evaporates (by both interception and transpiration) or percolates to the deeper groundwater where it feeds the base flow. As the storm proceeds, the proportion of effective rainfall increases, resulting in a strongly rising limb.

The peak of the hydrograph is reached after the effective rainfall has reached its maximum. The time difference between the maximum effective rainfall intensity and the maximum runoff is called the time lag. There is a time required for the surface runoff to reach the station where the hydrograph is observed. First the area closest to the station contributes to the surface runoff, followed by the areas further upstream. This means that in a small catchment, for a given uniformly distributed rainfall, the time to peak, and also the lag time, will be shorter than in a large catchment.

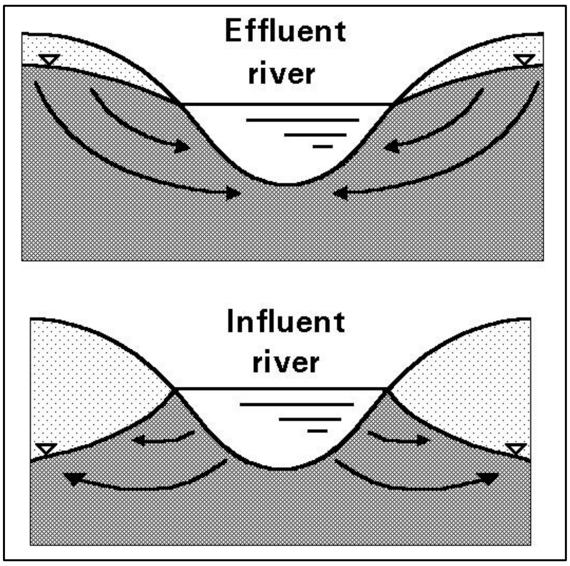

Figure 2.6: Influent and effluent streams

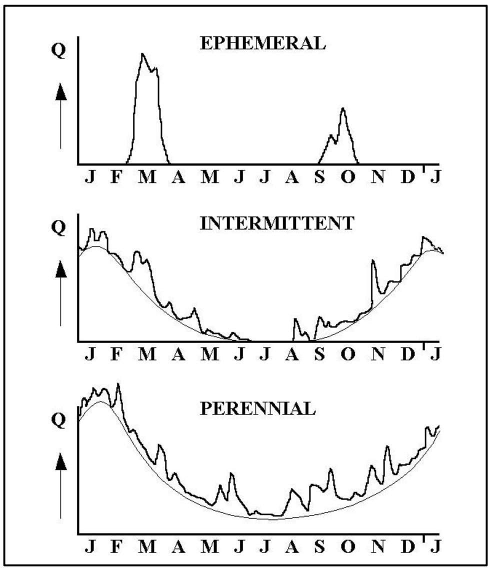

Figure 2.7: Classification of rivers

The separation between surface runoff and base flow is difficult to make and depends strongly on the geological structure and composition of the catchment. Permeable aquifers, such as limestone and sandstone strata, react much faster than impervious clays. During the course of an individual rainfall event, the base flow component continues to fall even after river levels have begun to rise, and only when the storm rainfall has had the time to percolate down to the water table does the base flow component begin to increase. The base flow component usually finishes at a higher level at the end of the storm than at the rise of the hydrograph, and thus there is an increase in the river runoff from groundwater seepage. Groundwater provides the total flow of the general recession curve until the next period of rainfall.

Since base flow represents the discharge of aquifers, changes occur slowly and there is a lag between cause and effect that can easily extend to periods of weeks or month. This depends on the transmissivity of the aquifers bordering the stream and the climate.

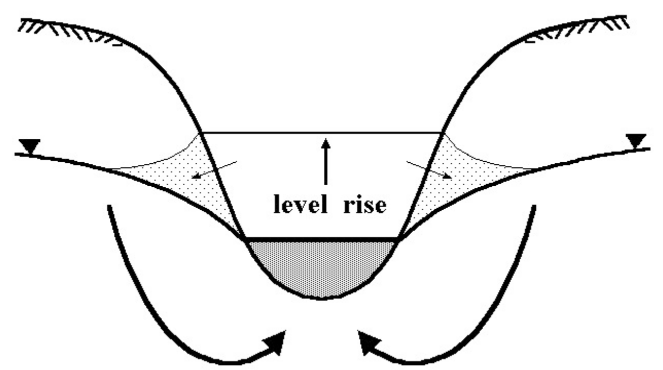

Figure 2.8: Temporary change of river, from effluent to influent due to a sudden rise of the water level

In this context it is important to distinguish between *influent* and *effluent* streams (see Figure 2.6). During periods when the groundwater level is higher than the water level in the stream channel, the river is considered effluent, draining the surrounding area. When the water level in the stream is higher than the groundwater level, however, water flows from the river into the soil and the stream is considered influent. A stream that is influent over a considerable length may dry up during rainless periods (e.g. wadi's in desert areas). Such a stream is called ephemeral. More common are intermittent rivers, which dry up during only a short period of the year during which (part of) the river becomes influent. Rivers that carry water all year round are often fed by rain as well as snow melt and known as perennial rivers. Characteristic hydrographs for the various types of rivers are given in Figure 2.7.

The fact that water may flow from the river into the groundwater affects the separation line between surface water and groundwater. Figure 2.8 shows the effect that temporary recharge of groundwater can have on the shape of this line, even leading to negative base flow as indicated in Figure 2.5. If the recession curve is solely the effect of groundwater seepage, it is called a depletion curve (for the derivation see Equation1.23). It may be described by as:

$$Q_t = Q_0 \exp\left(-\frac{(t-t_0)}{K}\right) \qquad \textit{Equation 2.6}$$

The equation produces a straight line when plotted on semi-logarithmic paper, which allows the determination of K. Once the value of K has been determined, Equation 2.6 may be used to forecast low discharges. For a continued dry period, the available flow can be accurately predicted by this equation.

In practise it is impossible to determine the separation line between the surface runoff and the base flow. Figure 2.9 shows an approximate method using a straight line, connecting the points A and B, which points are found from a plot of the hydrograph on semi-logarithmic paper as shown in Figure 2.9. The point A and B are located where the extrapolated depletion curves, depart from the observed hydrograph, just before the start of the storm and immediately after the cessation of the surface runoff.

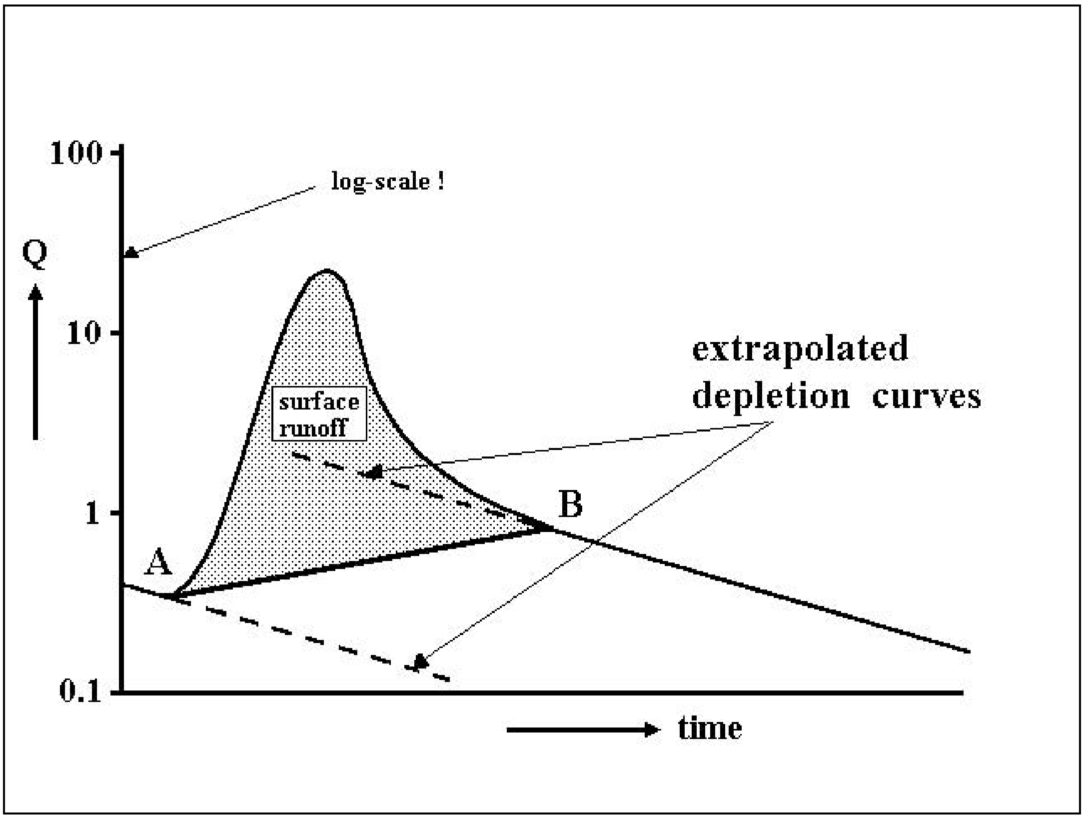

Figure 2.9: Hydrograph separation

2.4 Factors affecting hydrograph shape

The time distribution of runoff (the shape of the hydrograph) is influenced by climatic, topographic and geological factors. The climatic and topographic factors mainly affect the rising limb whereas the geological factors determine the recession limb.

