CT5450
HYDROLOGY OF CATCHMENTS, RIVERS AND DELTA'S

Lecture notes

Prof. Dr. ir. H.H.G. Savenije

University of Technology Delft
Faculty of Civil Engineering and Geosciences
Water Resources Section

FOREWORD

Scope of the lecture Hydrology of catchments, Rivers and Deltas

This set of lecture notes supports the lecture Hydrology of Catchments, Rivers and Deltas. The lecture deals with advanced subjects of hydrology, including: Hydrology and water resources; Flood occurrence and analysis; Rainfall-runoff analysis; Reservoir and channel routing; and the Hydrology of coastal areas. Throughout the lecture the hydrology is approached from a global perspective, giving due attention to hydrological processes in tropical and sub-tropical regions, which generally experience a higher variability. Knowledge of more extreme circumstances than are generally experiences in a temperate climate is very useful, not only to the hydrologist who considers working in tropical environments, but also to the professional who chooses to work in a less dynamic climate and who is concerned with an environments where extremes become more and more dominant, be it as a result of climate change or as a result of human interference in our environment.

The domain of Hydrology and Water Resources

The field of Hydrology is broad and with the increasing societal attention for water, it is broadening even further. Therefore, it is useful to distinguish three major fields of Hydrology:

- Hydrology as an Earth Science
- Hydrology and Water Resources (Water Resources Research)
- Engineering Hydrology (Water Resources Engineering)

The first field encompasses the description and analysis of hydrological processes of the hydrological cycle. Distinctions can be made between atmospheric processes, surficial processes, river processes (hydraulics), soil processes, groundwater processes, and hydro-ecological processes, at different spatial and temporal scales. The most interesting, however, is the interconnection between these processes and the existence of feedback mechanisms.

In contrast of the first field, the second field is anthropocentric. It has to do with the use that Man makes of water, be it consumptive or non- consumptive. Water resources research is the study that looks into the most beneficial and sustainable use that Man can make of water. It is an important component of Water Resources Management. It is a science that primarily supports water policies and the formulation of strategies. It builds, of course, on hydrology as an earth science. It is a rapidly developing field.

The last field is closest to the Civil Engineer. It deals with hydrological research required for human interventions. It is hydrology that supports Water Resources Development. As a results, it is often more technology then science. It goes beyond saying that it should have a firm foundation in hydrological sciences, though. The lecture note that lies in front of you addresses primarily the second and third field. It aims at providing the tools and concepts that the modern engineer requires both in the Netherlands and abroad.

This lecture note, which first appeared in May 2000, has been revised and updated.

May 2006,
Dr. Hubert H.G. Savenije

Acknowledgements
This lecture note is the result of the work of many people. Chapters 1 and 2 are based on the joint work of the author and dr. P.J.M. de Laat of IHE. Section 3.2 is largely based on the book of R. Linsley and J. Franzini “Water Resources Engineering” (McGraw-Hill). Chapter 4 is based on the original lectures by Prof. A. Volker, whose work is gratefully acknowledged.

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1 HYDROLOGY AND WATER RESOURCES

Hydrology is the science that describes the occurrence of water on our planet and the processes that drive the circulation of the water between different stocks and locations where the water resides. When we use the words "water resources" we implicitly refer to the use that is made of the water, for whatever function. A resource is an input into some process of use, be it consumptive or non-consumptive. When we use the word resource, we imply a use or a function. Hence hydrology describes the occurrence and circulation of water, whereas water resources refer to the availability of water. Obviously the two are closely linked.

The origin of all terrestrial water resources is the rainfall. In that sense, rainfall is the most important water resource of all. Sections 1.1 and 0 deal with the origin and occurrence of rainfall, and hence deal with hydrology. Subsequently, sections 1.3 and 1.4 deal with water resources and water balances, in relation to the potential water use.

1.1 Precipitation, the origin of all water resources

1.1.1 The atmosphere

The lower layer of the atmosphere, up to a height of approximately 10 km, known as the troposphere, is the most interesting part from a hydrological point of view as it contains almost all of the atmospheric moisture. The percentage of water in moist air is usually less than 4 %. The composition of dry air is given in Table 1.1.

Table 1.1: Composition of dry air

	Mass %	Volume %
Nitrogen N_2	75.5	78.1
Oxygen O_2	23.1	20.9
Argon A	1.3	0.9
Others	0.1	0.1

The atmosphere depends for its heat content on the radiant energy from the sun. The atmosphere directly absorbs about one-sixth of the available solar energy, more than one third is reflected into space and less than one half is absorbed by the earth's surface. Heat from the earth's surface is released to the atmosphere by conduction (action of molecules of greater energy on those of less), convection (vertical interchange of masses of air) and radiation (long wave radiation). The process of conduction does not play a significant role. Radiation is an important link in the energy balance of the atmosphere. The "greenhouse effect", the increase of the atmospheric temperature as a result of a decrease in the long wave radiation, has grown into a worldwide concern. It is believed that man-produced gasses such as CO_2, CH_4 and N_2O obstruct the outgoing radiation, gradually leading to an accumulation of energy and hence a higher atmospheric temperature.

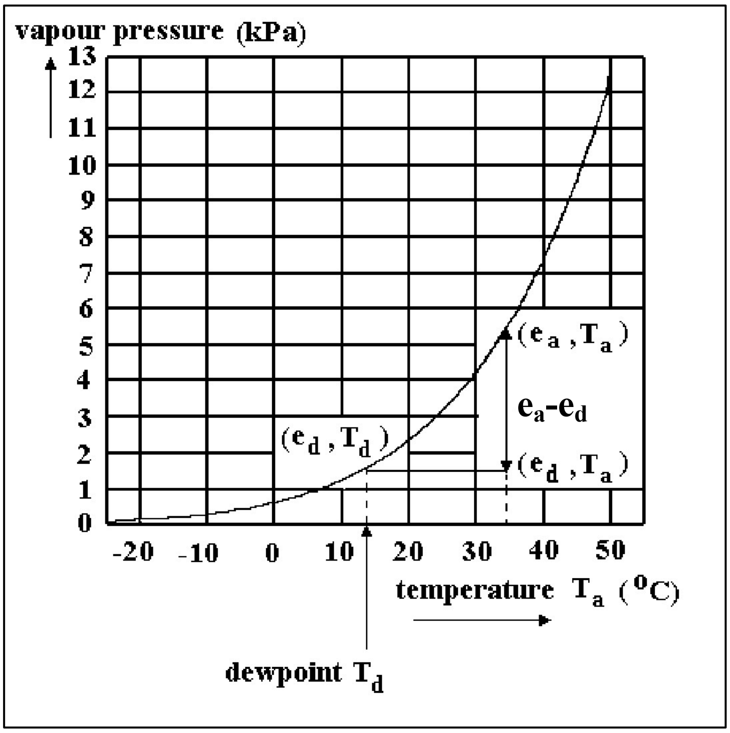


Figure 1.1: Relation between saturation vapour pressure e_a and air temperature T_a

Convection involves the vertical interchange of masses of air. The process is usually described assuming a parcel of air with approximate uniform properties, moving vertically without mixing with the surroundings. When a parcel of air moves upward it expands due to a decrease of the external pressure. The energy required for the expansion causes the temperature to fall. If the heat content of the air parcel remains constant, which is not uncommon (the parcel is transparent and absorbs little radiant heat), the conditions are called *adiabatic*. The rate of change of air temperature with height is known as the *lapse rate*. The average lapse rate is 0.65 °C per 100 m rise and varies from 1.0 under *dry-adiabatic* conditions to 0.56 under *saturated-adiabatic* conditions. The lower value for the lapse rate of saturated air results from the release of latent heat due to condensation of water vapour. Its value depends on the air temperature.

The pressure of the air is often specified in mbar and generally taken equal to 1013 mbar or 1.013 bar at mean sea level. In this note the SI unit for pressure Pa (Pascal) will be used, where 1000 mbar = 1 bar = 100,000 Pa = 100 kPa. Hence the pressure at mean sea level is taken as 101.3 kPa. The atmospheric pressure decreases with the height above the surface. For the lower atmosphere this rate is approximately 10 kPa per kilometre.

The water vapour content of the air, or humidity, is usually measured as a water vapour pressure (kPa). The water vapour pressure at which the air is saturated with water vapour, the *saturation vapour pressure* of the air, e_a is related to the temperature of the air as shown in Figure 1.1. If a certain mass of air is cooled while the vapour pressure remains constant; the air mass becomes saturated at a temperature known as *dewpoint temperature*, T_d. The corresponding *actual or dewpoint vapour pressure* of the air mass is indicated by e_d. The *vapour pressure deficit* is defined as the difference between the actual vapour pressure and the saturation vapour pressure that applies for the prevailing air temperature T_a, thus $(e_a - e_d)$. The *relative humidity* of the air is the ratio of actual and saturation vapour pressure, or when expressed as percentage:

$$RH = \frac{e_d}{e_a} 100 \qquad \textit{Equation 1.1}$$

1.1.2 Formation of precipitation

The conditions for precipitation to take place may be summarized stepwise as follows:

- a supply of moisture
- b cooling to below point of condensation
- c condensation
- d growth of particles

The supply of moisture is obtained through evaporation from wet surfaces, transpiration from vegetation or transport from elsewhere. The cooling of moist air may be through contact with a cold earth surface causing dew, white frost, mist or fog, and loss of heat through long wave radiation (fog patches). However, much more important is the lifting of air masses under adiabatic conditions (dynamic cooling) causing a fall of temperature to near its dew point.

Five lifting mechanisms can be distinguished:

1. *Convection*, due to vertical instability of the air. The air is said to be unstable if the temperature gradient is larger than the adiabatic lapse rate. Consequently a parcel moving up obtains a temperature higher than its immediate surroundings. Since the pressure on both is the same the density of the parcel becomes less than the environment and buoyancy causes the parcel to ascend rapidly. Instability of the atmosphere usually results from the heating of the lower air layers by a hot earth surface and the cooling of the upper layers by outgoing radiation. Convective rainfall is common in tropical regions and it usually appears as a thunderstorm in temperate climates during the summer period. Rainfall intensities of convective storms can be very high locally; the duration, however, is generally short.

2. *Orographic lifting*. When air passes over a mountain it is forced to rise which may cause rainfall on the windward slope. As a result of orographic lifting rainfall amounts are usually highest in the mountainous part of the river basin.
3. *Frontal lifting*. The existence of an area with low pressure causes surrounding air to move into the depression, displacing low pressure air upwards, which may then be cooled to dew point. If cold air is replaced by warm air (warm front) the frontal zone is usually large and the rainfall of low intensity and long duration. A cold front shows a much steeper slope of the interface of warm and cold air usually resulting in rainfall of shorter duration and higher intensity (see Figure 1.2). Some depressions are died-out cyclones.

4. *Cyclones, tropical depressions or hurricanes*. These are active depressions which gain energy while moving over warm ocean water and which dissipate energy while moving over land or cold water. They may cause torrential rains and heavy storms. Typical characteristics of these tropical depressions are high intensity rainfall of long duration (several days). Notorious tropical depressions occur in the Caribbean (hurricanes), the Bay of Bengal (monsoon depressions), the Far East (typhoons), Southern Africa (cyclones), and on the islands of the Pacific (cyclones, willy-willies). This group of depressions is quite different in character from other lifting mechanisms; data on extreme rainfall originating from cyclones should be treated separately from other rainfall data, as they belong to a different statistical population. One way of dealing with cyclones is through mixed distributions (see section 1.2.2).

5. *Convergence*. The Inter-Tropical Convergence Zone is the tropical region where the air masses originating from the Tropics of Cancer and Capricorn converge and lift. In the tropics, the position of the ITCZ governs the occurrence of wet and dry seasons. This convergence zone moves with the seasons. In July, the ITCZ lies to the North of the equator and in January it lies to the South (see Figure 1.3). In the tropics the position of the ITCZ determines the main rain-bringing mechanism which is also called monsoon. Hence, the ITCZ is also called the Monsoon Trough, particularly in Asia. In certain places near to the equator, such as on the coast of Nigeria, the ITCZ passes two times per year, causing two wet seasons; near the Tropics of Capricorn and Cancer (e.g. in the Sahel), however, there is generally only one dry and one wet season.

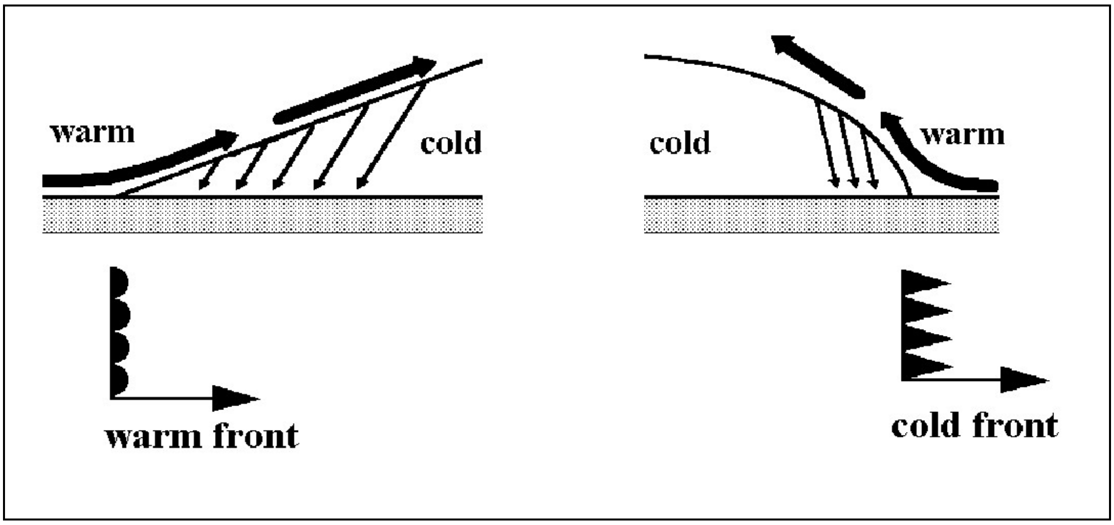


Figure 1.2: Frontal lifting

Condensation of water vapour into small droplets does not occur immediately when the air becomes saturated. It requires small airborne particles called aerosols which act as nuclei for water vapour to condense. Nuclei are salt crystals from the oceans, combustion products, dust, ash, etc. In the absence of sufficient nuclei the air becomes supersaturated. When the temperature drops below zero, freezing nuclei with a structure similar to ice are needed for condensation of water vapour into ice crystals. These nuclei are often not available and the droplets become super-cooled to a temperature of -40 °C.

Cloud droplets with a diameter of 0.01 mm need an up draught of only 0.01 m/s to prevent the droplet from falling. A growth of particles is necessary to produce precipitation. This growth may be achieved through coalescence, resulting from collisions of cloud droplets. Small droplets moving upwards through the clouds collide and unite with larger droplets which have a different velocity or move downwards. Another method of growth known as the Bergeron process is common in mixed clouds which consist of super-cooled water droplets and ice crystals. Water vapour condenses on the ice crystals, the deficit being replenished by the evaporation of numerous droplets. In the temperature range from -12 °C to -30 °C the ice crystals grow rapidly and fall through the clouds. Aggregates of many ice crystals may form snow flakes. Depending on the temperature near the surface they may reach the ground as rain, snow, hail or glazed frost.

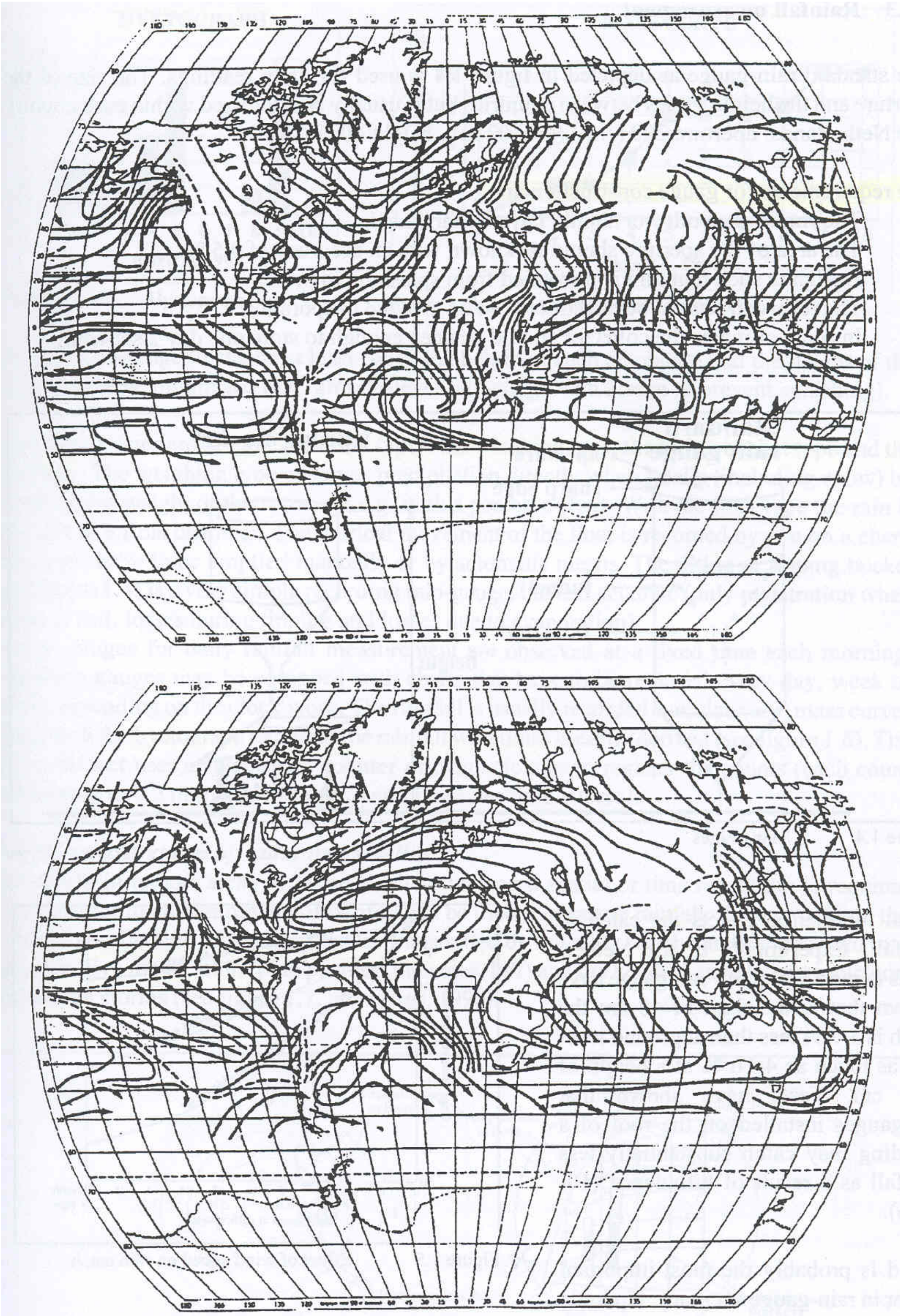


Figure 1.3: Position of the Inter-Tropical Convergence Zone in January and July

1.1.3 Rainfall measurements

The standard rain gauge as depicted in Figure 1.4 is used for daily readings. The size of the aperture and the height varies between countries but is usually standardized within each country (the Netherlands: aperture 200 $(cm)^2$ (or 0.02 m^2), height 40 cm).

The requirements for gauge construction are:

1. The rim of the collector should have a sharp edge.
2. The area of the aperture should be known with an accuracy of 0.5 %.
3. Design is such that rain is prevented from splashing in or out.
4. The reservoir should be constructed so as to avoid evaporation.
5. In some climates the collector should be deep enough to store one day's snowfall

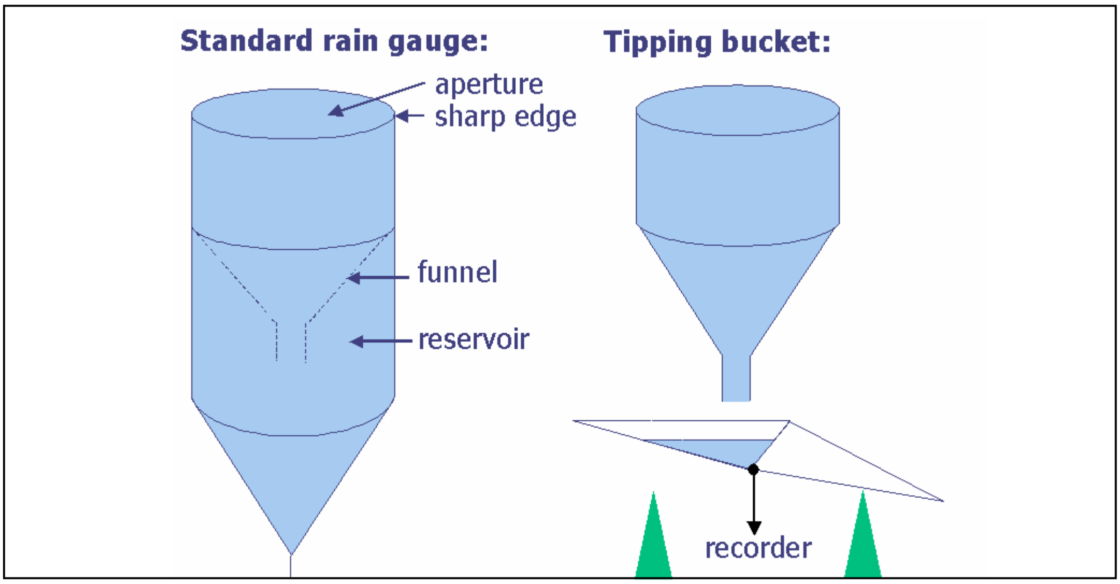


Figure 1.4: Rain gauges

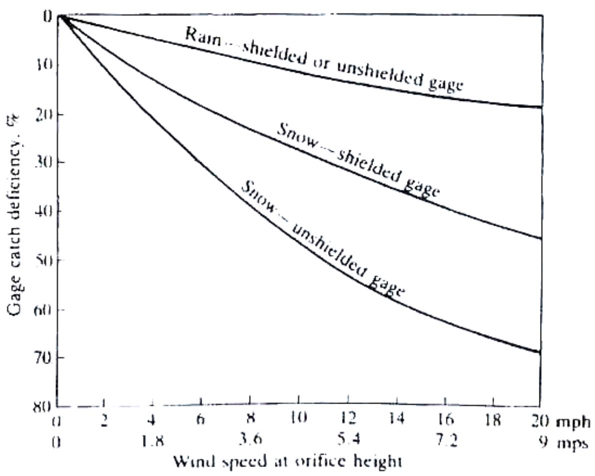


Figure 1.5: Effect of wind speed on rain catch

Wind turbulence affects the catch of rainfall. Experiments in the Netherlands using a 400 $(cm)^2$ rain gauge have shown that at a height of 40 cm the catch is 3-7% less than at ground level and as much as 4-16% at a height of 150 cm. Tests have shown that rain gauges installed on the roof of a building may catch substantially less rainfall as a result of turbulence (10-20%).

Wind is probably the most important factor in rain-gauge accuracy. Updrafts resulting from air moving up and round the instrument reduce the rainfall catch. Figure 1.5 shows the effect of wind speed on the catch according to Larson & Peck (1974). To reduce the effects of wind, rain gauges can be provided with windshields. Moreover, obstacles should be kept far from the rain gauge (distance at least twice the height of such an object) and the height of the gauge should be minimized (e.g. ground-level rain-gauge with screen to prevent splashing).

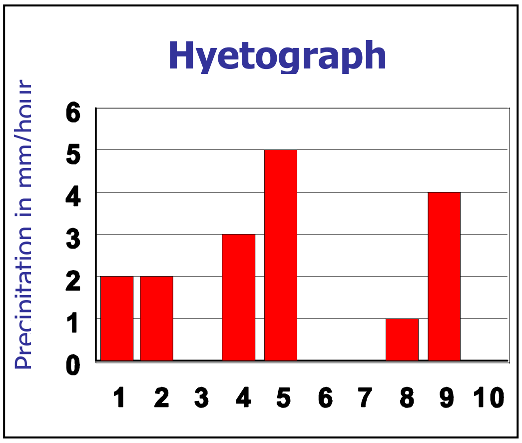


Figure 1.6: Hyetograph and corresponding mass curve

Recording gauges are of three different types: the weighing type, the tilting bucket type and the float type. The weighing type observes precipitation directly when it falls (including snow) by recording the weight of the reservoir e.g. with a pen on a chart. With the float type the rain is collected in a float chamber. The vertical movement of the float is recorded by pen on a chart. Both types have to be emptied manually or by automatic means. The tilting or tipping bucket type (Figure 1.4) is a very simple recording rain gauge, but less accurate (only registration when bucket is full, losses during tipping and losses due to evaporation).

Storage gauges for daily rainfall measurement are observed at a fixed time each morning. Recording gauges may be equipped with charts that have to be replaced every day, week or month, depending on the clockwork. The rainfall is usually recorded cumulatively (mass curve) from which the hyetograph (a plot of the rainfall with time) is easily derived (see Figure 1.6). The tipping bucket uses an electronic counter or magnetic tape to register the counts (each count corresponds to 0.2 mm), for instance, per 15 minutes time interval.

Rainfall measurement by radar and satellite

Particularly in remote areas or in areas where increased spatial or time resolution is required radar and satellite measurement of rainfall can be used to measure rainfall. Radar works on the basis of the reflection of an energy pulse transmitted by the radar which can be elaborated into maps that give the location (plan position indicator, PPI) and the height (range height indicator, RHI) of the storms (see Figure 1.7, after Bras, 1990).

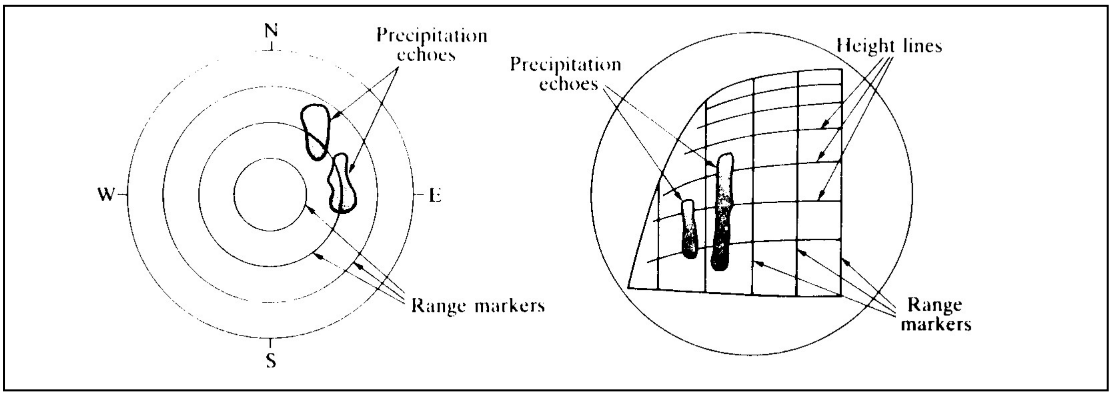


Figure 1.7: Forms of radar display: PPI (left) and RHI (right)

A technology which has a large future is rainfall monitoring through remote sensing by satellite. Geostationary satellites, that orbit at the same velocity as the earth's rotation, are able to produce a film of weather development with a time interval

between observations of several minutes. The resolution of the images is in the order of one km. This allows monitoring closely the development of convective storms, depressions, fronts, orographic effects and tropical cyclones.
Successful efforts have been made to correlate rainfall with Cold Cloud Coverage (CCC) and Cold Cloud Duration (CCD) through simple mathematical regression. Particularly in remote areas the benefits of such methods are obvious.

Parameters defining rainfall

When discussing rainfall data the following elements are of importance:

1. Intensity or rate of precipitation: the depth of water per unit of time in m/s, mm/min, or inches/hour (see section 1.1.4).
2. Duration of precipitation in seconds, minutes or hours (see section 1.1.4).
3. Depth of precipitation expressed as the thickness of a water layer on the surface in mm or inches (see section 1.1.4).
4. Area, that is the geographic extent of the rainfall in $(km)^2$ or Mm^2 (see section 1.1.5).
5. Frequency of occurrence, usually expressed by the 'return period' e.g. once in 10 years (see section 1.2.1).

1.1.4 Intensity, duration and rainfall depth

We have seen that rainfall storms can be distinguished by their meteorological characteristics (convective, orographic, frontal, cyclonic). Another way to characterize storms is from a statistical point of view. Distinction is made between "interior" and "exterior" statistics:

- exterior statistics refer to total depth of the storm, duration of the storm, average intensity of the storm and time between storms;
- interior statistics refer to the time and spatial distribution of rainfall rate within storms.

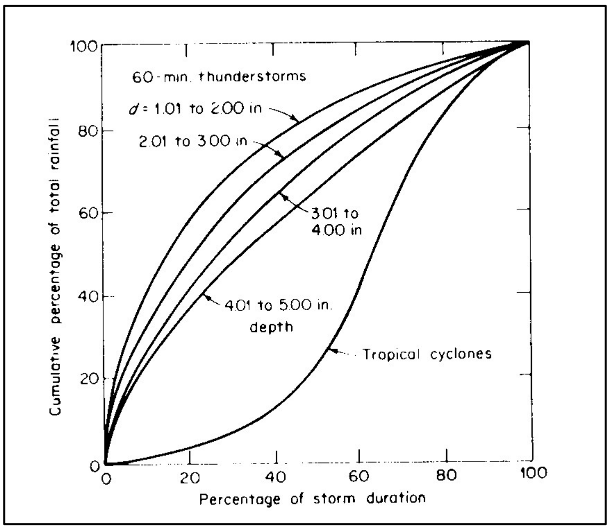


Figure 1.8: Typical percentage mass curves of rainfall for thunderstorms and cyclones

The exterior statistics can generally be described by probabilistic distributions, which can be seasonally and spatially dependent. They are generally not statistically independent. Depth is related to duration: a long duration is associated with a large depth; average intensity is related to short duration. Moreover there is spatial correlation between points.

With regard to storm interior, Eagleson (1970) observed that for given locations and climatic conditions some type of storms gave similar histories of rainfall accumulation. The percentage-mass curve of Figure 1.8 shows how typical curves are obtained for thunderstorms (convective storms) and cyclonic storms. The derivative (slope) of these curves is a graph for the rainfall intensity over time. Such a graph is referred to as the hyetograph and is usually presented in histogram form (see Figure 1.6). It can be seen from Figure 1.8 that convective storms have more or less triangular (mound like) hyetographs with the highest intensity at the beginning of the storm (a steep start), whereas tropical cyclones have a bell-shaped hyetograph with the largest intensity in the middle of the storm. It should be observed that tropical cyclones have a much longer storm duration and a much larger rainfall depth than thunderstorms.

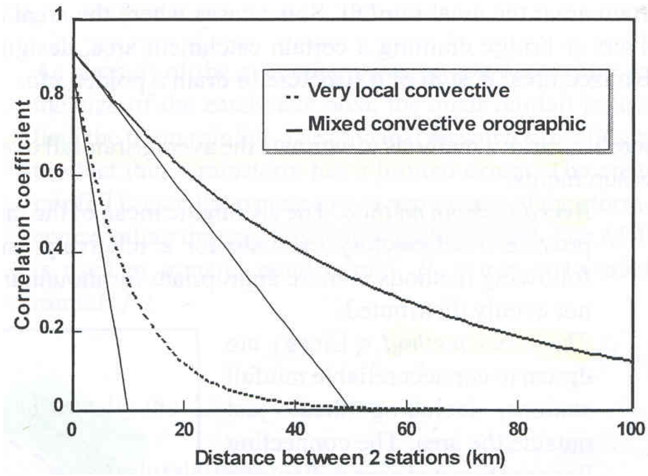


Figure 1.9: Spatial correlation of daily rainfalls

Another interior characteristic of a storm is the spatial distribution. The spatial distribution of a storm generally concentrates around one or two centres of maximum depth. The total depth of point rainfall distributed over a given area is a decreasing function of the distance from the storm centre. One can draw lines of equal rainfall depth around the centres of maximum depth (isohyets).

When two rainfall stations are closely together, data from these stations may show a good correlation. The further the stations lay apart, the smaller the chances of coincidence become. In general, correlation is better when the period of observation is larger. For a given period of observation, the correlation between two stations is defined by the correlation coefficient Δ (–1<Δ<1). The square of Δ is known as the coefficient of determination (Δ^2). If there is no correlation, Δ is near to zero, if the correlation is perfect, Δ=1. It is defined by:

$$\rho = \frac{\text{cov}(x,y)}{\sigma(x)\sigma(y)} \qquad \textit{Equation 1.2}$$

Table 1.2: Typical spatial correlation structure for different storm types

Rain type	Period 1 hour		Period 1 day		Period 1 month	
	r_0 (km)	ρ_0	r_0 (km)	ρ_0	r_0 (km)	ρ_0
Very local convec-tive	5	0.8	10	0.88	50	0.95
Mixed convec-tive orogra-phic	20	0.85	50	0.92	1500	0.98
Frontal rains from depres-sions	100	0.95	1000	0.98	5000	0.99

For a number of stations in a certain area the correlation coefficient can be determined pair-wise. In general, the larger the distance, the smaller the correlation will be. If *r* is defined as the distance between stations, then the correlation between stations at a distance *r* apart is often described by Kagan's formula:

$$\rho(r) = \rho_0 \exp(-r/r_0) \qquad \textit{Equation 1.3}$$

This is an exponential function which equals Δ_0 at *r=0* and which decreases gradually as *r* goes to infinity. The distance r_0 is defined as the distance where the tangent at *x=0* intersects the *x*-axis (see Figure 1.9). For convective storms, the steepness is very large and the value of r_0 is small; for frontal or orographic rains, the value is larger. Also the period considered is reflected in the steepness; for a short period of observation r_0 is small. Table 1.2 gives indicative values of r_0 and Δ_0.

1.1.5 Areal rainfall

In general, for engineering purposes, knowledge is required of the average rainfall depth over a certain area: the areal rainfall. Some cases where the areal rainfall is required are: design of a culvert or bridge draining a certain catchment area, design of a pumping station to drain an urbanized area; design of a structure to drain a polder, etc.

There are various methods to estimate the average rainfall over an area (areal rainfall) from point-measurements.