Climatic factors

The climatic factors that influence the hydrograph shape and the volume of runoff are:

1. rainfall intensity
2. rainfall duration;
3. distribution of rainfall on the basin;
4. direction of storm movement;
5. type of storm.

1. Rainfall intensity affects the amount of runoff and the peak flow rate. For a given rainfall duration, an increase in intensity will increase the peak discharge and the runoff volume, provided the infiltration rate of the soil is exceeded.

2. Rainfall duration affects the amount of runoff, the peak flow rate and the duration of surface runoff. For a rain of given intensity, the rainfall duration determines, in part, the peak flow. If a storm lasts long enough, eventually almost all the

precipitation will become runoff (the time after which this occurs is called the time of concentration); consequently the peak flow will approach a rate equal to the product i*A, where i is the rainfall intensity and A is the area of the basin. This situation is never reached in large basins, but may occur in small watersheds and is frequently used as the criterion for design of storm sewers, airport drainage or small culverts.

3. The areal distribution of rainfall can cause variations in hydrograph shape. If an area of high rainfall is near to the basin outlet, a rapid rise, sharp peak and rapid recession of the hydrograph usually result. If a larger amount of rainfall occurs in the upper reaches of a basin, the hydrograph exhibits a lower and broader peak.

4. The direction of storm movement with respect to orientation of the basin affects both the magnitude of the peak flow and the duration of surface runoff. Storm direction has the greatest effect on elongated basins. On these basins, storms that move upstream tend to produce lower peaks of a longer duration than storms that move downstream.

5. The type of storm is important in that thunderstorms produce peak flows on small basins, whereas large cyclonic or frontal-type storms are generally determinant in larger basins.

Topographic and geologic factors

The topographic and geologic factors affecting runoff represent the physical characteristics of the basin. The factors involved are numerous, some having a major bearing on the phenomena, whereas others may have a negligible effect, depending on the catchment under consideration. The following are the dominant factors:

1. catchment size;
2. catchment shape;
3. distribution of watercourses;
4. slope of the catchment;
5. storage in the catchment;
6. geology of the catchment;
7. land use.

1. The major effect of increasing the drainage area on the hydrograph shape is that the time base of the hydrograph is lengthened. The peak flow per unit area thus reduces with catchment size for a given rainfall depth. This is partly due to the rainfall intensity being less for storms of extensive size, and partly due to the longer time required for the total catchment area to contribute to the peak runoff (time of concentration).

2. The effect of shape can best be demonstrated by considering the hydrographs of discharges from three differently shaped catchments with the same surface area, subject to rainfall of the same intensity (see Figure 2.10). The lines of equal run-time to the outlet show that shape B has the smallest time of concentration (5 hours), and thus reaches the peak after 5 hours. The most elongated catchment needs 10 hours to reach the peak. Also the effect is shown in the hydrographs of a storm that moves upstream (a1) and downstream (a2) in the elongated catchment. It can be seen that the rise is sudden when the storm moves downstream and slow when it moves upstream.

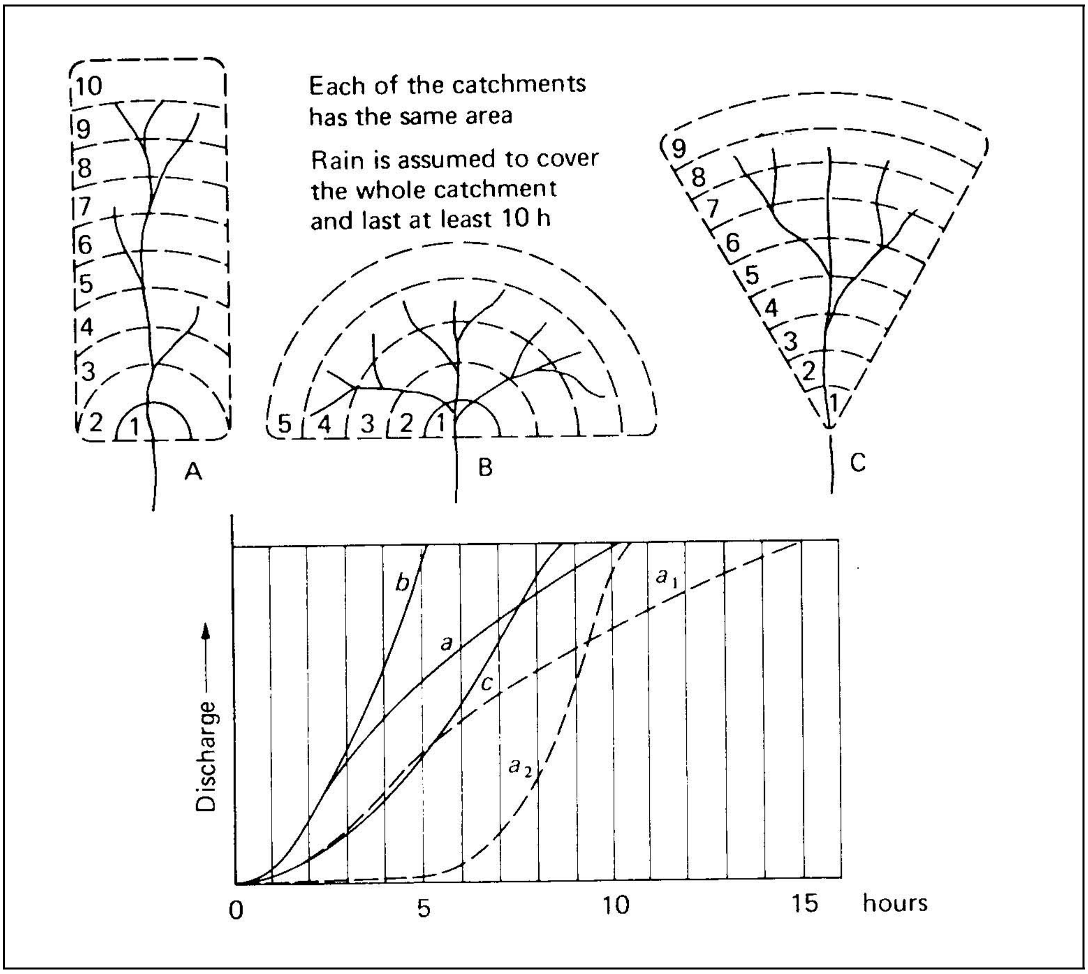

Figure 2.10: The effect of shape on catchment runoff (after Wilson, 1983)

3. The pattern and arrangement of the natural stream channels determine the efficiency of the drainage system. Other factors being constant, the time required for water to flow a given distance is directly proportional to length. Since a well-defined system reduces the distance water must move overland, the corresponding reduction in time involved is reflected by an outflow hydrograph having a short time to peak.

4. The steeper the slope of the catchment, the more rapidly surface runoff will travel. The time to peak will be shorter and the peaks will be higher. Infiltration capacities tend to be lower as slopes get steeper, thus accentuating runoff.

5. Since storage must first be filled before it empties, it has a delaying and modifying effect on hydrograph shape. Much of the variation caused by the above factors are smoothed out by natural or artificial storage.

6. The pedology and geology of the catchment influence primarily the sub-surface runoff and the "losses". High infiltration rates reduce the surface runoff; high permeabilities combined with high transmissivities substantially enhance the baseflow component. The occurrence of preferential path ways in the unsaturated and saturated zone can have a substantial influence on the fast component of sub-surface runoff. The type of stream (influent effluent, or intermittent) can have a substantial impact on hydrograph shape (see Figure 2.7).

7. Land use, finally, can strongly influence the runoff coefficient. Urbanized areas may have a runoff coefficient of almost 100%, whereas natural vegetation may have low runoff. Ploughing, drainage, cropping intensity, afforestation etc. also have a considerable effect on runoff.

2.5 Runoff data analysis

Of primary importance in the study of surface water are river discharges and related questions on the frequency and duration of normal flows (e.g. for hydropower production or for water availability) and extreme flows (floods and droughts). We shall first deal with the normal flows, and subsequently with extreme flows.

The question, which a hydraulic engineer would ask a hydrologist concerning normal flows, is the length of time (duration) that a certain river flow is expected to be exceeded. An answer to this question is provided by the flow duration curve that is the relationship between any given discharge and the percentage of time that the discharge is exceeded. The flow duration curve only applies for the period for which it was derived. If this is a long period, say more than 10 to 20 years, the flow duration curve may be regarded as a probability curve, which may be used to estimate the percentage of time that a specified discharge will be equalled or exceeded in the future.

As a numerical example to elucidate the derivation of the flow duration curve, consider the average daily discharges in m^3/s for a 20-day period as presented in Table 2.1. A period of 20 days is used in this example to restrict the number of values, but it should be noted that the derivation of flow duration curves for periods less than one year is generally not very useful.

The range of discharges are normally divided into 20 to 30 class intervals. In this example only 12 intervals are used as shown in Table 2.2. In the second column the number of days is given in which the flow belongs to the respective class interval. The cumulative totals of the number of days are presented in the third column. In the last column the cumulative percentages are listed.