1. *Average depth method*. The arithmetic mean of the rainfall amounts measured in the area provides a satisfactory estimate for a relatively uniform rain. However, one of the following methods is more appropriate for mountainous areas or if the rain gauges are not evenly distributed.

2. *Thiessen method*. Lines are drawn to connect reliable rainfall stations, including those just outside the area. The connecting lines are bisected perpendicularly to form a polygon around each station (see Figure 1.10). To determine the mean, the rainfall amount of each station is multiplied by the area of its polygon and the sum of the products is divided by the total area.

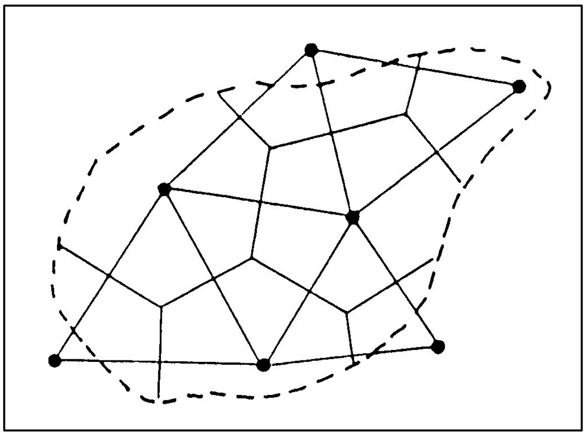


Figure 1.10: Thiessen polygons

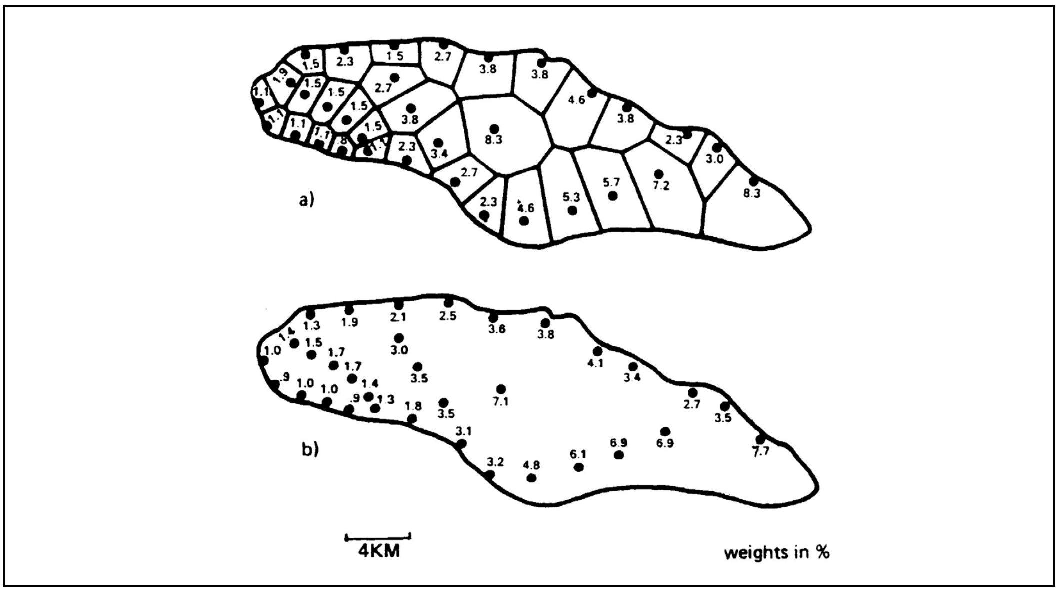


Figure 1.11: Comparison of Thiessen's and Kriging method (after Delhomme, 1978)

3. *Kriging*. D.G. Krige, a mining expert, developed a method for interpolation and averaging of spatially varying information, which takes account of the spatial

variability and which – unlike other methods – can also indicate the level of accuracy of the estimates made. The Kriging weights obtained are tailored to the variability of the phenomenon studied. Figure 1.11 shows the comparison between weights obtained through Thiessen's method (a) and Kriging (b).

4. *Isohyetal method*. Rainfall observations for the considered period are plotted on the map and contours of equal precipitation depth (isohyets) are drawn (Figure 1.12). The areal rainfall is determined by measuring the area between isohyets, multiplying this by the average precipitation between isohyets, and then by dividing the sum of these products by the total area.

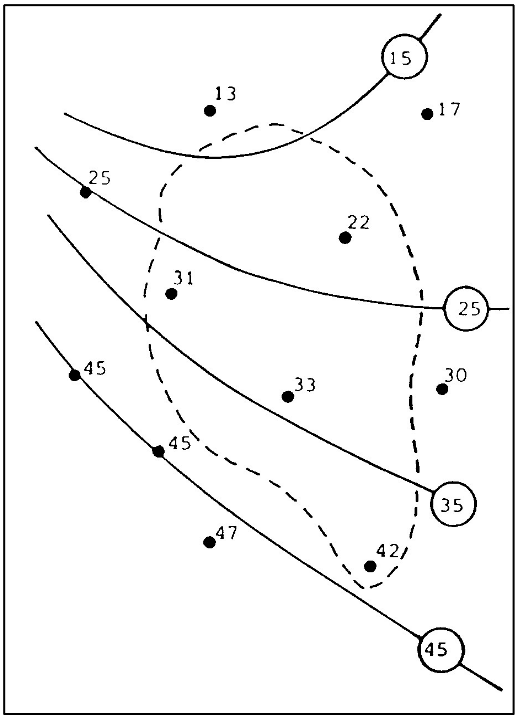


Figure 1.12: Isohyetal method

As a result of the averaging process, and depending on the size of the catchment area, the areal rainfall is less than the point rainfall. The physical reason for this lies in the fact that a rainstorm has a limited extent. The areal rainfall is usually expressed as a percentage of the storm-centre value: the *areal reduction factor* (ARF). The ARF is used to transfer point rainfall P_p extremes to areal rainfall P_a:

$$\mathrm{ARF} = P_a / P_p \qquad \textit{Equation 1.4}$$

Basically the ARF is a function of:

- rainfall depth
- storm duration
- storm type
- catchment size
- return period (see section 1.2.1)

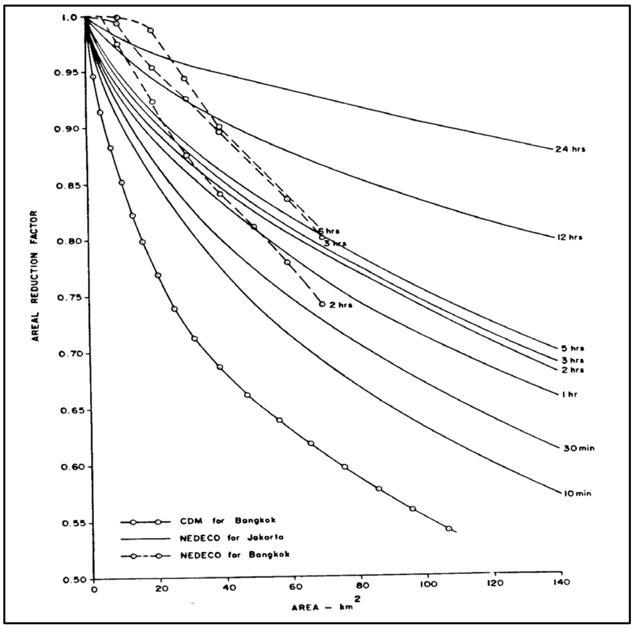


Figure 1.13: The areal reduction factor as a function of drainage area and duration (after NEDECO et al., 1983)

The ARF increases (comes nearer to unity) with increasing total rainfall depth, which implies higher uniformity of heavy storms. It also increases with increasing duration, again implying that long storms are more uniform. It decreases with the area under consideration, as a result of the storm-centred approach.

Storm type varies with location, season and climatic region. Published ARF's are, therefore, certainly not generally applicable. From the characteristics of storm types, however, certain conclusions can be drawn.

A convective storm has a short duration and a small areal extent; hence, the ARF decreases steeply with distance. A frontal storm has a long duration and a much larger area of influence; the ARF, hence, is expected to decrease more slowly with distance. The same applies to orographic lifting. Cyclones also have long durations and a large areal extent, which also leads to a more gradual reduction of the ARF than in the case of thunderstorms. In general, one can say that the ARF-curve is steepest for a convective storm, that a cyclonic storm has a more moderate slope and that orographic storms have an even more moderate slope.

The functional relationship between the ARF and return period is less clear. Bell (1976) showed for the United Kingdom that ARF decreased more steeply for rainstorms with a high return period. Similar findings are reported by Begemann (1931) for Indonesia. This is, however, not necessarily so in all cases. If widespread cyclonic disturbances, instead of more local convective storms, constitute the high return period rainfall, the opposite may be true.

Again, it should be observed that cyclones belong to a different statistical population from other storm types, and that they should be treated separately. If cyclones influence the design criteria of an engineering work, then one should consider a high value of the ARF.

Figure 1.13, as an example, shows ARF's as a function of catchment size and rainfall duration for Bangkok by Nedeco (1983), and Camp, Dresser and McKee (1968), and for Jakarta by Nedeco (1973).

1.1.6 Rainfall data screening

Of all hydrological data, rainfall is most readily available. Though quantitatively abundant its quality should not, a priori, be taken for granted, despite the fact that precipitation data are easily obtained. Several ways of specific rainfall data screening are available.

daily rainfall

- tabular comparison, maximum values check
- time series plotting and comparison
- spatial homogeneity test

monthly rainfall

- tabular comparison, maximum, minimum, P_{80}, P_{20} check
- time series plotting
- spatial homogeneity test
- double mass analysis

Tabular comparison

In a table a number of checks can be carried out to screen rainfall data. One way is through internal operations and another through comparison with other tables. Internal operations are the computation of the maximum, minimum, mean and standard deviation for columns or ranges in the table. Spreadsheet programs are excellent tools for this purpose. A table of daily rainfall organised in monthly columns can be used to compute the monthly sums, the annual total, and the annual maximum. Comparison of these values with those of a nearby station could lead to flagging certain information as doubtful. A table showing monthly totals for different years can be used for statistical analysis. In many cases the normal or lognormal distribution fits well to monthly rainfall. In such a case the computation of the mean and standard deviation per month (per column) can be used to compute the rainfall with a probability of non-exceedence of 20% (a dry year value) or 80% (a wet year value) through the following simple formula:

$$P_{20} = Mean - 0.84 * Std$$
$$P_{80} = Mean + 0.84 * Std$$

If the lognormal distribution performs better (as one would expect since rainfall has a lower boundary of 0), than the values of mean and standard deviations should be based on the logarithms of the recorded rainfall: *Lmean*, and *Lstd*. The values of P_{20} and P_{80} then read:

$$P_{20} = Exp\ (Lmean - 0.84 * Lstd)$$
$$P_{80} = Exp\ (Lmean + 0.84 * Lstd)$$

If individual monthly values exceed these limits too much, they should be flagged as suspect, which does not, a priori, mean that the data are wrong; it is just a sign that one should investigate the data in more detail.

Double mass

The catch of rainfall for stations in the same climatic region is, for long-time periods, closely related. An example showing the mass curves of cumulative annual precipitation of two nearby stations *A* and *B* is given in Figure 1.14a. The double mass curve of both stations plots approximately as a straight line (see Figure 1.14b). A deviation from the original line indicates a change in observations in either station *A* or *B* (e.g. new observer, different type of rain-gauge, new site, etc.). This is called a spurious (= false) trend, not to be mistaken for a real trend, which is a gradual change of climate. If the cumulative annual rainfall data of station *X* are plotted against the mean of neighbouring stations the existence of a spurious trend indicates that the data of station *X* are inconsistent (Figure 1.15). When the cause of the discrepancy is clear, the monthly rainfall of station *X* can be corrected by a factor equal to the proportion of the angular coefficients.

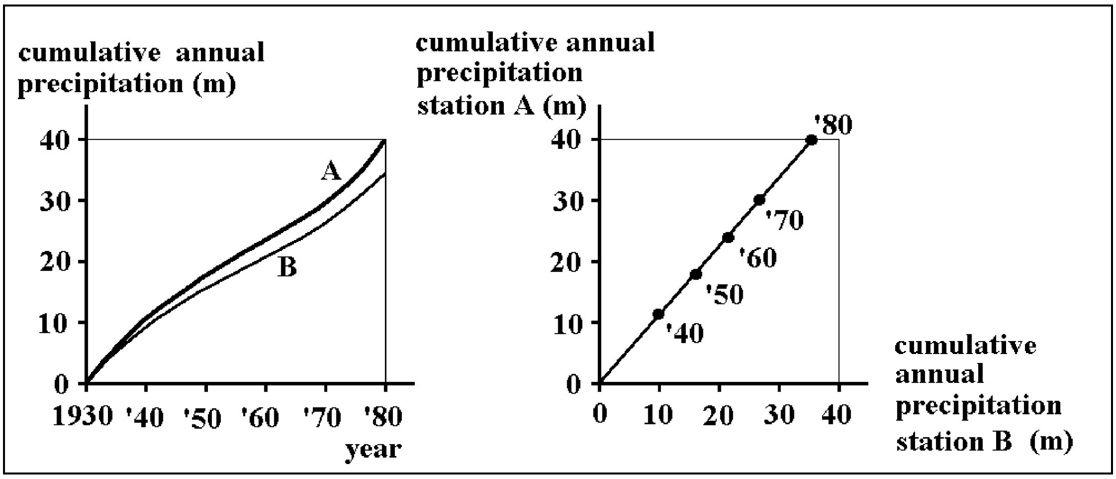


Figure 1.14: Mass curves of individual stations A and B (left) and Double mass curve of stations A and B (right)

Time series plotting

Although these tabular operations can help to identify possible sources of errors, there is nothing as powerful to pinpoint suspect data as a graphical plot. In a spreadsheet a graphical plot is easy to perform. One can use a bar-graph for a simple time plot, a stacked bar-graph to view two combined sets, or an X-Y relation to compare one station with another nearby station. The latter is a powerful tool to locate strange values.

Spatial homogeneity

In the spatial homogeneity test a base station is related to a number of surrounded stations. A maximum distance r_{max} is defined on the basis of (1.3) beyond which no significant correlation is found (e.g. Δ<0.75 or 0.5). To investigate the reliability of point rainfall, the observed rainfall is compared with an estimated rainfall depth on the basis of spatial correlation with others (computed from a weighted average through multiple regression). A formula often used for the weighted average is by attributing weights that are inversely proportional to the square of the distance from the station in question:

$$P_{est} = \frac{\sum\left(P/r^b\right)_i}{\sum\left(1/r^b\right)_i} \qquad \textit{Equation 1.5}$$

where *b* is the power for the distance, which is often taken as 2, but which should be determined from experience. If the observed and the estimated rainfall differ more than an acceptable error criterion, both in absolute and relative terms, then the value should be further scrutinized, or may have to be corrected (see exercise hydrology).

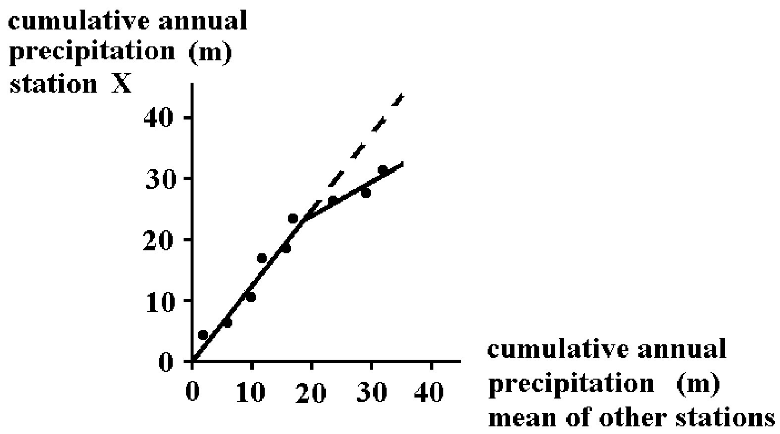


Figure 1.15: Double mass curve showing apparent trend for station X

Table 1.3: Derivation of rainfall data for duration (k) of 2, 5 and 10 days form 15 daily values

k \ #	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
1	0	0	2	8	0	0	0	0	0	12	3	8	24	2	0
2		0	2	10	8	0	0	0	0	12	15	11	32	26	2
5					10	10	10	8	0	12	15	23	47	49	37
10										22	25	33	55	49	49

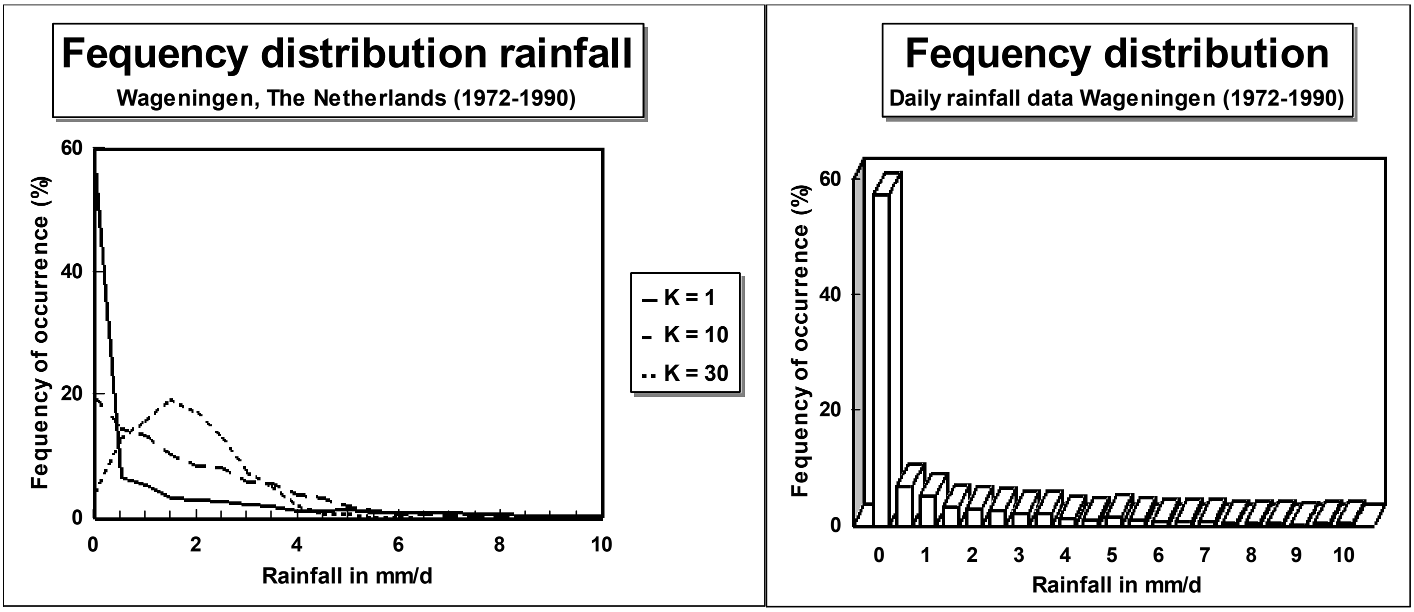


Figure 1.16: Frequency distribution of 1, 10 and 30 day rainfall (a); histogram of daily rainfall (b)

1.2 Analysis of extreme rainfall events

1.2.1 Frequency analysis

Consider daily rainfall data over a period of many years and compute the percentage of days with rainfall between 0-0.5 mm, 0.5-1.0 mm, 1.0-1.5 mm, etc. The frequency of occurrence of daily rainfall data may then be plotted. An example for a station in the Netherlands is given in figure 1.16b. It shows that the frequency distribution is extremely skew, as days without rain or very little rain occur most frequently and high rainfall amounts are scarce. The distribution becomes less skew if the considered duration (k) is taken longer. Table 1.3 gives an example of the derivation of rainfall data for longer durations (k = 2, 5 and 10 days) from daily values for a record length of 15 days. An example of frequency distributions of average rainfall data for k = 1, 10 and 30 days for a rainfall station in the Netherlands is given in figure 1.16a. For the design of drainage systems, reservoirs, hydraulic works in river valleys, irrigation schemes, etc. knowledge of the frequency of occurrence of rainfall data is often essential. The type of data required depends on the purpose; for the design of an urban drainage system rainfall intensities in the order of magnitude of mm/min are used, while for agricultural areas the frequency of occurrence of rainfall depths over a period of several days is more appropriate.

When dealing with extremes it is usually convenient to refer to probability as the *return period* that is, the average interval in years between events which equal or exceed the considered magnitude of event. If p is the probability that the event will be equalled or exceeded in a particular year, the return period T may be expressed as:

$$T = \frac{1}{p} \qquad \textit{Equation 1.6}$$

One should keep in mind that a return period of a certain event e.g. 10 years does not imply that the event occurs at 10 year intervals. It means that the probability that a certain value (e.g. a rainfall depth) is exceeded in a certain year is 10%. Consequently the probability that the event does not occur (the value is not exceeded) has a probability of 90%. The probability that the event does not occur in ten consecutive years is $(0.9)^{10}$ and thus the probability that it occurs once or more often in the ten years period is $1-(0.9)^{10} = 0.65$; i.e. more than 50%. Similarly, the probability that an event with a return period of 100 years is exceeded in 10 years is $1-(0.99)^{10} = 10\%$. This probability of 10% is the actual risk that the engineer takes if he designs a structure with a lifetime of 10 years using a design criterion of T=100. In general, the probability P that an event with a return period T actually occurs (once or more often) during an n years period is:

$$P = 1 - \left(1 - \frac{1}{T}\right)^n \qquad \textit{Equation 1.7}$$

Table 1.4: Totals of k-days periods with rainfall greater or equal than the bottom of the class interval for k = 1, 2, 5 and 10 days

Class interval (mm)	1	2	5	10
0	18262	18261	18258	18253
10	384	432	730	2001
20	48	127	421	1539
30	5	52	243	713
40	1	12	158	493
50		5	83	286
60		2	49	221
70			25	170
80			16	96
90			9	76
100			5	49
110			3	31
120			1	22
130			1	16
140				9
150				7
160				5
170				4
180				2
190				2
200				1

A frequency analysis to derive intensity-duration-frequency curves requires a length of record of at least 20 - 30 years to yield reliable results. The analysis will be explained with a numerical example, using hypothetical data. Consider a record of 50 years of daily precipitation data, thus 365.25 * 50 = 18262 values (provided there are no missing or unreliable data). Compute the number of days with rainfall events greater than or equal to 0, 10, 20, 30, etc. mm. For this example large class intervals of 10 mm are used to restrict the number of values. In practice a class interval of 0.5 mm (as in Figure 1.16b) or even smaller is more appropriate.

Table 1.4 shows that all 18262 data are greater than or equal to zero (of course), and that there is only one day on which the amount of rainfall equalled or exceeded 40 mm. Similar to the procedure explained in table 1.3, rainfall data are derived for k = 2, 5 and 10 days. For k = 2 this results in 18261 values which are processed as for the daily rainfall data, resulting in two values with 60 mm or more. The results for k = 2, 5 and 10 are also presented in the Table. The data are used to construct duration curves as shown in Figure 1.17.

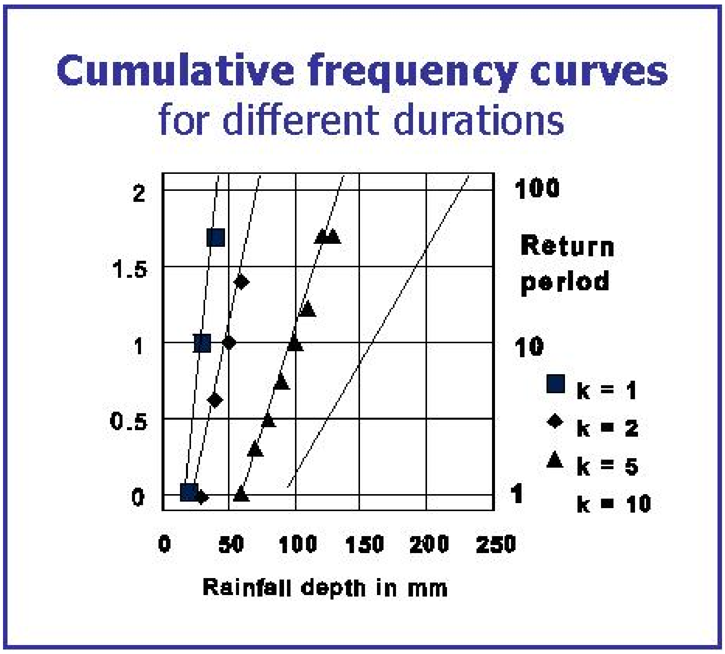


Figure 1.17: Cumulative frequency curves for different durations (k =1, 2, 5 and 10 days)

The procedure is the following. Consider the data for k = 1 which show that on one day in 50 years only the amount of rainfall equals or exceeds 40 mm, hence the probability of occurrence in any year is P =1/50 = 0.02 or 2% and the return period $T = 1/P = 50$ years. This value is plotted in Figure 1.17. Similarly for rainfall events greater than or equal to 30 mm, P = 5/50 = 0.1 or T = 10 years and for 20 mm, P = 48/50 = 0.96 or $T \approx 1$ year. Figure 1.17 shows the resulting curves for a rainfall duration of one day as well as for k = 2, 5 and 10 days.

Cumulative frequency or duration curves often approximate a straight line when plotted as in Figure 1.17 on semi-logarithmic graph paper, which facilitates extrapolation. Care should be taken with regard to extrapolation frequency estimates, in particular if the return period is larger than twice the record length.

Figure 1.17 shows that for durations of 1, 2, 5 and 10 days the rainfall events equal or exceed respectively 20, 30, 60 and 100 mm with a return period of one year. These values are plotted in Figure 1.18 to yield the depth-duration curve with a frequency of T = 1 year. Similarly the curves for T = 10 and the extrapolated curve for T = 100 years are constructed. On double logarithmic graph paper these curves often approximate a straight line. Dividing the rainfall depth by the duration yields the average intensity. This procedure is used to convert the *depth-duration-frequency curves* into *intensity-duration-frequency curves* (Figure 1.19).

For the above analysis *all* available data have been used, i.e. the *full series*. As the interest is limited to relatively rare events, the analysis could have been carried out for a *partial duration series*, i.e. those values that exceed some arbitrary level. For the analysis of extreme precipitation amounts a series which is made up of annual extremes, the *annual series*, is favoured as it provides a good theoretical basis for extrapolating the series beyond the period of observation. One could say that the partial duration series is more complete since it is made up of all the large events above a given base, thus not just the annual extreme. On the other hand, sequential peaks may not be fully independent.

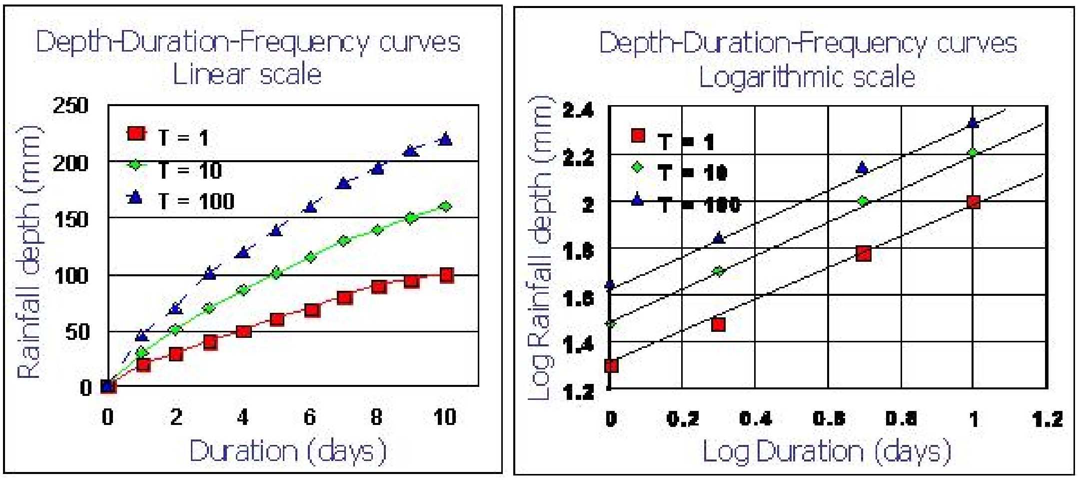


Figure 1.18: Depth-Duration-Frequency curves (left); Double logarithmic plot of DDF curves (right)

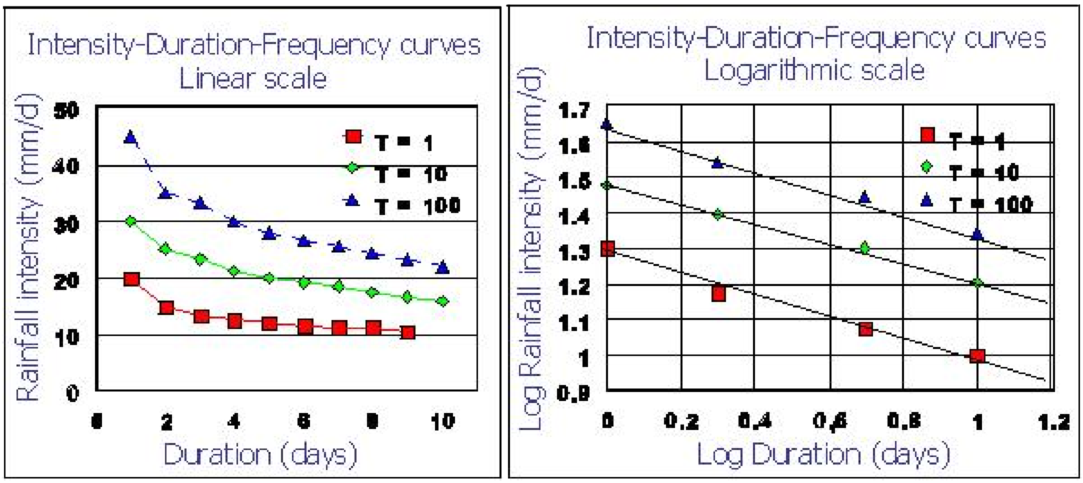


Figure 1.19: Intensity-Duration-Frequency curves (left); Double logarithmic plot of IDF curves (right)

Langbein (see Chow (1964)) developed a theory for partial duration series which considers all rainfall exceeding a certain threshold. The threshold is selected in such a way that the number of events exceeding the threshold equals the number of years under consideration. Then, according to Langbein, the relationship between the return period of the annual extremes T and return period of the partial series T_p is approximately:

$$p = \frac{1}{T} = 1 - \exp\left(-\frac{1}{T_p}\right)$$ *Equation 1.8*

Figure 1.20 shows the comparison between the frequency distribution of the extreme hourly rainfall in Bangkok computed with partial duration series (= annual exceedances) and with annual extremes. A comparison of the two series shows that they lead to the same results for larger return periods, say T > 10 years. Hershfield (1961) proposes to multiply the rainfall depth obtained by the annual extremes method by 1.13, 1.04, and 1.01 for return periods of 2, 5 and 10 years respectively.

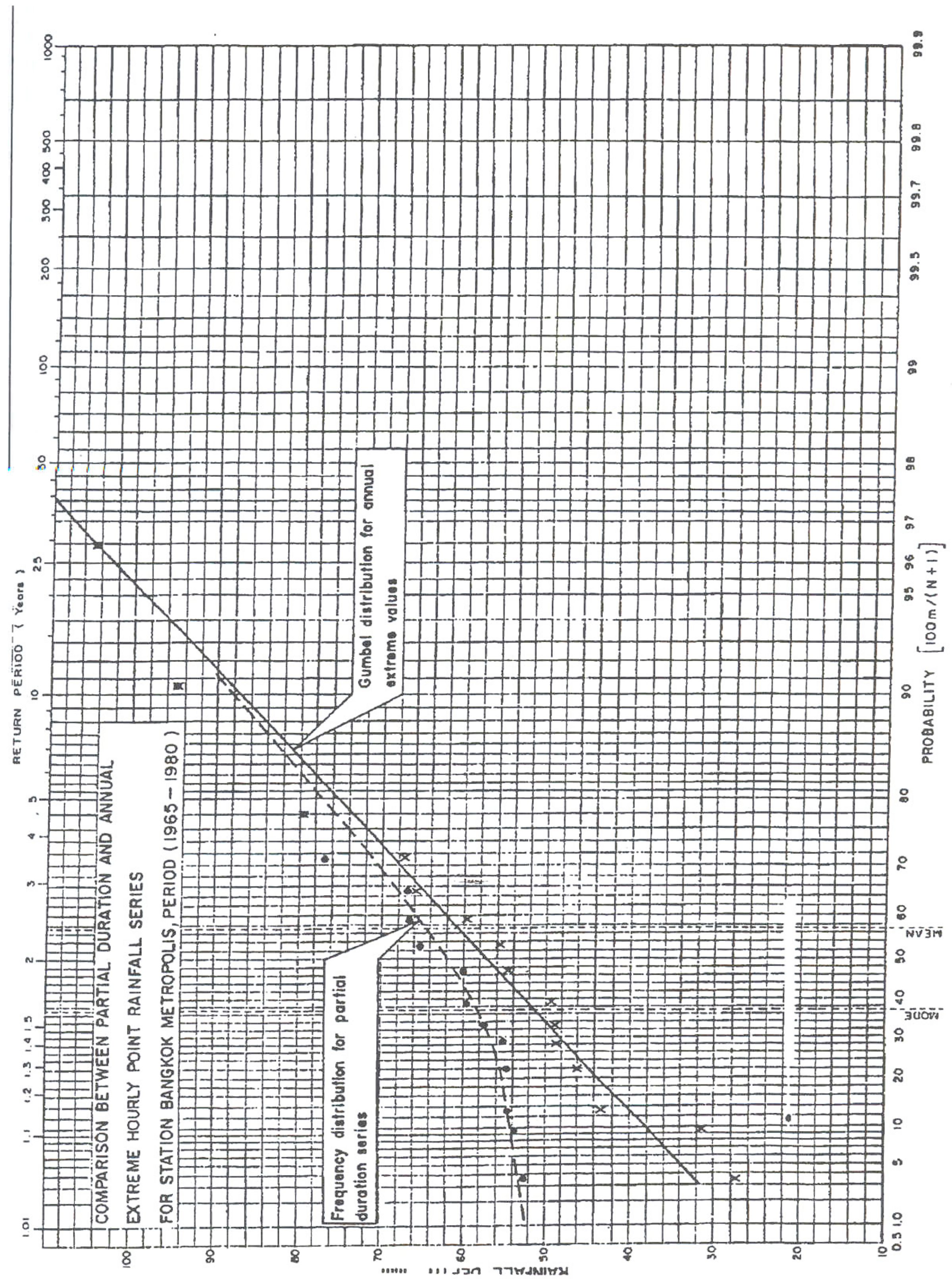


Figure 1.20: Comparison between the method of annual exceedances and annual extremes for extreme hourly rainfall in Bangkok

The analysis of annual extreme precipitation is illustrated with the following numerical example. For a period of 10 years the maximum daily precipitation in each year is listed in Table 1.5. For convenience a (too) short period of 10 years is considered. Rank the data in descending order (see Table 1.6). Compute for each year the probability of exceedance using the formula:

$$p = \frac{m}{n+1} \qquad \textit{Equation 1.9}$$

where *n* is the number of years of record and *m* is the rank number of the event. Equation 1.9 is also known as the plotting position. More accurate formulae may be used. The return period *T* is computed as *T = 1/p* and is also presented in Table 1.6. A plot of the annual extreme precipitation versus the return period on linear paper does not yield a straight line as shown in Figure 1.22. Using semi-logarithmic graph paper may improve this significantly as can be seen in Figure 1.23.