Table 2.1: Average discharges for daily (k=1) and 10-days periods (k=10 days) observed during the 20 consecutive days

Discharges in m^3/s		
Days	**K = 1**	**K =10**
1	402	
2	493	
3	912	
4	1256	
5	1580	
6	1520	
7	1416	
8	1193	
9	1048	
10	966	1079
11	890	1127
12	846	1163
13	792	1151
14	731	1098
15	682	1008
16	602	917
17	580	833
18	512	765
19	473	707
20	442	655

Table 2.2: Derivation of flow duration curve

Class interval lower bound	Total class interval	Number greater than bottom of class interval	Percentage greater than bottom of c.i.
1500	2	2	10
1400	1	3	15
1300	0	3	15
1200	1	4	20
1100	1	5	25
1000	1	6	30
900	2	8	40
800	2	10	50
700	2	12	60
600	2	14	70
500	2	16	80
400	4	20	100

A plot of the discharge against the percentage of time that the discharge is exceeded (Figure 2.11) shows the typical shape of the flow duration curve. The presentation of the flow duration curve is much improved by plotting the cumulative discharge frequencies on log-probability paper (Figure 2.12). The logarithmic scale applies well to flows because the argument of the logarithmic function, just like real river flow, has a lower boundary of zero, which is reached asymptotically. The probability scale on the horizontal axis of Figure 2.12, is the Normal (or Gaussian) distribution. For normal flows, the Normal distribution generally leads to good results in combination with the logarithms of the flow. This is also called the Log-Normal distribution.

The same procedure can also be followed for periods of longer duration, e.g. ten days (k=10) or one month (k=30). The average discharges for 10 day periods are presented in Table 2.1 and the resulting flow duration curve is shown in Figure 2.12. The slope of this curve is flatter since the averaging of discharges removes the extremes. One may read from this curve that in 90 % of the time the average discharge during 10 consecutive days is equal to or less than 700 m^3/s. This type of information is important for instance in sewage works design, because it indicates the time during which the flow in the river may not provide adequate dilution for the effluent; but also for the design or licensing of water withdrawal, power plants, storage reservoirs, etc.

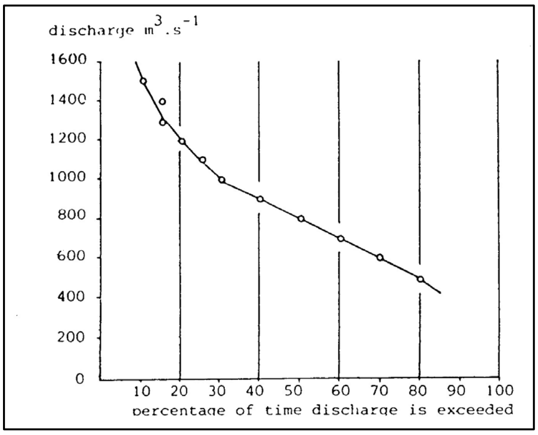

Figure 2.11: Flow duration curve for the example discussed in the text

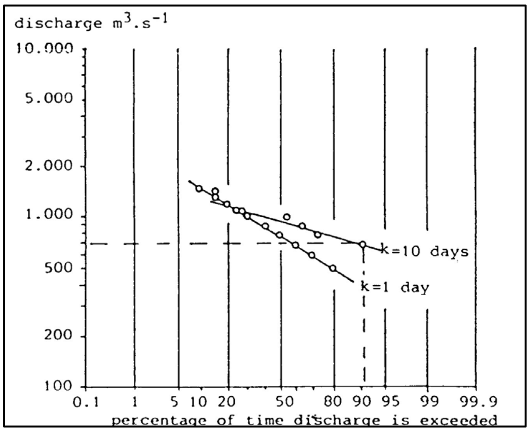

Figure 2.12: Flow duration curve on log-probability paper for flow durations of k = 1 and k = 10 days

2.6 Flood frequency analysis

Apart from normal flow frequency, hydrologists are also interested in the occurrence of extreme events. For this purpose flow frequency curves may be derived which yield the probability that a certain annual maximum discharge is exceeded. For minimum flows similar curves may be developed giving the probability of occurrence of an annual minimum less than a given discharge. The statistical methods are similar to the methods discussed in Chapter 1 for precipitation data.

Depending on the phenomenon, different probability distributions are recommended. For example for droughts the Log-Gumbel type III distribution may be used, and for flood flows the Gumbel type I, Log-Gumbel, Pearson or Log-Pearson type III distribution. The Gumbel type I distribution applied to flood levels is presented first.

In the case of floods, where possible, extreme analysis should be done on flows. However, if flow records do not exist, or if a rating curve has not yet been established, then we may not have another choice than to analyse water levels. This is, however, risky particularly if we want to design flood protection (a dike).

If a river has a flood plain where we consider building a dike, then the water levels in the original situation (without a dike) will be less high during a flood than in the case where the dike is present. Hence, a dike elevation, which is based on the recorded flood levels, will underestimate the flood level after the dike has been built. Flows are not effected by the shape of the cross-section, or, at least, to a lesser extent. Therefore, extreme analysis should be done on flows whenever possible. If no rating curve exists, then flood level analysis should be done with caution.

In the following, the Gumbel type I and Pearson type III distributions are presented for the analysis of flood levels. The same method, however, can be applied to flows. In addition a method is presented to estimate the flood frequency distribution in the case that no records exist.

Gumbel type I

In 1941, Gumbel developed the Extreme Value Distribution. This distribution has been used with success to describe many hydrological events. As applied to extreme values, the fundamental theorem can be stated:

> If X_1, X_2, X_3, ... X_n are independent extreme values observed in n samples of equal size N (e.g. years), and if X is an unlimited exponentially-distributed variable, then as n and N approach infinity, the cumulative probability q that any of the extremes will be less than a given value X_i is given by:
>
> $$q = \exp\left(-\exp\left(-y\right)\right) \qquad \textit{Equation 2.7}$$

where q is the probability of non-exceedance, y is the reduced variate. If the probability that X will be exceeded is defined as p=1-q, then the Equation 2.7 yields:

$$y = -\ln\left(-\ln\left(1-p\right)\right) = -\ln\left(-\ln\left(1-1/T\right)\right) \qquad \textit{Equation 2.8}$$

where T is the return period measured in sample sizes N (e.g. years). Table 2.3 lists a number of values of the reduced variate y, the probability of non-occurrence p and the return period T.

Table 2.3: Values of the reduced variate as a function of the probability of non-exceedance q and return period T

q [%]	T [years]	y
1.0	1.01	-1.53
5.0	1.05	-1.10
10.0	1.11	-0.83
20.0	1.25	-0.48
30.0	1.43	-0.19
50.0	2	0.37
66.6	3	0.90
80.0	5	1.50
90.0	10	2.25
95.0	20	2.97
98.0	50	3.90
99.0	100	4.60
99.5	200	5.30
99.8	500	6.21
99.9	1000	6.91

According to Gumbel, the reduced variate is defined as a linear function of X:

$$y = a\left(X - b\right) \qquad \textit{Equation 2.9}$$

where a is the dispersion factor and b is the mode. This reduced variate is much like the reduced variate of the Gaussian probability distribution: $t=(x-\mu)/\sigma$.

Gumbel showed that if the sample n goes to infinity:

$$b = X_m - 0.45005s \qquad \textit{Equation 2.10}$$

$$a = 1.28255/s \qquad \textit{Equation 2.11}$$

where X_m is the mean of X and s the standard deviation of the sample. If the sample is finite, which they always are - already a series of 20 years (n=20) is large -, the coefficients a and b are adjusted according to the following equations:

$$b = X_m - s\frac{y_m}{s_y}$$ *Equation 2.12*

$$a = \frac{s_y}{s}$$ *Equation 2.13*

values of s_y (the standard deviation of the reduced variate) and y_m (the mean of the reduced variate) as a function of n are tabulated in
Table 2.4. Equation 2.9 is thus modified to:

$$X = X_m + \frac{(y - y_m)s}{s_y}$$ *Equation 2.14*

Table 2.4: Theoretical values for the mean and the standard deviation of the reduced variate

N	$\mathbf{y_m}$	$\mathbf{s_y}$	$\mathbf{y_m/s_y}$
5	0.459	0.793	0.579
10	0.495	0.950	0.521
20	0.524	1.062	0.493
30	0.536	1.112	0.482
40	0.544	1.141	0.477
50	0.549	1.160	0.473
60	0.552	1.175	0.470
70	0.555	1.185	0.468
80	0.557	1.193	0.467
90	0.559	1.200	0.466
100	0.560	1.206	0.464
150	0.565	1.225	0.461
200	0.567	1.236	0.459
	0.577	1.283	0.450

On probability paper where the horizontal axis is linear in y, Equation 2.14 plots a straight line. To plot the data points on the horizontal axis a - so called - plotting position, or estimator, of the probability of non-exceedance q is required.
The following plotting position is used:

$$q = 1 - p = 1 - \frac{i}{n+1}$$ *Equation 2.15*

where i is the rank number of the maximum occurrences in decreasing order and n is the total number of years of observations. At the bottom of the figure the linear axis of the reduced variate is accompanied by the frequency scale of q, and on the top of the figure by the scale of the return period T. In Figure 2.13 an example is given of the maximum annual flood levels occurred in the Sabie river in Mozambique at Machatuine.

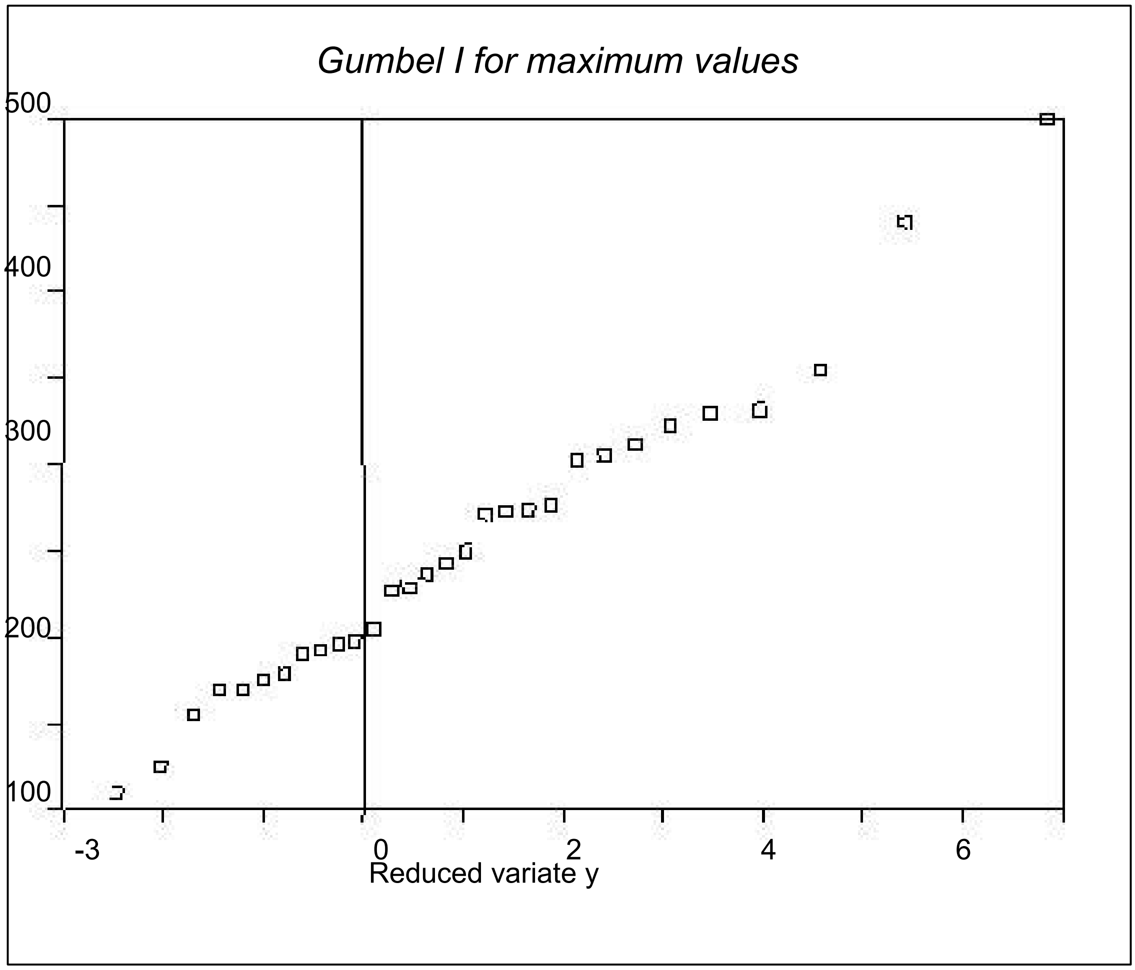

Figure 2.13: Gumbel probability distribution of annual maximum flood levels in the Sabie river at Machatuine

Log-Gumbel type III

The Log-Gumbel III distribution is often quite adequate for the analysis of extreme flows. In Figure 2.15 it is applied to the Sabie river catchment in Mozambique at Machutuine. The probability of exceedance p for the case of minimum flow is computed by:

$$p = 1 - \frac{i - 0.25}{n + 0.5} \qquad \textit{Equation 2.16}$$

which is a - so called - plotting position, where i is the rank number in decreasing order and n is the number of observations. The reduced variate y is computed by:

$$y = -\ln\left(-\ln\left(p\right)\right) \qquad \textit{Equation 2.17}$$

For maximum flow the Log-Gumbel distribution can be used as well. Figure 2.14 presents the case of the Incomati river in Mozambique at Ressano Garcia. In this case the probability of non-exceedance q is determined by:

$$q = 1 - \frac{i}{n + 1} \qquad \textit{Equation 2.18}$$

and the reduced variate is given by:

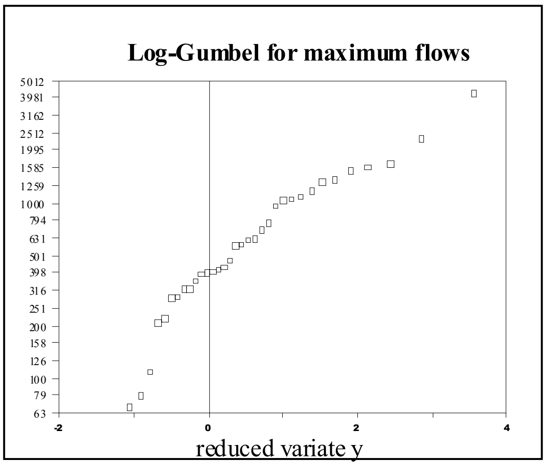

Figure 2.14: Log-Gumbel distribution for maximum flows in the Incomati river at Ressano Garcia

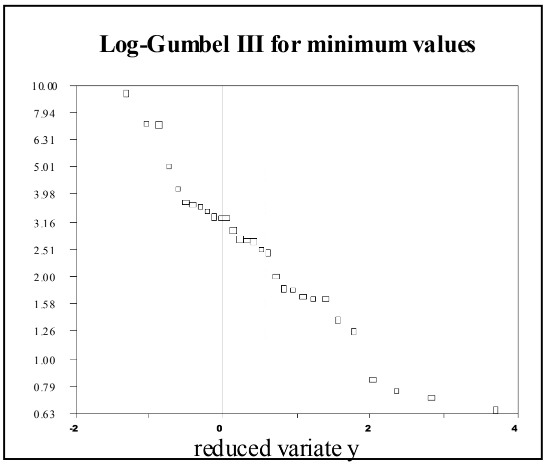

Figure 2.15: Log-Gumbel III analysis for minimum flow in the Sabie river at Machatuine

$$y = -\ln(-\ln(q))$$ *Equation 2.19*

It is not advisable to use partial duration series for runoff, as the observations within one year are generally correlated. The chance of independent data is greatly enhanced if an annual series is used on the basis of a properly selected hydrological year. Application of statistical methods is, in general, more difficult compared to precipitation data, because a time series of runoff observations is often not homogeneous (activity of man) and the record is seldom long enough.

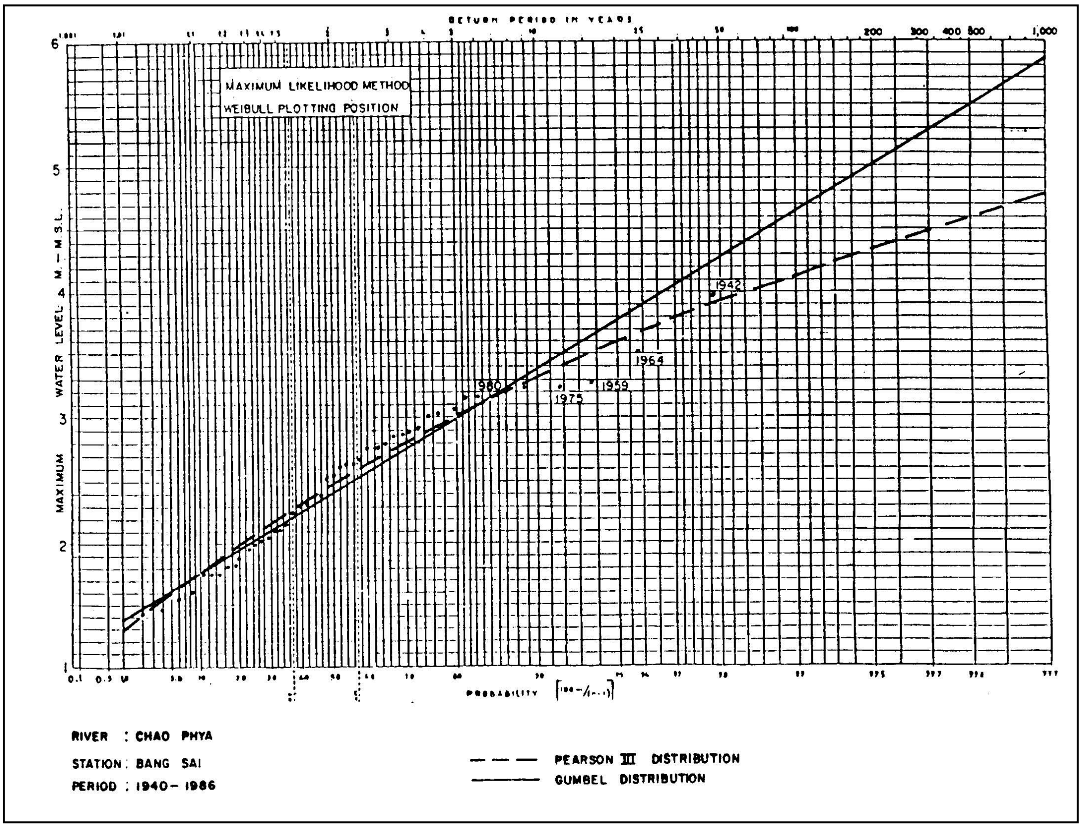

Figure 2.16: Gumbel and Pearson III distribution of annual flood levels in Bang Sai, Chao Phya river, Thailand (from Euroconsult, 1987)

Pearson type III

The Pearson type III equation does not necessarily plot a straight line on Gumbel paper. It assumes that the extreme values are distributed as a three-parameter gamma distribution. For more details about the computation of the distribution, reference should be made to textbooks. In Figure 2.16, the frequency distributions obtained with both Gumbel and Pearson III are presented for the water levels of the hydrometric station of Bang Sai in the Chao Phya river in Thailand (after Euroconsult, 1987). It can be clearly seen that the Pearson type III distribution performs better. However, this does not mean that the Pearson type III distribution reflects the real physical situation, and may be extrapolated indiscriminately. We have to take into account that Fig. 2.16 refers to water levels and not discharges. Even though, in this case, there was no overbank storage, we should be cautious to apply extreme values distributions to water levels.