Table 1.5: Annual maximum daily rainfall amounts

Year	1971	1972	1973	1974	1975	1976	1977	1978	1979	1980
	56	52	60	70	34	30	44	48	40	38

Table 1.6: Rank, probability of exceedance, return period and reduced variate for the data in table 1.5

Rank	Rainfall amount (mm)	P Probability of exceedance	T Return period	Log T	q Probability of non-exceedance	y Reduced variate
1	70	0.09	11.0	1.041	0.91	2.351
2	60	0.18	5.5	0.740	0.82	1.606
3	56	0.27	3.7	0.564	0.73	1.144
4	52	0.36	2.8	0.439	0.64	0.794
5	48	0.45	2.2	0.342	0.55	0.501
6	44	0.54	1.8	0.263	0.46	0.238
7	40	0.64	1.6	0.196	0.36	-0.012
8	38	0.73	1.4	0.138	0.27	-0.262
9	34	0.82	1.2	0.087	0.18	-0.533
10	30	0.91	1.1	0.041	0.09	-0.875

Annual rainfall extremes tend to plot as a straight line on extreme-value-probability (Gumbel) paper (see Figure 1.21). The extreme value theory of Gumbel is only applicable to annual extremes. The method uses, in contrast to the previous example, the probability of non-exceedance *q = 1 - p* (the probability that the annual maximum daily rainfall is less than a certain magnitude). The values are listed in Table 1.6.

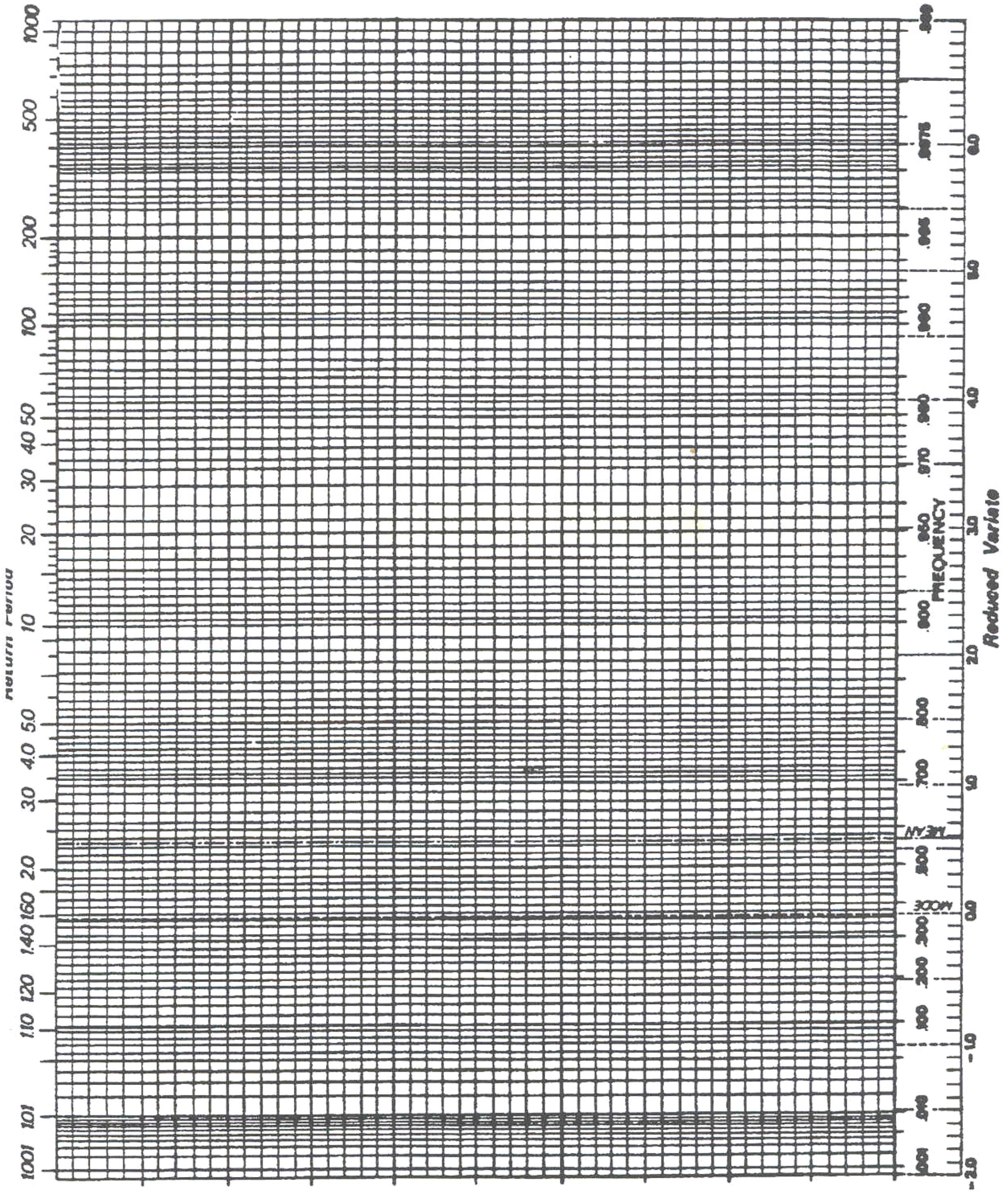


Figure 1.21: Gumbel paper

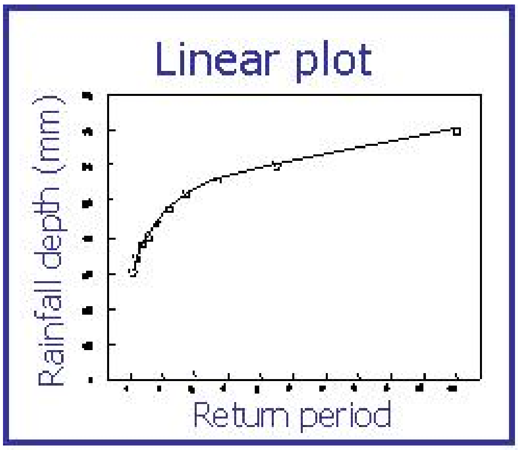


Figure 1.22: Annual maximum daily rainfall (linear plot)

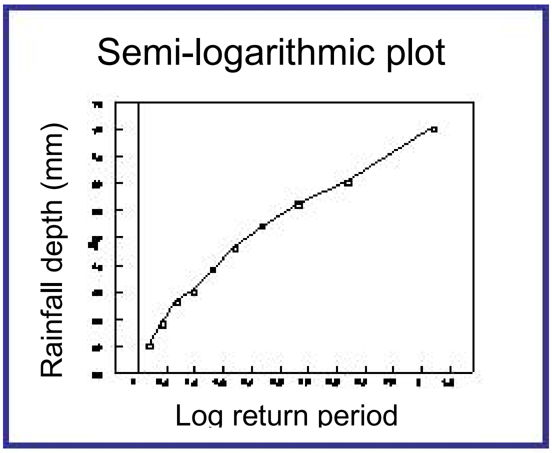


Figure 1.23: Annual maximum daily rainfall (semi-log plot)

Gumbel makes use of a reduced variate y as a function of q, which allows the plotting of the distribution as a linear function between y and X (the rainfall depth in this case).

$$y = a\left(X - b\right) \qquad \textit{Equation 1.10}$$

The equation for the reduced variate reads:

$$y = -\ln\left(-\ln\left(q\right)\right) = -\ln\left(-\ln\left(1-p\right)\right) \qquad \textit{Equation 1.11}$$

meaning that the probability of non-exceedance equals:

$$P\left(X \le X_0\right) = q = \exp\left(-\exp\left(-y\right)\right) \qquad \textit{Equation 1.12}$$

The computed values of y for the data in Table 1.5 are presented in Table 1.6. A linear plot of these data on extreme-value-probability paper (see Figure 1.24) is an indication that the frequency distribution fits the extreme value theory of Gumbel. This procedure may also be applied to river flow data. In addition to the analysis of maximum extreme events, there also is a need to analyze minimum extreme events; e.g. the occurrence of droughts. The probability distribution of Gumbel, similarly to the Gaussian probability distribution, does not have a lower limit; meaning that negative values of events may occur. As rainfall or river flow do have a lower limit of zero, neither the Gumbel nor Gaussian distribution is an appropriate tools to analyse minimum values. Because the logarithmic function has a lower limit of zero, it is often useful to first transform the series to its logarithmic value before applying the theory. Appropriate tools for analysing minimum flows or rainfall amounts are the Log-Normal, Log-Gumbel, or Log-Pearson distributions.

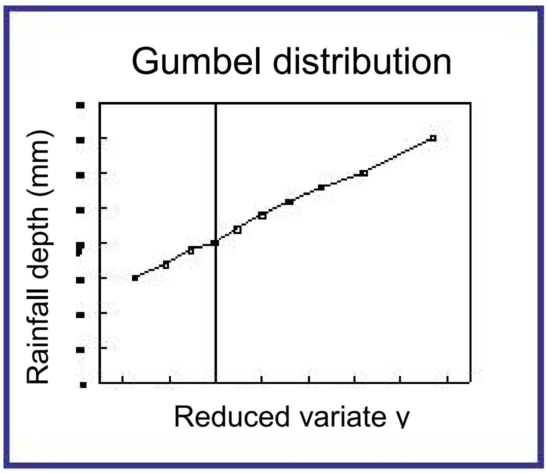


Figure 1.24: Annual maximum dialy rainfall (Gumbel distributions)

A final remark of caution should be made with regard to frequency analysis. None of the above mentioned frequency distributions has a real physical background. The only information having physical meaning is the measurements themselves. Extrapolation beyond the period of observation is dangerous. It requires a good engineer to judge the value of extrapolated events of high return periods. A good impression of the relativity of frequency analysis can be acquired through the comparison of results obtained from different statistical methods. Generally they differ considerably. And finally, in those tropical areas where cyclonic disturbances occur, one should not be misled by a set of data in which the extreme event has not yet occurred at full force. The possibility always exists that the cyclone passes right over the centre of the study area. If that should occur, things may happen that go far beyond the hitherto registered events.

1.2.2 Mixed distributions

There are four types of lifting mechanisms that cause quite distinct rainfall types with regard to depth, duration and intensity. Of these, the tropical cyclones, or a combination of tropical depressions with orographic effects are the ones which, by far, exceed other lifting mechanisms with regard to rainfall depth and intensity. As a result, such occurrences, which are generally rare, should not be analysed as if they were part of the total population of rainfall events; they belong to a different statistical population. However, our rainfall records contain events of both populations. Hence, we need a type of statistics that describes the occurrence of extreme rainfall on the basis of a mixed distribution of two populations: say the population of cyclones and the population of non-cyclones, or in the absence of cyclones, the population of orographic lifting and the population of other storms.

Such a type of statistics is presented in this section. Assume that the rainfall occurrences can be grouped in two sub-sets: the set of tropical cyclone events which led to an annual maximum daily rainfall *C* and the set of non-cyclone related storms that led to an annual extreme daily rainfall *T*. Given a list of annual maximum rainfall events, much in the same way as one would prepare for a Gumbel analysis, one indicates whether the event belonged to subset *C* or *T* (see Table 1.7). The probability $P(C)$ is defined as the number of times in *n* years of records that a cyclone occurred. In this case $P(C)$ = 4/33. The probability $P(T)$, similarly, equals 29/33. The combined occurrence $P(X>X_0)$, that the stochastic representing annual rainfall *X* is larger than a certain value X_0 is computed from:

$$P\left(X > X_0\right) = P\left(X > X_0 \middle| C\right) \cdot P\left(C\right) + P\left(X > X_0 \middle| T\right) \cdot P\left(T\right) \qquad \textit{Equation 1.13}$$

where: $P(X>X_0 \mid C)$ is the conditional probability that the rainfall is more than X_0 given that the rainfall event was a cyclone;
$P(X>X_0 \mid T)$ is the conditional probability that the rainfall is more than X_0 given that the rainfall event was a thunder storm.

In Figure 1.25, representing the log-normal distribution, three lines are distinguished. One straight line representing the log-normal distribution of thunderstorms, one straight line the log-normal distribution of the cyclones, and one curve which goes asymptotically from the thunderstorm distribution to the cyclone distribution representing the mixed distribution. The data plots can be seen to fit well to the mixed distribution curve.

Table 1.7: Maximum annual precipitation as a result of cyclonic storms and thunderstorms

Year	Max. Daily Rainfall	Thunderstorm (T) or Cyclone (C)
1952-1953	155.3	T
1953-1954	87.1	T
1954-1955	103.8	T
1955-1956	150.6	T
1956-1957	55.3	T
1957-1958	46.9	T
1958-1959	70.3	T
1959-1960	54.2	T
1960-1961	71.2	T
1961-1962	75.0	T
1962-1963	108.2	T
1963-1964	61.2	T
1964-1965	50.5	T
1965-1966	228.6	C (Claud)
1966-1967	90.0	T
1967-1968	64.6	T
1968-1969	83.3	T
1969-1970	71.5	T
1970-1971	51.3	T
1971-1972	104.4	T
1972-1973	105.4	T
1973-1974	103.5	T
1974-1975	92.0	T
1975-1976	189.3	C (Danae)
1976-1977	130.0	C (Emilie)
1977-1978	71.8	T
1978-1979	84.6	T
1979-1980	77.7	T
1980-1981	97.1	T
1981-1982	53.5	T
1982-1983	56.9	T
1983-1984	107.0	C (Demoina)
1984-1985	89.2	T

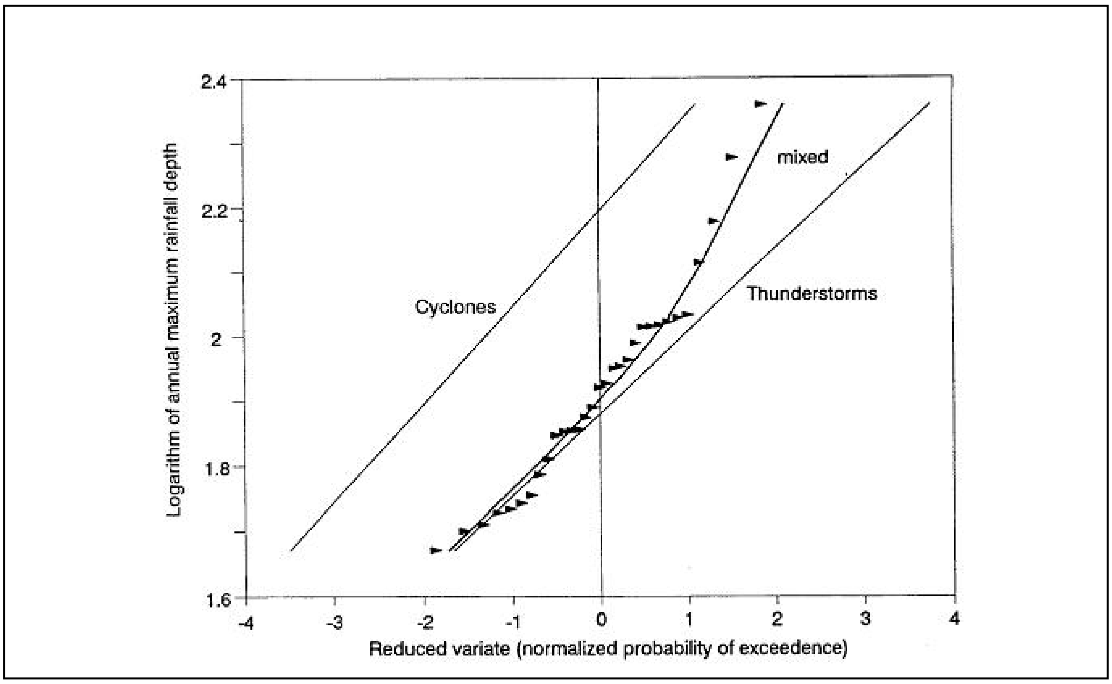


Figure 1.25: Mixed distribution of combined cyclonic storms and thunderstorms

1.2.3 Probable maximum precipitation

In view of the uncertainty involved in frequency analysis, and its requirement for long series of observations which are often not available, hydrologists have looked for other methods to arrive at extreme values for precipitation. One of the most commonly used methods is the method of the Probable Maximum Precipitation (PMP).