It has been stated before that frequency analysis is a useful, but very limited method of determining design discharges or design water levels. The statistical methods used for frequency analysis have no physical basis; they are merely tools to help extrapolation. Therefore one has to check in the field if a discharge or a water level indicated by frequency analysis has indeed some physical meaning. The above methods for slope area calculations can be used to determine the discharge corresponding to a design water level, or to determine the water level corresponding to a design discharge. The effect of flood plains, natural levees, dikes etc. should then be taken into account.

Mixed distributions

In analogy with Section 1.2.2, where mixed distributions are presented for rainfall events, mixed distributions are appropriate in situations where there are different flood causing mechanisms at work. In such cases, the flood events have to be organised in sub-sets, to be analysed independently. Subsequently, the frequency distributions of the sub-sets should be combined using Equation 1.13. Different flood causing mechanisms that may have to be distinguished are: small scale convective events (thunder storms), cyclones or large scale events, ice melt, rain on snow, etc.

2.7 Lack of data

The statistical analysis of extremes requires a homogeneous time series of data of at least some 20 to 30 years. If such a time series is not available, which is often the case, one may distinguish between the situations that absolutely no data are available or that a short series has been observed.

No records available

If no records of any substantial length are available the following rule of thumb may prove valuable. The mode of the Gumbel annual flood frequency distribution (return period $T \approx 1.5$ years) generally lies at bank-full level, the elevation of the natural levee, or the natural bank height (see Figure 2.17). The consideration behind this is that the natural bank elevation is maintained at a level where it is regularly replenished with new alluvial material. This knowledge allows fixing one point of the frequency distribution on Gumbel probability paper.

The second point should be derived from interviews in the field. Detailed questioning of as many inhabitants of the area as possible may give you an indication of the largest flood in say 20 years.

Interviews should contain questions like:

- In which year do you remember the biggest flood?
- What age were you then?
- What age are you now?
- Where were you at the time of the flood?
- Can you indicate the level of the flood (on a tree, a house, a wall)?
- When did the second biggest flood that you can remember occur? (followed by the same detailing questions as above)

Always ask some questions that may confirm or contradict earlier statements like:

- Did you stay in the area or did you flee?
- If you fled, how do you know?
- Could you walk through the water or did you use a boat?
- Was there loss of life?
- Did cattle drown? If not, why?

Care should be taken to interview the right people. The more influential people who often present themselves as resource people, are not always the people who were on the spot during the event. Also, do not solely interview men, in many countries, the women do the work on the land and know the land best. Men are often away from home and hear the story afterwards.

In Figure 2.17 the second point that determines the Gumbel line has been indicated, as an example. In this case it followed from interviews that the flood with a return period of 20 years had a level of 7 m.

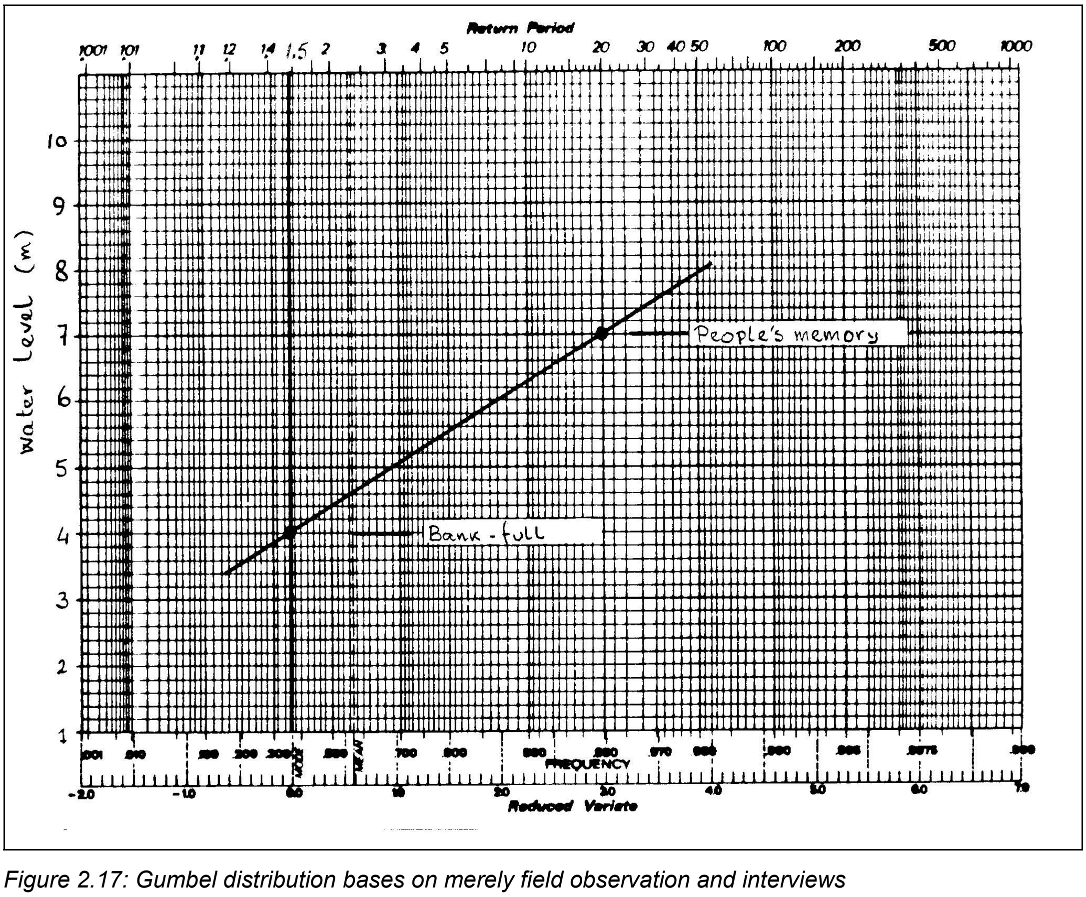

Figure 2.17: Gumbel distribution bases on merely field observation and interviews

Short series available

In the event of a short homogeneous record one may proceed along two different lines. One approach is known as **stochastic hydrology**. The short time series is analysed with respect to a trend component (a gradual change), a cyclic variation (periodic component) and a stochastic component. The first two components are deterministic in nature and relatively easy to quantify. The stochastic component contains random elements and is less easily identified. Each component is reproduced by mathematical simulation. The stochastic model is used to generate synthetic runoff data in sufficient large quantities to allow a statistical analysis.

This method is very risky when used as a frequency analysis tool. It has no physical foundation and the statistical material used to arrive at a long duration series is purely based on the short series of observations. Therefore, no justification exists to extrapolate the probability function beyond the duration of the observation period. Data generation, however, may be useful to determine time series for system simulation, for example to calculate the size of a reservoir. Thus it is an appropriate tool to generate normal flows but not to facilitate the analysis of extreme flows.

A completely different approach is called **deterministic hydrology**. It involves the development of a mathematical model to simulate the rainfall-runoff process. The deterministic approach requires a relatively short period of simultaneously observed rainfall and runoff to calibrate and verify the deterministic model. A deterministic model is a mathematical analogon that describes the link between cause (rainfall) and event (runoff). This does not necessarily mean that a deterministic model is physically based.

Some deterministic models are more physically based than others. So called black box models do not really try to describe the physics; they are merely a one-to-one relation between input and output. This means that black box models have to be recalibrated after changes have occurred in a catchment, and that they may not be used far beyond the range of calibration.

By making use of a deterministic model, the problem of finding a sufficiently long flow sequence is reduced to finding a sufficiently long rainfall series, which are generally more easily obtainable than runoff series.

From the rainfall series a design rainfall with a frequency of occurrence of for example once in 100 years may be derived. The deterministic model is then used to simulate the runoff from this event, implicitly assuming that the simulated runoff has the same return period of 100 years. Another approach is the simulation of runoff for the complete time series of rainfall observations, followed by a statistical analysis of the simulated runoff data. Especially in the case of complicated water resource systems, for example downstream of a confluence of different catchments, or in a system with a number of reservoirs, the latter method may yield different flow return periods from the rainfall return periods assumed.

Deterministic models are also used in real time to forecast flows on the basis of an existing situation. This subject is further dealt with in 2.8.3.

2.8 Rainfall runoff relations

A distinction is made between short and long duration rainfall-runoff relations. Short duration rainfall-runoff relations describe the process of how extreme rainfall becomes direct storm runoff. It yields peak flows and hydrographs that are used for the design of drainage systems (such as culverts, bridges sewers, retention ponds, etc.), spillways, or flood control storage. Long duration rainfall-runoff relations aim at establishing catchment yield for the purpose of water resources assessment.

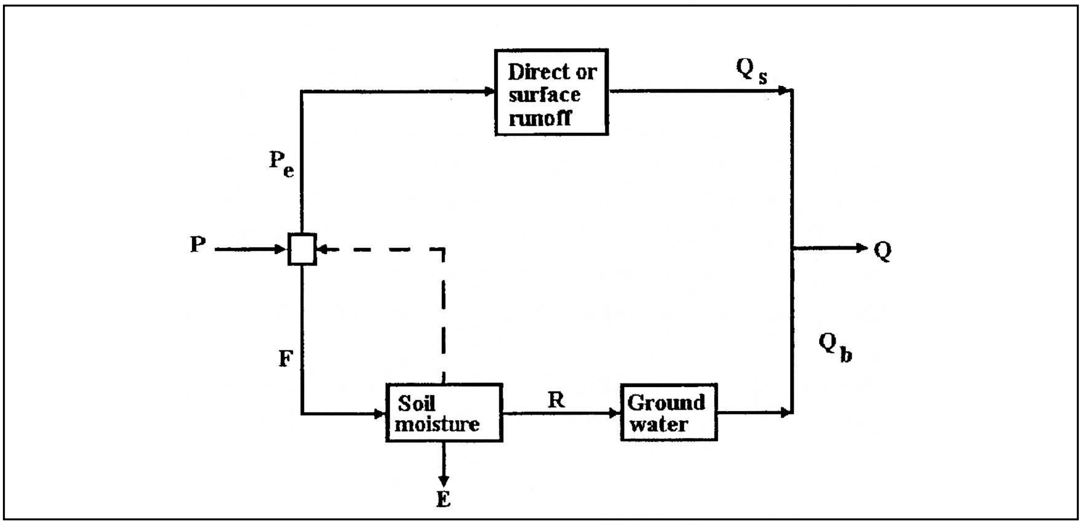

Figure 2.18: Simplified catchment model (Dooge, 1973)

2.8.1 Short duration peak runoff

For the determination of direct storm runoff, use can be made of the simplified catchment model of Dooge (see Figure 2.18). After subtraction of interception (I), part of the net rainfall *P* runs off over land (effective precipitation P_e) and the remaining part infiltrates *F*. The surface runoff, also termed direct runoff, contributing to the total river discharge *Q*, is indicated by Q_s. The infiltrated water *F* replenishes the soil moisture deficit or, if there is no deficit, it recharges the groundwater system *R*. The wetness of the soil affects the infiltration, so there is a feedback of the soil moisture situation on the division of precipitation into effective precipitation and infiltration. The soil moisture may evaporate (mostly transpiration) *E* or flow into the saturated groundwater system *R*. Outflow from the aquifer into the river is called base flow, Q_b. To determine the direct runoff, the effective precipitation P_e is required as input data. The effective precipitation P_e may be written as:

$$P_e = P - F - I \qquad \text{Equation 2.20}$$

where *P* is the precipitation, I is the evaporation from interception and *F* is the infiltration. It should be noted that in a more detailed approach to rainfall-runoff modelling, surface detention should be added to this equation.

When discussing rainfall-runoff processes, infiltration, interception and surface detention are generally considered "losses". Although it is not right to speak of losses (see Section 1.3.4), in the following, the term losses will in some places be used to indicated reduction of runoff, since this is considered common terminology in this regard.

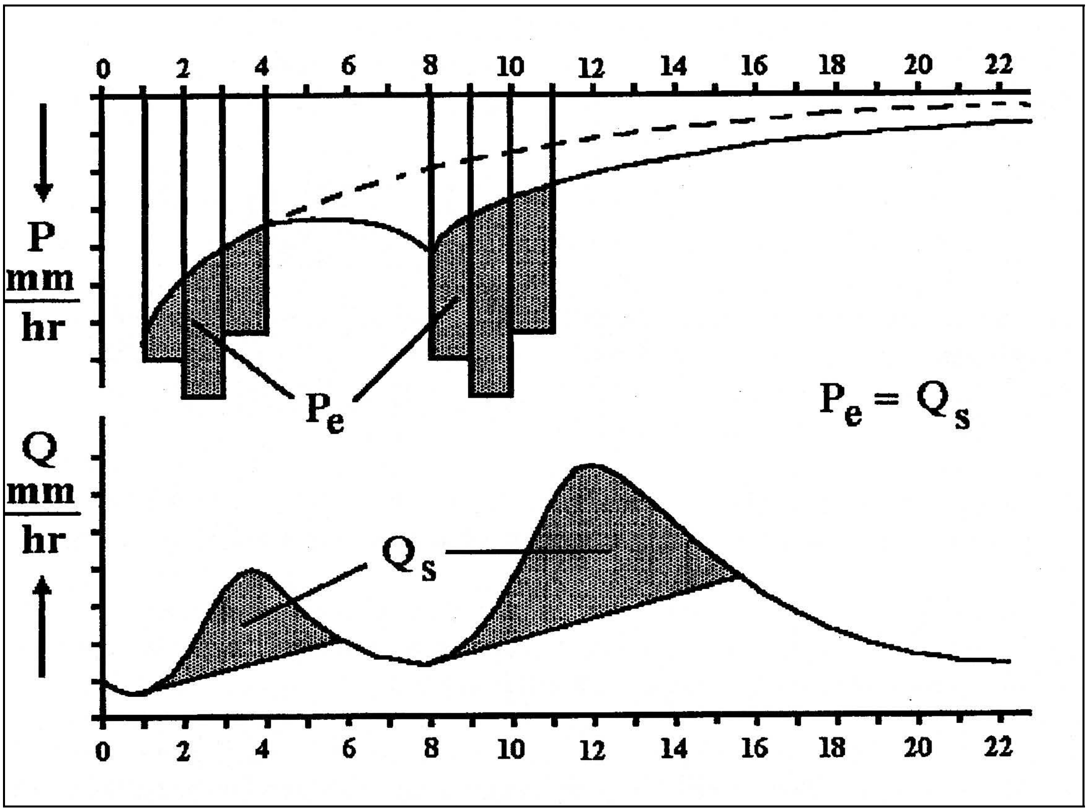

Figure 2.19: The effect of infiltration losses on the effective precipitation and direct runoff

The infiltration changes with time and depends on antecedent conditions (wet or dry soil). A plot of two exactly equal rainstorms and the corresponding runoff hydrograph under different antecedent conditions is given in Figure 2.19. Note that the discharge Q is usually measured in m^3/s. For convenience the values have been divided by the area of the catchment (m^2) to yield m/s. This unit is converted to mm/h, so that it is compatible with the unit of observed precipitation P.

At the start of the first storm the basin is dry and the infiltration rate, which is initially high, decreases with time. After the end of the rainstorm some recovery of the infiltration capacity takes place.

At the start of the second storm the basin is relatively wet and the infiltration is less than for the first storm. Subtraction of the infiltration from the observed rainfall P yields the effective rainfall P_e which is indicated by the shaded area. Since the direct storm runoff Q_s (in mm/h) equals the effective precipitation the second storm causes a larger peak, provided the base flow contribution to the total runoff remains unchanged.

Figure 2.19 shows the variation in infiltration with time. In general, the available data do not justify such a detailed approach. An alternative is the separation of the base flow from the total runoff, which yields the surface runoff Q_s. Since $Q_s = P_e$ (if expressed in mm/h) the 'losses' are found by subtracting Q_s from the observed rainfall. This may be done for each individual rainstorm and expressed as a fixed loss rate, which is known as the Φ-index. Consider for example a rainstorm producing 30 mm of rain (see Figure 2.20). Assume that the direct runoff, found from hydrograph separation equals 17 mm. The Φ-index line is drawn such that the shaded area above this line equals 17 mm. For this example the Φ-index is 2 mm/h.

Though a constant loss rate is not very realistic, the method has the advantage that it includes interception and detention losses.

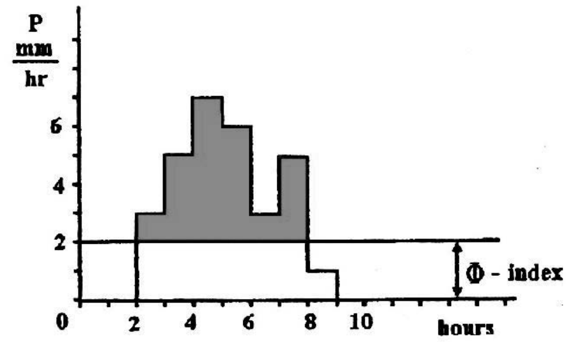

Figure 2.20: The Φ-index to indicate losses

There are various other methods to estimate the 'losses' or its complement, the runoff coefficient (the fraction of rainfall that comes to runoff). Well known empirical methods are the Coaxial Correlation Analysis developed by the US Weather Bureau and the Curve Number Method published by the Soil Conservation Service of US Department of Agriculture. The best results, however, are obtained by simulation of the relevant processes (transpiration, interception, unsaturated flow, etc.)

If no records are available to evaluate the relation between rainfall and peak runoff, a number of methods exist to arrive at peak runoff and hydrograph shape. For small catchments the most widely known and applied method is the "Rational Formula".