The idea behind the PMP method is that there must be a physical upper limit to the amount of precipitation that can fall on a given area in a given time. An accurate estimate is both desirable from an academic point of view and virtually essential for a range of engineering design purposes, yet it has proven very difficult to estimate such a value accurately. Hence the word probable in PMP; the word probable is intended to emphasize that, due to inadequate understanding of the physics of atmospheric processes, it is impossible to define with certainty an absolute maximum precipitation. It is not intended to indicate a particular level of statistical probability or return period.

The PMP technique involves the estimation of the maximum limit on the humidity concentration in the air that flows into the space above a basin, the maximum limit to the rate at which wind may carry the humid air into the basin and the maximum limit on the fraction of the inflowing water vapour that can be precipitated. PMP estimates in areas of limited orographic control are normally prepared by the maximization and transposition of real, observed storms while in areas in which there are strong orographic controls on the amount and distribution of precipitation, storm models have been used for the maximization procedure for long-duration storms over large basins.

The maximization-transposition technique requires a large amount of data, particularly volumetric rainfall data. In the absence of suitable data it may be necessary to transpose storms over very large distances despite the considerable uncertainties involved. In this case reference to published worldwide maximum observed point rainfalls will normally be helpful. The world envelope curves for data recorded prior to 1948 and 1967 are shown in Figure 1.26, together with maximum recorded falls in the United Kingdom (from: Ward & Robinson, 1990). By comparison to the world maxima, the British falls are rather small, as it is to be expected that a temperate climate area would experience less intense falls than tropical zones subject to hurricanes or the monsoons of southern Asia. The values of Cherrapunji in India are from the foothills of the Himalayas mountains, where warm moist air under the influence of monsoon depressions are forced upward resulting in extremely heavy rainfall. Much the same happens in La Reunion where moist air is forced over a 3000 m high mountain range. Obviously, for areas of less rugged topography and cooler climate, lower values of PMP are to be expected.

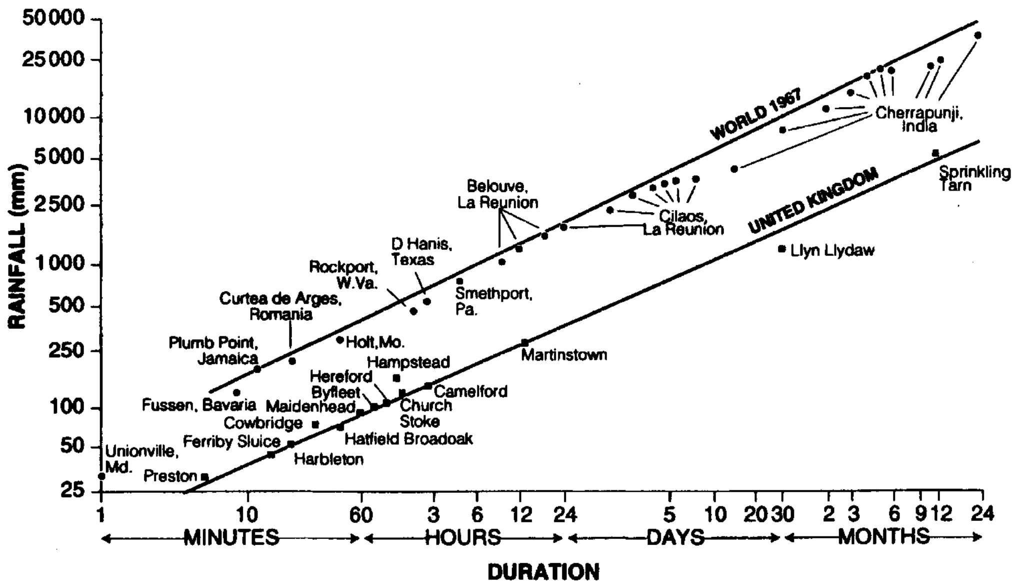


Figure 1.26: Magnitude-duration relationship for the world and the UK extreme rainfalls (source: Ward & Robinson, 1990)

1.2.4 Analysis of dry spells

The previous sections dealt with extremely high, more or less instantaneous, rainfall. The opposite, extremely low rainfall, is not so interesting, since the minimum rainfall is no rainfall. Although some statistics can be applied to minimum annual or monthly rainfall, in which case often adequate use can be made of the log-normal distribution, for short periods of observation statistical analysis is nonsense. What one can analyse, however, and what has particular relevance for rain fed agriculture, is the occurrence of dry spells. In the following analysis, use is made of a case in the north of Bangladesh where wet season agriculture takes place on sandy soils. It is known that rain fed agriculture seldom succeeds without supplementary irrigation. In view of the small water retaining capacity of the soil, a dry spell of more than five days already causes serious damage to the crop.

The occurrence of dry spells in the wet season is analysed through frequency analysis of daily rainfall records in Dimla in northern Bangladesh during a period of 18 years. The wet season consists of 153 days. In Table 1.8 the procedure followed is presented, which is discussed briefly on the next page.

Table 1.8: Probability of occurrence of dry spells in the Buri Teesta catchment

spell duration	number of spells	accum. spells	start days per season	total start days *n* *18	*I/N*	*1-p*	*1-q^n*
t	*i*	*I*	*n*	*N*	*p*	*q*	*P*
3	46	165	151	2718	0,0607	0,9393	0,9999
4	20	119	150	2700	0,0441	0,9559	0,9988
5	29	99	149	2682	0,0369	0,9631	0,9963
6	15	70	148	2664	0,0263	0,9737	0,9806
7	11	55	147	2646	0,0208	0,9792	0,9544
8	8	44	146	2628	0,0167	0,9833	0,9150
9	2	36	145	2610	0,0138	0,9862	0,8665
10	5	34	144	2592	0,0131	0,9869	0,8506
11	3	29	143	2574	0,0113	0,9887	0,8022
12	1	26	142	2556	0,0102	0,9898	0,7659
13	2	25	141	2538	0,0099	0,9901	0,7524
14	1	23	140	2520	0,0091	0,9909	0,7230
15	0	22	139	2502	0,0088	0,9912	0,7070
16	1	22	138	2484	0,0089	0,9911	0,7070
17	1	21	137	2466	0,0085	0,9915	0,6901
18	2	20	136	248	0,0806	0,9194	1,0000
19	2	18	135	2430	0,0074	0,9926	0,6335
20	1	16	134	2412	0,0066	0,9934	0,5901
21	1	15	133	2394	0,0063	0,9937	0,5665
22	0	14	132	2376	0,0059	0,9941	0,5416
23	0	14	131	2358	0,0059	0,9941	0,5416
24	0	14	130	2340	0,0060	0,9940	0,5416
25	0	14	129	2322	0,0060	0,9940	0,5417
26	0	14	128	2304	0,0061	0,9939	0,5417
27	2	14	127	2286	0,0061	0,9939	0,5417
28	0	12	126	2268	0,0053	0,9947	0,4875
29	2	12	125	2250	0,0053	0,9947	0,4875
30	0	10	124	2232	0,0045	0,9955	0,4270
31	1	10	123	2214	0,0045	0,9955	0,4270
33	1	9	121	2178	0,0041	0,9959	0,3941
34	1	8	120	2160	0,0037	0,9963	0,3593
35	1	7	199	2142	0,0033	0,9967	0,4787
42	1	6	112	2016	0,0030	0,9970	0,2838
43	1	5	111	1998	0,0025	0,9975	0,2428
49	1	4	105	1890	0,0021	0,9979	0,1995

In the 18 years of records, the number of times *i* that a dry spell of a duration *t* occurs has been counted. Then the number of times *I* that a dry spell occurs of a duration longer than or equal to *t* is computed through accumulation. The number of days within a season on which a dry spell of duration *t* can start is represented by *n=153+1-t*.

The total possible number of starting days is $N=n*18$. Subsequently the probability p that a dry spell starts on a certain day within the season is defined by:

$$p = \frac{I}{N}$$ *Equation 1.14*

The probability q that a dry spell of a duration longer than t does not occur at a certain day in the season, hence, is defined by:

$$q = 1 - \frac{I}{N}$$ *Equation 1.15*

The probability Q that a dry spell of a duration longer than t does not occur during an entire season, hence, is defined by:

$$Q = \left(1 - \frac{I}{N}\right)^n$$ *Equation 1.16*

Finally the probability that a dry spell of a duration longer than t does occur at least once in a growing season is defined by:

$$P = 1 - \left(1 - \frac{I}{N}\right)^n$$ *Equation 1.17*

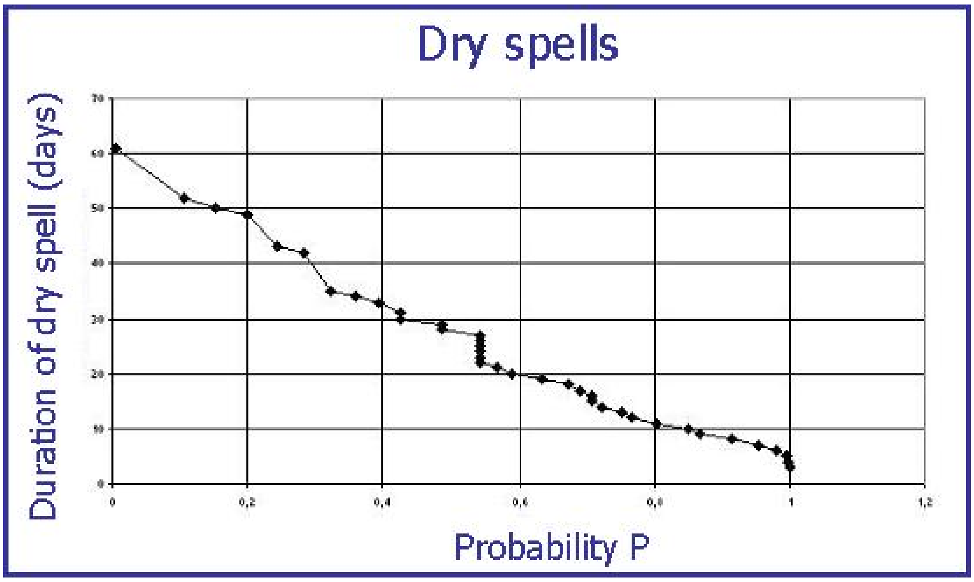
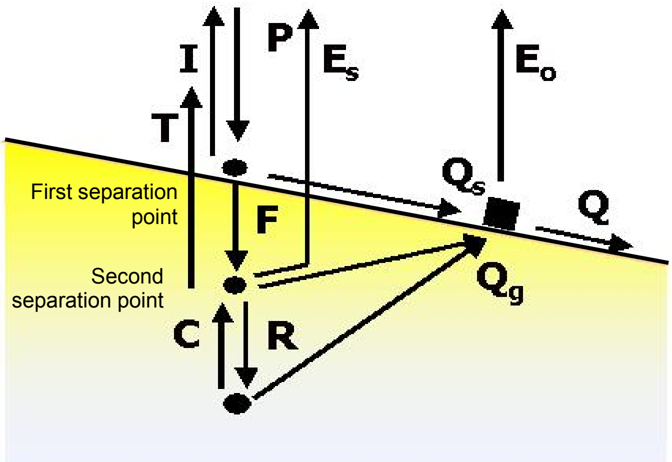


Figure 1.27: Probability of occurrence of dry spells in the Buri Teesta catchment

In Figure 1.27, the duration of the dry spell t is plotted against this probability P. It can be seen that the probability of a dry spell longer than five days, which already causes problems in the sandy soils, has a probability of occurrence of 99.6%, meaning that these dry spells occur virtually every year. Hence, Figure 1.27 illustrates the fact that rain fed agriculture is impossible in the area during the wet season.

1.3 Water resources

The origin of water resources is rainfall. As rainfall P reaches the surface it meets **the first separation point**. At this point part of the rainwater returns directly to the atmosphere, which is called evaporation from **interception,** I. The remaining rainwater infiltrates into the soil until it reaches the capacity of infiltration. This is called **infiltration** F. If there is enough rainfall to exceed the interception and the infiltration, or if the land surface is saturated with water, then **overland flow** (also called surface runoff) Q_s is generated. The overland flow is a fast runoff process, which generally carries soil particles. A river that carries a considerable portion of overland flow has a brown muddy colour and carries debris.



The infiltration reaches the soil moisture. Here lies the **second separation point**. From the soil moisture part of the water returns to the atmosphere through **transpiration** T. If the soil moisture content is above field capacity (or if there are preferential pathways) part of the soil moisture percolates towards the groundwater. This process is called groundwater **recharge** (R) or percolation. The reverse process of percolation is **capillary rise** (C). The recharge feeds and renews the groundwater stock. On hillslopes there is a process that generates runoff from infiltrated water through preferential path ways, e.g. through root channels, horizontal cracks or along contact zones between different soil layers. This rapid sub-surface flow is often called interflow and can be substantial on hillslopes with a shallow layer of weathered rock. Finally there is direct evaporation from the soil (E_s). Whether **soils evaporation** should be taken from the first or the second separation point depends on the depth of the soil. Since soil evaporation is a shallow process, it is often better to consider it as a process connected to the first separation point, similar to evaporation from interception.

On average the percolation minus the capillary rise equals the **seepage** of groundwater Q_g to the surface water. The seepage water is clean and does not carry soil particles. A river that has clear water carries water that stems from groundwater seepage. This is the slow component of runoff. During the rise of a flood in a river when the watercolour is brown, the water stems primarily from overland flow. During the recession of the flood, when the water is clear, the river flow stems completely from groundwater seepage.

1.3.1 Water scarcity and the rainbow of water

Water scarcity

In the eyes of the public, water scarcity is associated with lack of drinking water. That is not so strange. Drinking water, although in terms of quantity a very small consumer of water resources, is closest to people's environment and experience. Consequently, in the discussion on water scarcity, the image most commonly conveyed by the media is that of thirst. We see pictures of people standing next to a dry well, or people walking large distances to collect a bucket of water. Or, on a more positive note, people happily crowded around a new water point that spills crystal clear water.

Thirst, however, is not a problem of water scarcity; it is a problem of water management. There is enough water, virtually everywhere in the world, to provide people with their basic water needs: drinking, cooking and personal hygiene. Shortage of water for primary purposes (essentially household water) is much more a problem of lifestyles and poor management than of water availability. As a result of the "sanitary revolution" of the Victorian age, drinking water is mainly used to convey our waste over large distances to places where we then try to separate the water from the waste. This way of sanitation, which probably was highly efficient at the beginning of this century when there was no scarcity of water nor an environmental awareness, is now highly inefficient in terms of energy consumption, money and water alike. An extra-terrestrial visiting the Earth would be very surprised to see that clean and meticulously treated drinking water, which is considered a precious and scarce commodity, is used for the lowest possible purpose: to transport waste. Subsequently, the waste is removed through a costly process, after which the water is often pumped back and retreated to be used again. We need a new sanitary revolution, as suggested by Niemczynowicz (1997), to restore this obvious inefficiency.

If drinking water is not the problem of global water scarcity, then what is? Of the 1700 m^3/cap/yr of renewable fresh water that is considered an individual's annual requirement (Gardner-Outlaw & Engelman, 1997), close to 90% is needed for food production. For primary water consumption 100 l/cap/day may be considered sufficient. After the second sanitary revolution it may become even less. On an annual basis this consumption amounts to about 40 m^3/cap/yr. Industrial use may be several times this amount, but also in the industrial sector, a sanitary revolution could seriously reduce the industrial water consumption.

The water scarcity problem is primarily a food problem (Brown, 1995). The production of a kilogram of grains under proper climatic and management conditions, requires about 1-2 m^3 of water, but it can reach as much as 4 m^3 of water per kg in tropical dry climates (Falkenmark & Lundqvist, 1998). A kilogram of meat requires a multiple of this amount. Apparently, the per capita water requirement primarily depends on our food needs and habits. Consequently, the main question to address is: how are we going to feed an ever growing population on our limited land and water resources?

A rainbow of water

Of all water resources, "green water" is probably the most under-valued resource. Yet it is responsible for by far the largest part of the world's food and biomass production. The concept of "green water" was first introduced by Falkenmark (1995), to distinguish it from "blue water", which is the water that occurs in rivers, lakes and aquifers. The storage medium for green water is the unsaturated soil. The process through which green water is consumed is transpiration. Hence the total amount of green water resources available over a given period of time equals the accumulated amount of transpiration over that period. In this definition irrigation is not taken into account. Green water is transpiration resulting directly from rainfall, hence we are talking about rain fed agriculture, pasture, forestry, etc. The average residence time of green water in the unsaturated zone is the ratio of the storage to the flux (the transpiration). At a global scale the storage in the unsaturated zone is about 440 mm (see Table 1.9 and Table 1.9: 65/149). In tropical areas the transpiration can amount to 100 mm/month. Hence the average residence time of green water in tropical areas is approximately 5 months. This residence time applies to deep rooting vegetation. For shallow rooting vegetation (most agricultural crops) the residence time in the root zone is substantially shorter. In temperate and polar areas, where transpiration is significantly less, the residence time is much longer. At a local scale, depending on climate, soils and topography, these numbers can vary significantly.