In the method of the Rational Formula it is assumed that the peak runoff occurs when the duration of the rainfall equals the time of concentration, the time required for the farthest point of the catchment to contribute to runoff. Numerous formulas exist for the time of concentration, each of which is applicable in the catchment for which it has been derived. However, the most widely applied formula is the Kirpich formula:

$$t_c = 0.015\left(\frac{L}{\sqrt{S}}\right)^{0.8} \qquad \textit{Equation 2.21}$$

where t_c is the time of concentration in minutes, L is the maximum length of the catchment in m and S is the slope of the catchment over the distance L. The formula to give the peak flow Q_p is:

$$Q_p = C \cdot i \cdot A \qquad \textit{Equation 2.22}$$

where C is the coefficient of runoff (dependent on catchment characteristics), i is the intensity of rainfall during time t_c and A is the catchment area. The value of i is assumed constant during t_c and the rain uniformly distributed over A. The peak flow Q_p occurs at time t_c. Figure 2.21 (after Gray, 1970) illustrates this. In the figure it can be seen that after the time t_c the maximum discharge continues until the end of the rain. The per unit area peak runoff $q_p = Q_p/A$ then equals $C\ i$. For the determination of a design flow, however, the duration of the rainfall t_R should be taken as equal to t_c. This is done by using an intensity-duration-frequency curve. For a certain return period, the intensity is selected corresponding to the duration t_c.

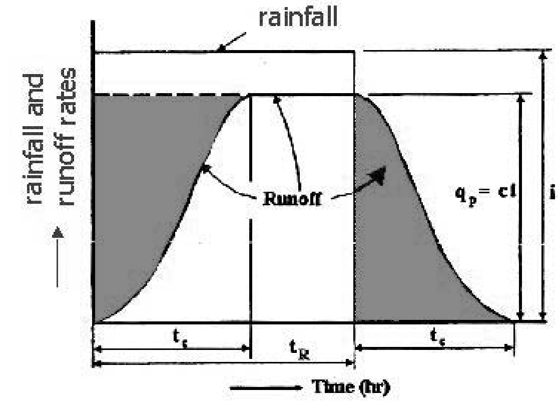

Figure 2.21: Runoff from uniform rainfall in the Rational Method (Gray, 1970)

It can be seen from Figure 2.21, that the volume of water accumulated in storage must be equal to the volume enclosed in the recession limb (shaded area). Thus, *C* represents not only the runoff coefficient for the peak, but also for the total volume of runoff. Values of *C* vary from 0.05 for flat sandy areas to 0.95 for impervious urban surfaces, and considerable knowledge is needed in order to estimate an acceptable value. Some values for the runoff coefficient are given in Table 2.5.

Table 2.5: Values of runoff coefficients

Type of drainage area	Runoff coefficient
Sandy soil	0.05-0.20
Heavy soil	0.13-0.35
Business	0.50-0.95
Residential	0.25-0.75
Industrial	0.50-0.90
Streets	0.75-0.95
Roofs	0.75-0.95
Forests	0.10-0.60
Pastures	0.10-0.60
Arable land	0.30-0.80

Although the Rational Formula is widely used, it is certainly not the best method available. Considerable uncertainty lies in the determination of the runoff coefficient, and its applicability is limited to small catchments (smaller than 15 Mm^2). A more advanced method of arriving at a peak flow and a design hydrograph is the unit hydrograph method, which can be applied on catchments of up to 5000 Mm^2. Unit hydrographs may be derived from observed rainfall storms and corresponding hydrographs, but there are also a number of methods of obtaining synthetic unit hydrographs.

2.8.2 Catchment yield

Water resources engineers are primarily concerned with catchment yields and usually study hydrometric records on a monthly basis. For that purpose short duration rainfall should be aggregated. In most countries monthly rainfall values are readily available. To determine catchment runoff characteristics, a comparison should be made between rainfall and runoff. For that purpose, the monthly mean discharges are converted first to volumes per month and then to an equivalent depth per month Q over the catchment area. Rainfall P and runoff Q being in the same units (e.g. in mm/month) may then be compared.

A typical monthly rainfall pattern is shown in Figure 2.22 for the catchment of the Cunapo river in Trinidad. The monthly runoff has been plotted on the same graph. Figure 2.23 shows the difference between Q and P, which partly consists of evaporation E (including interception, open water evaporation, bare soil evaporation and transpiration) and partly is caused by storage.
On a monthly basis one can write:

$$Q = P - E - \Delta S/\Delta t \qquad \textit{Equation 2.23}$$

The presence of the Evaporation and the Storage term makes it difficult to establish a straightforward relation between Q and P. The problem is further complicated in those regions of the world that have distinctive rainy and dry seasons. In those regions the different situation of storage and evaporation in the wet and dry season make it difficult to establish a direct relation.

Figure 2.24 shows the plot of monthly rainfall P against monthly runoff Q for a period of four years in the Cunapo catchment in Trinidad. The plots are indicated by a number which signifies the number of the month. The following conclusions can be drawn from studying the graph.

- There appears to be a clear threshold rainfall below which no runoff takes place. The threshold would incorporate such effects as interception, surface detention, and bare soil evaporation.
- It can be seen that the same amount of rainfall gives considerably more runoff at the end of the rainy season than at the start of the rainy season. The months with the numbers 10, 11 and 12 are at the end of the rainy season, whereas the rainy season begins (depending on the year) in the months of May to July. At the start of the rainy season the contribution of seepage to runoff is minimal, the ground-water storage is virtually empty and the amount to be replenished is considerable; the value of $\Delta S/\Delta t$ in Equation 2.23 is thus positive, reducing the runoff R. At the end of the rainy season the reverse occurs.

The threshold rainfall is quite in agreement with Equation 2.23 and has more physical meaning than the commonly used proportional evaporation “losses”. Proportional evaporation losses are rather a result of averaging. They can be derived from the fact that a high amount of monthly rainfall is liable to have occurred during a large number of rainy days, so that threshold losses like interception and open water evaporation have occurred a corresponding number of times.