Green water is a very important resource for global food production. About 60% of the world staple food production relies on rain fed irrigation, and hence green water. The entire meat production from grazing relies on green water, and so does the production of wood from forestry. In Sub-Saharan Africa almost the entire food production depends on green water (the relative importance of irrigation is minor) and most of the industrial products, such as cotton, tobacco, wood, etc.

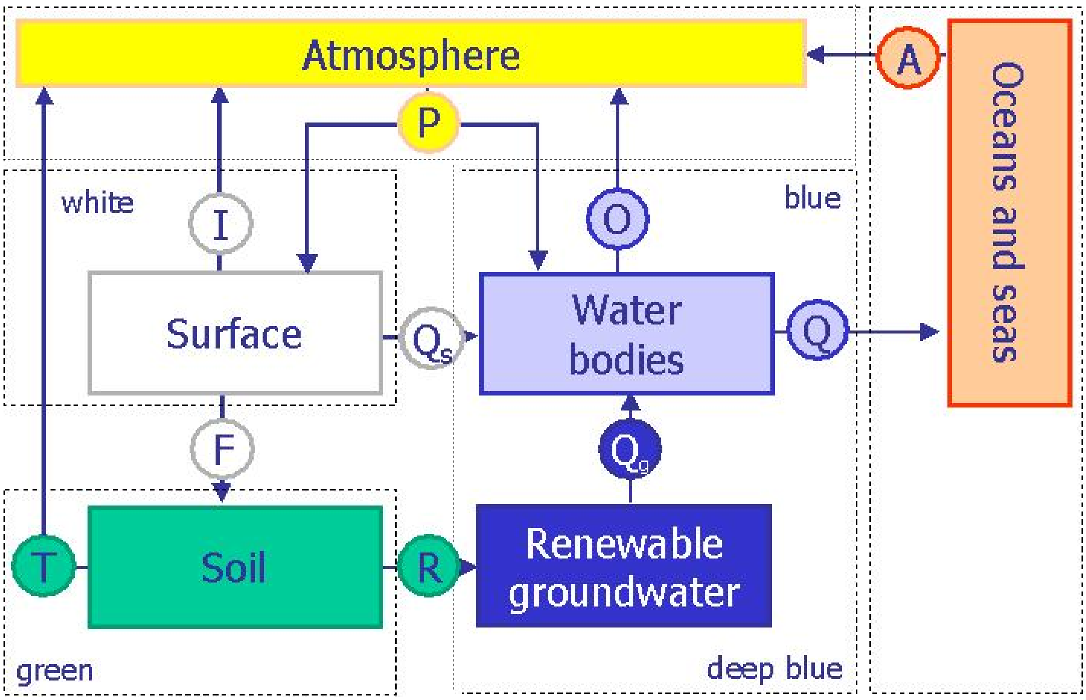


Figure 1.28: Global Water Resources: Blue, Green, White

There is no green water without blue water, as their processes of origin are closely related. Blue water is the sum of the water that recharges the groundwater and the water that runs-off over the surface. Blue water occurs as renewable groundwater in aquifers and as surface water in water bodies. These two resources can not simply be added, since the recharge of the renewable groundwater eventually ends up in the surface water system. Adding them up often implies double counting. Depending on the climate, topography and geology, the ratio of groundwater recharge to total blue water varies. In some parts the contribution of the groundwater to the blue water can be as high as 70-80%, in some parts (on solid rock surface), it can be negligible. Generally the groundwater contribution to the blue water is larger than one thinks intuitively. The reason that rivers run dry is more often related to groundwater withdrawals, than to surface water consumption.

Engineers always have had a preference for blue water. For food production, engineers have concentrated on irrigation and neglected rain fed agriculture, which does not require impressive engineering works. Irrigation is a way of turning blue water into green water. Drainage is a way of turning green water into blue water.

To complete the full picture of the water resources, besides green water and blue water, there is "**white water**". White water is the part of the rainfall that feeds back directly to the atmosphere through evaporation from interception and bare soil. Some people consider the white water as part of the green water, but that adds to confusion since green water is a productive use of water whereas the white water is non-productive. The white and green water together form the vertical component of the water cycle, as opposed to the blue water, which is horizontal. In addition, the term white water can be used to describe the rainfall, which is intercepted for human use, including rainwater harvesting. Figure 1.28 gives a schematic representation of these three colours. The groundwater is part of the blue water and may be painted "deep blue'. The fossil water does not enter into the picture, since it is unrenewable and not related to rainfall.

Table 1.9: Global Water Resources, fluxes, storage and average residence times

Resource	Fluxes	[L/T] or [L^3/T]	Storage	[L] or [L^3]	Residence time	[T]
Green	T	100 mm/month	S_u	440 mm	S_u/T	4 months
White	I	5 mm/d *)	S_s	3 mm *)	S_s/I	0.6 days
Blue	Q	46 x 10^{12} m^3/a	S_w	124 x 10^{12} m^3	S_w/Q	2.7 years
Deep blue	Q_g	5 x 10^{12} m^3/a *)	S_g	750 x 10^{12} m^3 *)	S_g/Q_g	150 years
Atmosphere	P	510 x 10^{12} m^3/a	S_a	12 x 10^{12} m^3	S_a/P	0.3 month
Oceans	A	46 x 10^{12} m^3/a	S_o	1.3 x 10^{18} m^3	S_o/A	28000 year

Note: transpiration and interception fluxes apply to tropical areas storage in the root zone can be significantly less than 440 mm

*) Indicate rough estimates

Table 1.9 presents the quantities of fluxes and stocks of these water resources, and the resulting average residence times, at a global scale. For catchments and sub-systems similar computations can be made. The relative size of the fluxes and stocks can vary considerably between catchments. Not much information on these resources exists at sub-catchment scale.

The study of the Mupfure catchment in Zimbabwe by Mare (1998) is an exception. Table 1.10 illustrates the importance of green water and renewable groundwater in a country where these resources have been mostly disregarded. Figure 1.29, based on 20 years of records (1969-1989) in the Mupfure basin in Zimbabwe (1.2 Gm^2), shows the separation of rainfall into interception (White), Green and Blue water. The model used for this separation is described by Savenije (1997). It can be seen that there is considerably more green water than blue water available in the catchment. Table 1.10 shows the average values over the 20 years of records. Moreover, the model showed that more than 60% of the blue water resulted from groundwater, a resource until recently neglected in Zimbabwe.

Finally, the last colour of the rainbow is the ultra-violet water, the invisible water, or the "**virtual water**". Virtual water is the amount of water required to produce a certain good. In agriculture, the concept of virtual water is used to express a product in the amount of water required for its production. The production of grains typically requires 2-3 m^3/kg, depending on the efficiency of the production process. Trading grains implies the trade of virtual water (Allan, 1994).

Nyagwambo (1998) demonstrated that in the Mupfure basin, blue water applied to tobacco has a productivity of around 3.40 Z$/m^3, whereas productivity of water for wheat is only around 0.50 Z$/m^3. Since wheat and tobacco can be both traded on the international market, the best use of water resources of the Mupfure would be to produce tobacco, export it and buy the required wheat on the international market. One cubic metre of water applied to tobacco would allow the importation of 7 m^3 of 'virtual water' in the form of grains. A net gain to the basin of 6 m^3 of water! Supplementary irrigation during the rainy season of rain fed crops has a relatively high productivity. In the communal areas, one cubic metre of blue water applied to a rain fed crop as supplementary irrigation results in production gains valued at Z$ 1.00 to Z$ 1.30 (1996 prices), equivalent to some US$ 0.10.

Table 1.10: Water resources partitioning and variability in the Mupfure River Basin, Zimbabwe

Muphure river Station: C70 Catchment area: 1.2 Gm^3 Record length: 1969-1989	Source	Vertical component		Horizontal component
Resource type	Rainfall (P)	"White" (W)	"Green" (G)	"Blue" (B)
Mean annual flux (μ)	775 mm/a	446 mm/a	202 mm/a	126 mm/a
Partitioning	100%	62%	23%	15%
Standard deviation (σ)	265 mm/a	48 mm/a	135 mm/a	87 mm/a
Inter-annual variability (σ/μ)	34%	11%	67%	69%

In water scarce regions, the exchange of water in its virtual form is one of the most promising approaches for sharing international waters. It allows a region, such as SADC, to produce water intensive products there where land and water conditions are most favourable, while the interdependency thus created, guarantees stability and sustainability of supply (Savenije & Van der Zaag, 1998, pp60-62).

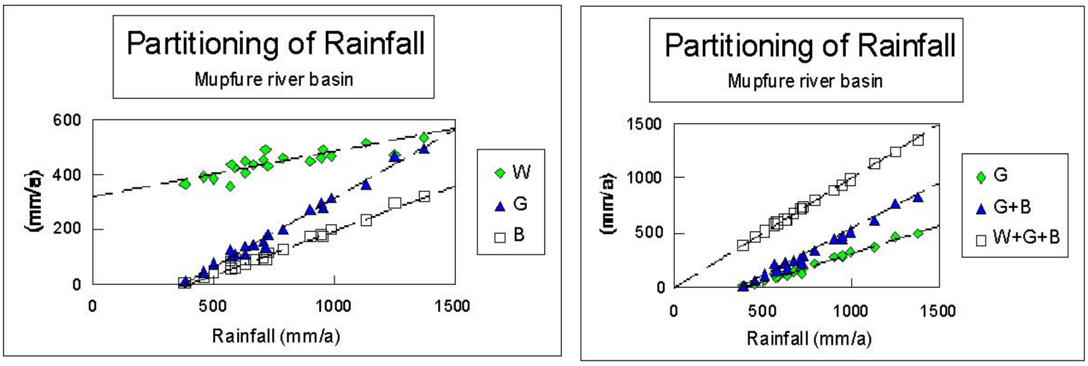


Figure 1.29: Partitioning of rainfall between "White", "Green" and "Blue" water in the Mupfure sub-catchment in Zimbabwe (records of 1969-1989)

Water scarcity and the deception of numbers

The data on the annual renewable per capita water availability are deceptive for a number of reasons. First of all, the different colours of the water trouble the global discussion on water scarcity. Statistics as in Engelman & LeRoy (1993) and Gardner-Outlaw & Engelman (1997) concentrate on blue water and disregard green water. In temperate climates food production is primarily from green water. The quantities of food produced and the further potential in North America, South America and Europe are large. Yet this resource is fully disregarded in the data. The inclusion of green water in the data on national water resources would paint a substantially less gloomy picture of global food security. Presently about 60-70% of the world food production is from rain fed agriculture (Lundqvist & Sandström, 1997; Winpenny, 1997). The trade of this food (virtual green water) is an important mechanism for food security. Or as Allan (1997, unpublished Internet report) puts it: “More water flows into the Middle East each year in its virtual form than is used for annual crop production in Egypt”. In addition, the recycling of grey water, or the harvesting of white water is not taken into consideration.

Secondly, the average annual data on water availability tend to hide an important characteristic of water resources: its natural temporal variability. In tropical countries, and particularly in (semi-) arid regions, the temporal variation of water availability, both within the year and between years, is large, much larger than in temperate climates. Figure 1.29 shows the inter-annual variability of the water resources for the Mupfure basin. It is interesting to note that the variability of the blue and green water is substantially larger than the variability of the rainfall, which in itself is erratic. This enhancement of the variability is the result of the interception component, which is relatively constant between years since it has an upper boundary in the potential evaporation. Because the latter is larger in tropical regions, the amounts of blue and green water in tropical countries (and their reliabilities) are lower for the same amount of rainfall. This variability is enhanced by the occurrence of the ENSO (El Niño Southern Oscillation) effect, now so popular in the media. This effect is a cyclic natural phenomenon and not, as is often suggested, a result of climatic change. The regular occurrence of these variabilities, however, reduces the reliability of the resource. In the present data on water availability a cubic metre of water in Europe is a much more reliable resource than a cubic metre in Africa.

Thirdly, the statistics do not distinguish between climatic conditions. Although Belgium and South Africa have the same per capita amounts of renewable water resources, the actual perceived water scarcity is quite different in both countries. The main reason for this inequality is the potential evaporation, which in South Africa is at least double the amount of Belgium and the larger spatial and temporal variation of rainfall in South Africa, both within the season and between seasons.

A contentious issue hidden in the data of Engelman & LeRoy (1993), and in later additions, is the key that was used to distribute the amounts of renewable fresh water among the riparians that share a water resource. It is not at all clear which key was used (Van der Zaag & Savenije, 1998). If it is based on hydrological recordings in individual countries, then there is a danger of double counting or of confirming the status quo, which may not be the proper key to use. One can approach the issue from two angles: either taking the source of the water as the key for distribution, or the occurrence of the water. The first approach is transparent but rather unfair. It implies that the water resources of a country are equal to the sum of the blue and green water resources that stem directly from the rain falling on its territory. It would imply that Egypt has virtually no water resources of its own. The latter approach seems fairer but is not transparent; it leads to double counting if a river flows through several countries. Although this approach appears to confirm the status quo (first come first served), the picture changes as soon as an upstream country starts to abstract water. Without going into the contentious problem of how to allocate water between riparians, we do need an objective key to compute the distribution of our global water resources between countries, taking also green water into account.

Finally, it is necessary to distinguish between primary water needs (basic water requirements of households, which do not have to exceed 100 l/cap/day) and secondary needs, which can be addressed through the trade of virtual water. Much can be said for including the needs of essential ecosystems in the primary needs (e.g. Saeijs & Van Berkel, 1995), as has been done implicitly in the new South African Water Act, where basic human needs and the needs of the environment are given priority above all other uses. The international consensus is that water for secondary purposes should be considered as an economic good, which means at least that priorities in allocation should be based on socio-economic criteria, but could go as far as economic pricing.

Also lifestyles have an important impact on the perceived scarcity. Water borne sanitation requires much more water than dry sanitation. People that eat a meat-rich diet require much more water for food production than vegetarians. If India or China assumed the lifestyles of the North Americans or Europeans, our global water resources would be insufficient already. Wackernagel & Rees (1995) in “Our Ecological Footprint” demonstrate that if we all started to live like North Americans, we would need three or four globes to support us (McDonald, 1998).

1.3.2 Groundwater resources

Groundwater can be split up into **fossil** groundwater and **renewable** groundwater. Fossil groundwater should be considered a finite mineral resource, which can be used only once, after which it is finished. Renewable groundwater is groundwater that takes an active part in the hydrological cycle. The latter means that the residence time of the water in the sub-surface has an order of magnitude relevant to human planning and considerations of sustainability. The limit between fossil and renewable groundwater is clearly open to debate. Geologists, who are used to working with time scales of millions of years, would only consider groundwater as fossil if it has a residence time over a million years. A hydrologist might use a timescale close to that. However, a water resources planner should use a time scale much closer to the human dimension, and to the residence time of pollutants.

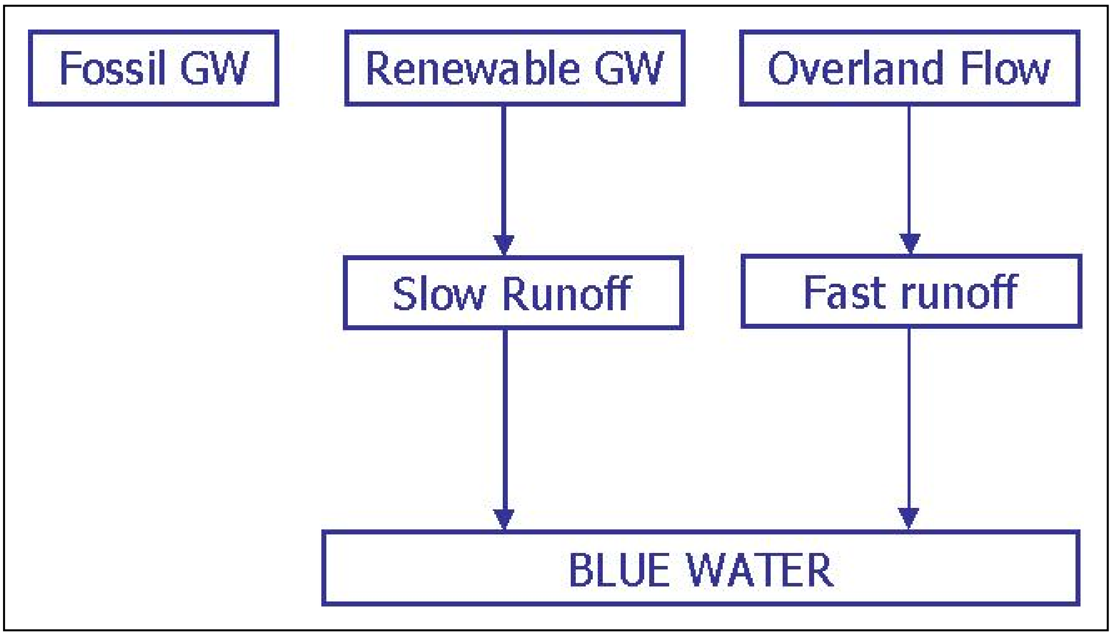


Figure 1.30: Blue water is surface runoff plus seepage from renewable groundwater

In our definition, the renewable groundwater takes active part in the hydrological cycle and hence is “blue water”. Groundwater feeds surface water and vice versa. In the Mupfure catchment in Zimbabwe, Mare (1997) showed that more than 60% of the total runoff of the catchment originated from groundwater. Hence most of the water measured at the outfall was groundwater. One can say that all renewable groundwater becomes surface water and most of the surface water was groundwater.