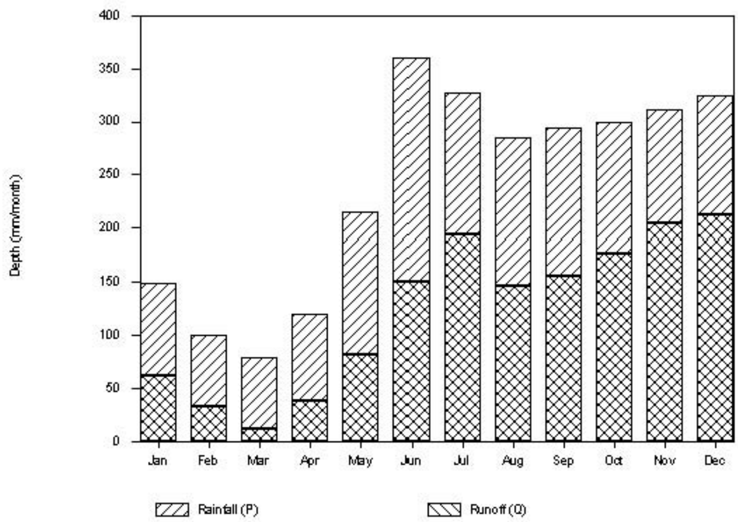

Figure 2.22: Monthly mean rainfall and runoff in the Cunapo catchment

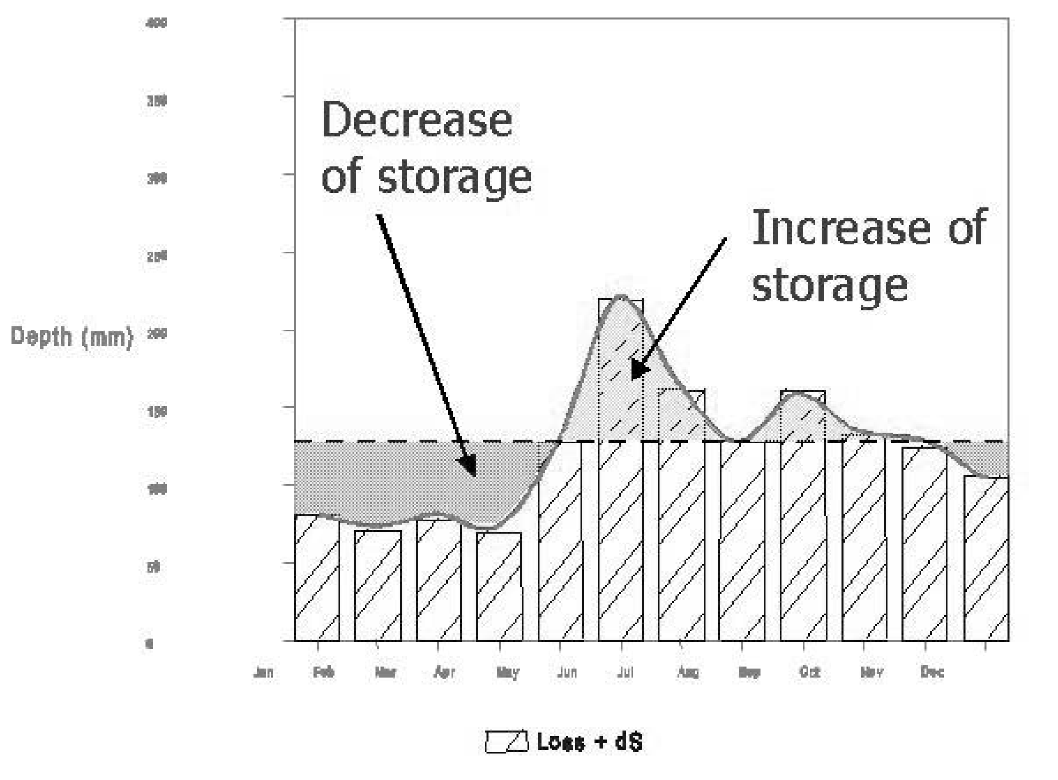

Figure 2.23: Mean monthly losses and change in storage in the Cunapo catchment

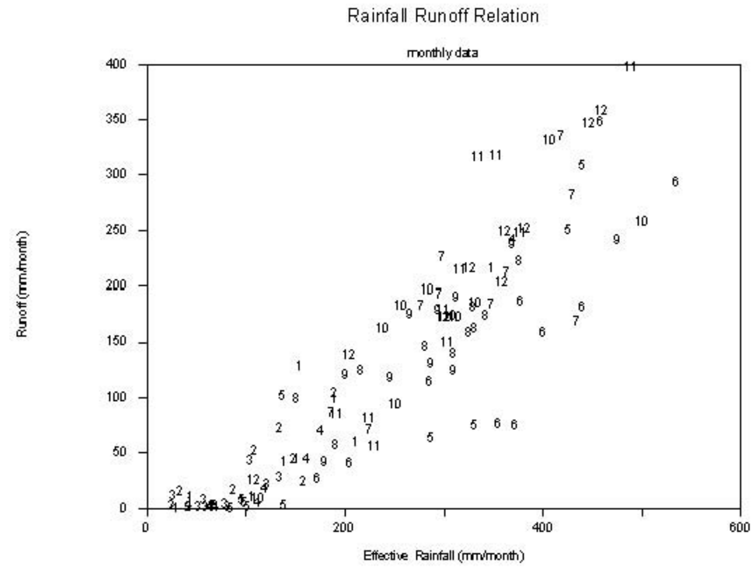

Figure 2.24: Rainfall plotted versus runoff in the Cunapo river basin

Moving average model for monthly runoff using a threshold

As the amount of storage available during a particular month depends on the amount of rainfall in the previous months, a relation is sought that relates the runoff in a particular month to the rainfall in the month itself and the previous months. A simple linear backward relation is used:

$$Q_t = a + b_0 \max(P_t - D,0) + b_1 \max(P_{t-1} - D,0) + ... \qquad \textit{Equation 2.24}$$

D is the threshold for fast evaporation on a monthly basis, b_i is the coefficient that determines the contribution of the effective rainfall in month *t-i* to the runoff in month *t*; and *a* is a coefficient which should be zero if the full set of rainfall contributions and evaporation losses were taken into account.

In matrix notation Equation 2.24 reads:

$$Q_t = B\left(P - D\right) + a \qquad \textit{Equation 2.25}$$

where Q_t is a scalar, the runoff in month *t*, **B** is an *n* by 1 matrix containing the coefficients b_i and (**P**-**D**) is a state vector of 1 by *n* containing the effective monthly precipitation values of the present and previous months. The value *n*-1 determines the memory of the system. Obviously *n* should never be more than 12, to avoid spurious correlation, but in practice *n* is seldom more than 6 to 7.

The effective runoff coefficient C, on a water year basis, is defined as:

$$C = \frac{\sum Q}{\sum \max(P - D, 0)} \qquad \textit{Equation 2.26}$$

It can be seen from comparison of Equations 2.25 and 2.26 that (if the coefficient *a* equals zero) the sum of the coefficients in **B** should equal the effective runoff coefficient C:

$$\sum (b_i) \approx C \qquad \textit{Equation 2.27}$$

meaning that the total amount of runoff that a certain net rainfall generates is the sum of all the components over n months. Obviously C should not be larger than unity.

The coefficients of **B** are determined through multiple linear regression. For the example of Figure 2.24 these results are presented in Table 2.6. Although the best correlation is obtained with a memory of 5 months, the most significant step is made by including the first month back. Moreover it can be seen that the correlation substantially improves by taking into account the threshold rainfall. Figure 2.25 shows the comparison between computed and measured runoff for a threshold value D = 120 mm/month.

Table 2.6: Summary of correlations Cunapo catchment

Threshold value: D = 0
Average effective runoff coefficient: C = 0.50

	t	t-1	t-2	t-3	t-4	t-5	t-6
R^2	0.76	0.76	0.77	0.78	0.78	0.78	0.80
Se	50.2	50.2	49.1	49.2	48.6	48.8	46.4

Threshold value: D = 120
Average effective runoff coefficient C = 0.90

	t	t-1	t-2	t-3	t-4	t-5	t-6
R^2	0.80	0.85	0.85	0.86	0.86	0.86	0.87
Se	45.6	39.6	39.7	38.9	38.9	38.3	37.7

Result best regression:

Memory: i = 3 months
Constant: a = 0
Coefficients:

b_0	b_1	b_2
0.73	0.18	0.01

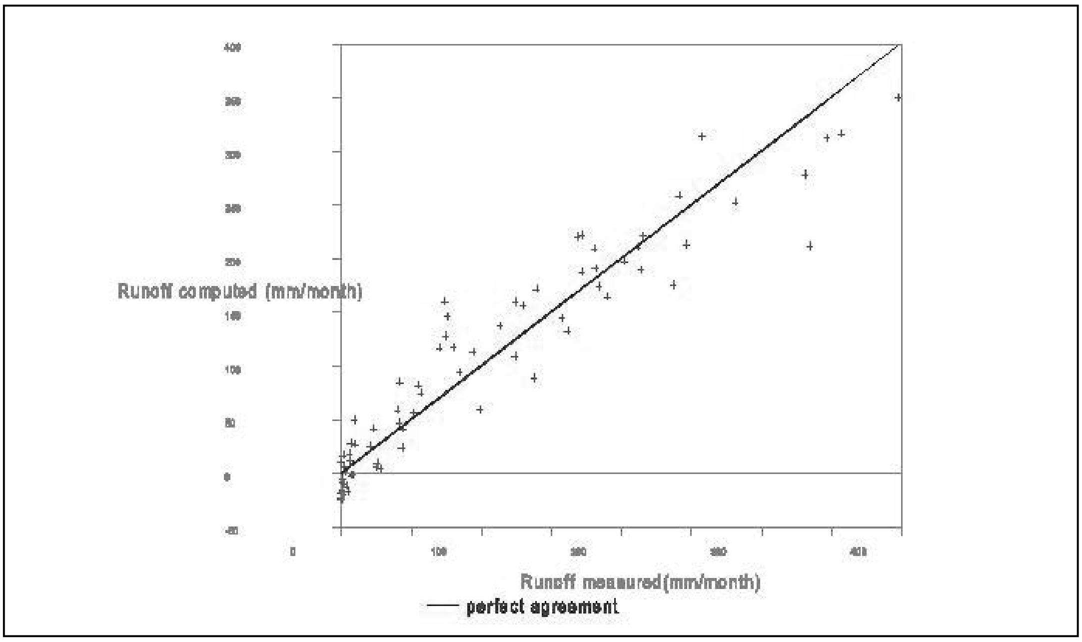

Figure 2.25: Measured versus computed runoff in the Cunapo catchment (D = 120)

2.8.3 Deterministic catchment models

The previous model is a statistical model on the basis of the water balance equation. In this section deterministic models are presented which try to describe the rainfall runoff process more or less on a physical basis.

Some models attempt to describe each physical process involved in the transfer of rainfall into runoff in detail (mathematical-physical models), while others do not try to describe the physics of the system at all (black box models). In between the two extremes are the conceptual models, which simplify (conceptualise) to a larger or lesser degree the complex rainfall-runoff process.

For the development of a deterministic model the hydrological processes are often drastically simplified. An example of a simplified catchment model has been presented earlier in Figure 2.18. The peak flow is dominated by the fast runoff component, the surface runoff Q_s. Many deterministic models, therefore, focus on the development of a relation between P_e and Q_s. The peak flow may then be found by adding the relatively small amount of base flow, which often constitutes less than 10 to 20 % of the peak discharge. Hence, a large error in the estimation of the base flow has only a small effect on the computed peak. The base flow component can be easily modelled by a linear reservoir method using Equation 1.21.

There is another reason why a separate modelling of the surface runoff system is attractive. It was found that, in particular for small watersheds, the relation between P_e and Q_s could often be considered as a linear system, i.e. twice the amount of P_e results in a doubling of the surface runoff values (e.g. the Unit Hydrograph concept). It appeared, furthermore, that the relationship is approximately constant in time. The properties of linearity and time-invariance allow the use of relatively simple techniques for the development of deterministic rainfall-runoff models. Application of these models requires a separation between surface flow and base flow.

The remaining part of the simplified catchment model involves evaporation and groundwater recharge, which constitutes a major problem due to the non-linearity of the processes.

Rainfall-runoff modelling discussed so far treats a whole catchment as if it were homogeneous in character and subject to uniform rainfall. These models are called lumped models. Most of the conceptual and black box models are lumped models. The mathematical-physical models usually belong to the category of distributed models. These models divide the catchment into small homogeneous subareas which are simulated separately and then combined to obtain the catchment response. Distributed models are generally much more complex than lumped models and require large amounts of distributed data. Recently hydrologists realise that the number of parameters required for distributed models is so large, that this leads to equifinality: different sets of parameters result in similar model output (see Beven, 1993, 1996, 2001; Savenije, 2001). Also the amount of data required is so large, that models can seldom be made operational. That's why many hydrologist turn back to simpler models (see Sivapalan & Zammit, 2001) or to data-based approaches.