Two zones can be distinguished in which water occurs in the ground:

- the saturated zone
- the unsaturated zone.

For the hydrologist both zones are important links and storage devices in the hydrological cycle: the unsaturated zone stores the "green water", whereas the saturated zone stores the "blue" groundwater. For the engineer the importance of each zone depends on the field of interest. An agricultural engineer is principally interested in the unsaturated zone, where the necessary combination of soil, air and water occurs for a plant to live. The water resources engineer is mainly interested in the groundwater, which occurs and flows in the saturated zone.

The process of water entering into the ground is called infiltration. Downward transport of water in the unsaturated zone is called percolation, whereas the upward transport in the unsaturated zone is called capillary rise. The flow of water through saturated porous media is called groundwater flow. The outflow from groundwater to surface water is called seepage.

The type of openings (voids or pores) in which groundwater occurs is an important property of the subsurface formation. Three types are generally distinguished:

1. Pores, openings between individual particles as in sand and gravel. Pores are generally interconnected and allow capillary flow for which Darcy's law (see below) can be applied.
2. Fractures, crevices or joints in hard rock which have developed from breaking of the rock. The pores may vary from super capillary size to capillary size. Only for the latter situation application of Darcy's law is possible. Water in these fractures is known as fissure or fault water.
3. Solution channels and caverns in limestone (karst water), and openings resulting from gas bubbles in lava. These large openings result in a turbulent flow of groundwater which cannot be described with Darcy's law.

The porosity n of the subsurface formation is that part of its volume which consists of openings and pores:

$$n = \frac{V_p}{V} \qquad \textit{Equation 1.18}$$

where V_p is the pore volume and V is the total volume of the soil.

When water is drained by gravity from saturated material, only a part of the total volume is released. This portion is known as specific yield. The water not drained is called specific retention and the sum of specific yield and specific retention is equal to the porosity. In fine-grained material the forces that retain water against the force of gravity are high due to the small pore size. Hence, the specific retention of fine-grained material (silt or clay) is larger than of coarse material (sand or gravel).

Groundwater is the water, which occurs in the saturated zone. The study of the occurrence and movement of groundwater is called groundwater hydrology or geohydrology. The hydraulic properties of a water-bearing formation are not only determined by the porosity but also by the interconnection of the pores and the pore size. In this respect the subsurface formations are classified as follows:

1. Aquifer, which is a water-bearing layer for which the porosity and pore size are sufficiently large to allow transport of water in appreciable quantities (e.g. sand deposits).
2. Aquiclude is an impermeable layer, which may contain water but is incapable of transmitting significant quantities.
3. Aquitard is a less permeable layer, not capable of transmitting water in a horizontal direction, but allowing considerable vertical flow (e.g. clay layer).
4. Aquifuge is impermeable rock neither containing nor transmitting water (e.g. granite layers).

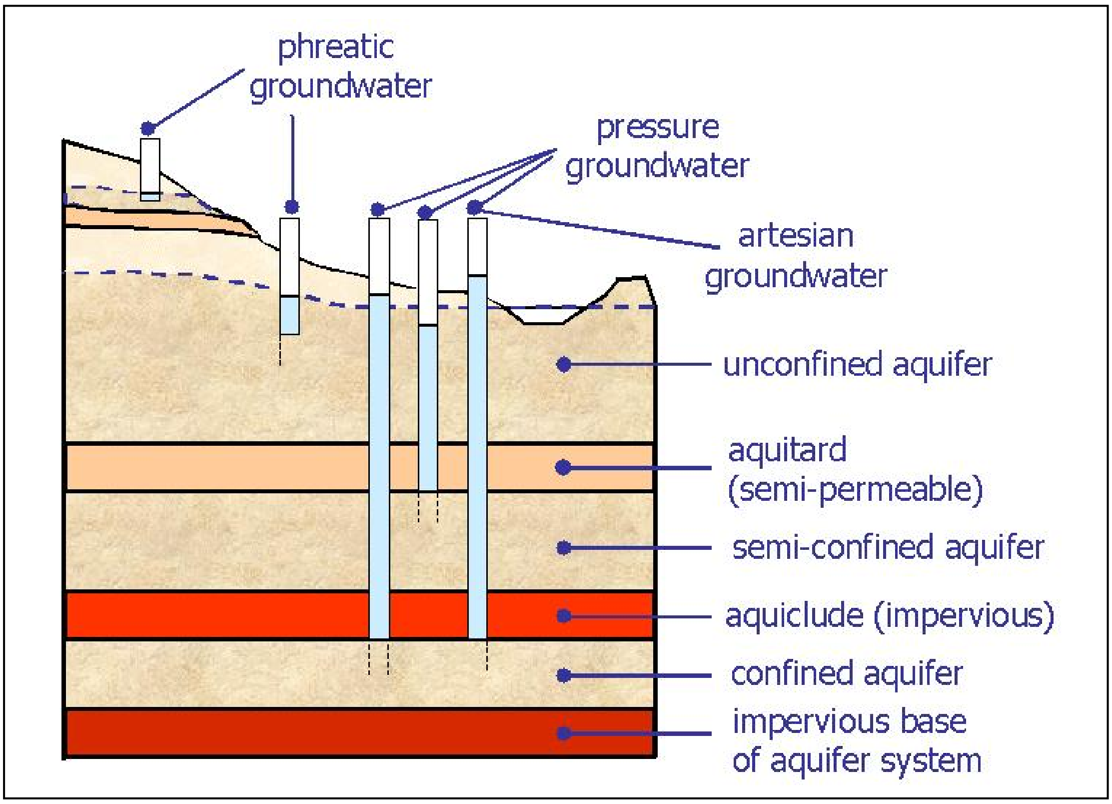


Figure 1.31: Aquifer types

Aquifers

For a description or mathematical treatment of groundwater flow the geological formation can be schematised into an aquifer system, consisting of various layers with distinct different hydraulic properties. The aquifers are simplified into one of the following types (see Figure 1.31):

1. Unconfined aquifer (also phreatic or water-table aquifer), which consists of a pervious layer underlain by a (semi-) impervious layer. The aquifer is not completely saturated with water. The upper boundary is formed by a free water table (phreatic surface).
2. Confined aquifer, consisting of a completely saturated pervious layer bounded by impervious layers. The water level in wells tapping those aquifers rises above the top of the pervious layer and sometimes even above soil surface (artesian wells).
3. Semi-confined or leaky aquifers consist of a completely saturated pervious layer, but the upper and/or under boundaries are semi-pervious.

The pressure of the water in an aquifer is measured with a piezometer, which is an open-ended pipe with a diameter of 3 - 10 cm. The height to which the water rises

with respect to a certain reference level (e.g. the impervious base, mean sea level, etc.) is called the hydraulic head. Strictly speaking the hydraulic head measured with a piezometer applies for the location at the lower side of the pipe, but since aquifers are very pervious, this value is approximately constant over the depth of the aquifer. For unconfined aquifers the hydraulic head may be taken equal to the height of the water table, which is known as the Dupuit-Forchheimer assumption.
Water moves from locations where the hydraulic head is high to places where the hydraulic head is low. For example, Figure 1.31 shows that the hydraulic head in the semi-confined aquifer is below the hydraulic head (or phreatic surface) in the unconfined aquifer. Hence, water flows through the semi-pervious layer from the unconfined aquifer into the semi-confined aquifer. A perched water table may develop during a certain time of the year when percolating soil moisture accumulates above a less pervious layer.

Groundwater flow

The theory on groundwater movement originates from a study by the Frenchman Darcy, first published in 1856. From many experiments (Figure 1.32) he concluded that the groundwater discharge *Q* is proportional to the difference in hydraulic head *H* and cross-sectional area *A* and inversely proportional to the length *s*, thus:

$$Q = A \cdot v = -A \cdot k \cdot \frac{\Delta H}{\Delta s}$$ *Equation 1.19*

where *k*, the proportionality constant, is called the hydraulic conductivity, expressed in m/d; and *v* is the specific discharge, also called the filter velocity.

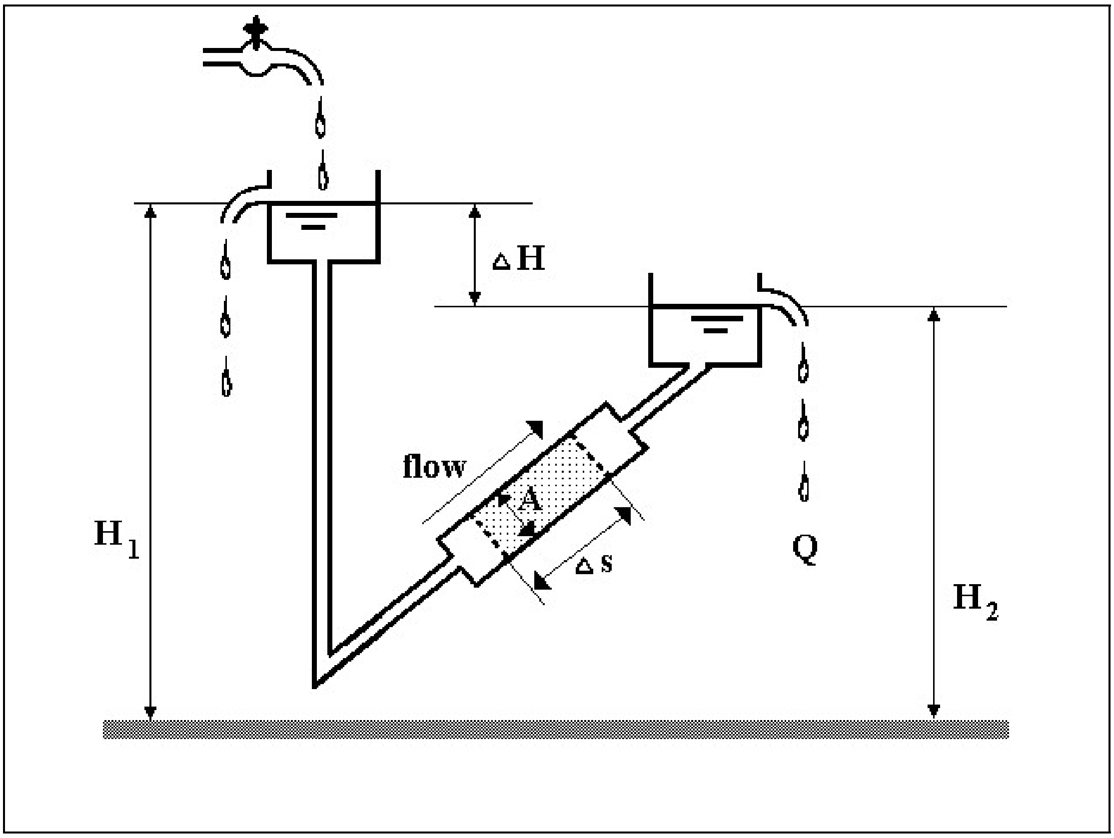


Figure 1.32: Experiment of Darcy

Since the hydraulic head decreases in the direction of flow, the filter velocity has a negative sign. The actual velocity v_{act} of a fluid particle is much higher because only the effective pore space n_e is available for transport, thus:

$$v_{act} = \frac{v}{n_e} \qquad \textit{Equation 1.20}$$

The effective porosity n_e is smaller than the porosity n, as the pores that do not contribute to the transport are excluded (dead-end pores). The actual velocity is important in water quality problems, to determine the transport of contaminants.

For a detailed description of groundwater flow and groundwater recovery, reference is made to the respective lectures.

Groundwater as a storage medium

For the water resources engineer groundwater is a very important water resource for the following reasons:

- it is a reliable resource, especially in climates with a pronounced dry season
- it is a bacteriological safe resource, provided pollution is controlled
- it is often available in situ (wide-spread occurrence)
- it may supply water at a time that surface water resources are limited
- it is not affected by evaporation loss, if deep enough
- there is a large storage capacity
- it can be easily managed

It also has a number of disadvantages:

- it is a strongly limited resource, extractable quantities are often low as compared to surface water resources.
- groundwater recovery is generally expensive as a result of pumping costs.
- groundwater, if phreatic, is very sensitive to pollution.
- groundwater recovery may have serious impact on land subsidence or salinization.

Especially in dry climates the existence of underground storage of water is of extreme importance. The water stored in the subsoil becomes available in two ways. One way is by artificial withdrawal (pumping) the other is by natural seepage to the surface water.

The latter is an important link in the hydrological cycle. Whereas in the wet season the runoff is dominated by surface runoff, in the dry season the runoff is almost entirely fed by seepage from groundwater (base flow). Thus the groundwater component acts as a reservoir, which retards the runoff from the wet season rainfall and smoothes out the shape of the hydrograph.

The way this outflow behaves is generally described as a linear reservoir, where outflow is considered proportional to the amount of storage:

$$Q = \frac{1}{K} \cdot S \qquad \textit{Equation 1.21}$$

where K is a conveyance factor of the dimension s. Equation 1.21 is an empirical formula which has some similarity with the Darcy equation (Equation 1.19). In combination with the water balance equation, and ignoring the effect of rainfall P and evaporation E ($\Delta S/\Delta t$=-Q), Equation 1.21 yields an exponential relation between the discharge Q and time t.

$$\frac{\Delta S}{S} = -\frac{\Delta t}{K}$$

hence:

$$S = S(t_0)\exp\left(-\frac{t-t_0}{K}\right) \qquad \textit{Equation 1.22}$$

and hence, using Equation 1.21:

$$Q = Q(t_0)\exp\left(-\frac{t-t_0}{K}\right) \qquad \textit{Equation 1.23}$$

Equation 1.23 is a useful equation for the evaluation of surface water resources in the dry season.

1.3.3 Surface water resources

Surface water resources are water resources that are visible to the eye. They are mainly the result of overland runoff of rainwater, but surface water resources can also origin from groundwater, as was stated in section 1.3. As Mare (1997) pointed out, more than 60% of the surface water in the Muphure basin stemmed from groundwater, a resource hitherto disregarded. Surface water is linked to groundwater resources through the processes of infiltration (from surface water to groundwater) and seepage (from groundwater to surface water).

Surface water occurs in two kinds of water bodies:

- water courses, such as rivers, canals, estuaries and streams;
- stagnant water bodies, such as lakes, reservoirs, pools, tanks, etc.

The first group of water bodies consists of conveyance links, whereas the second group consists of storage media. Together they add up to a surface water system.

The amount of water available in storage media is rather straightforward as long as a relation between pond level and storage is known. The surface water available in channels is more difficult to determine since the water flows. The water resources of a channel are defined as the total amount of water that passes through the channel over a given period of time (e.g. a year, a season, a month). In a given cross-section of a channel the total available amount of surface water runoff over a time step t is defined as the average over time of the discharge Q.

$$\overline{Q} = \frac{1}{\Delta t}\int_{t}^{t+\Delta t} Q\,dt \qquad \textit{Equation 1.24}$$

The discharge Q 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 calm pool, just upstream of a rapid. 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:

$$Q = a\left(H - H_0\right)^b \qquad \textit{Equation 1.25}$$

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.

Equation 1.25 is compatible with the Manning formula where the cross-sectional area A, and the hydraulic radius R are functions of (H-H_0).

$$Q = \frac{A}{n} R^{2/3} \sqrt{S} \qquad \textit{Equation 1.26}$$

Consequently, it can be shown that the coefficient b in equation 1.25 should have a value of 1.59 in a rectangular channel, a value of 1.69 in a trapezoidal channel with 1:1 side slopes, a value of 2.16 in a parabolic channel, and a value of 2.67 in a triangular channel.

Water quality

The total water resources of a catchment are formed by the sum of surface water and groundwater. Both resources may not be considered separately from the water quality. Abundant water resources of poor quality are still useless for consumption. A consumer of water who pollutes the water resources system through its return flows, consumes in fact more water than its actual consumption, as he makes the remaining water useless.

1.3.4 Green water resources

The blue water (Q) can be determined through rating. The difficulty lies with the green water (T).
On an annual basis, the sum of the white water (I), the green water (T), and the open water evaporation (O) equals the overall average evaporation from a catchment E=T+I+O=P-Q (where E is the total annual evaporation, P is the annual rainfall and Q is the annual runoff, all in mm/annum). The white water (I) consists of the direct evaporation from stagnant pools, bare soil evaporation and interception.
Hence, the sum of the blue and green water differs from the total rainfall P by the direct evaporation losses (interception, bare soil and open water evaporation). The green water is productive, or can be made productive. The white water is generally unproductive. Savenije (1997) developed a method to determine the direct evaporation from interception (I), which in fact corresponds to a threshold evaporation “loss”, where I=min(D,P), with a threshold D. On a monthly basis, the transpiration equals the amount of green water T consumed by the vegetation: T=E-I. This method is a further elaboration of the rainfall-runoff model presented in Section 2.8.2.

Figure 1.33 presents the distribution of monthly values of the total evaporation E, the direct evaporation losses W and the rainfall P over time in the Bani catchment in Mali. Of the total rainfall, only the direct evaporation is a loss to the water resources in the catchment. The remainder is the green water and the blue water. However, is this direct evaporation a total loss to the water resources system?

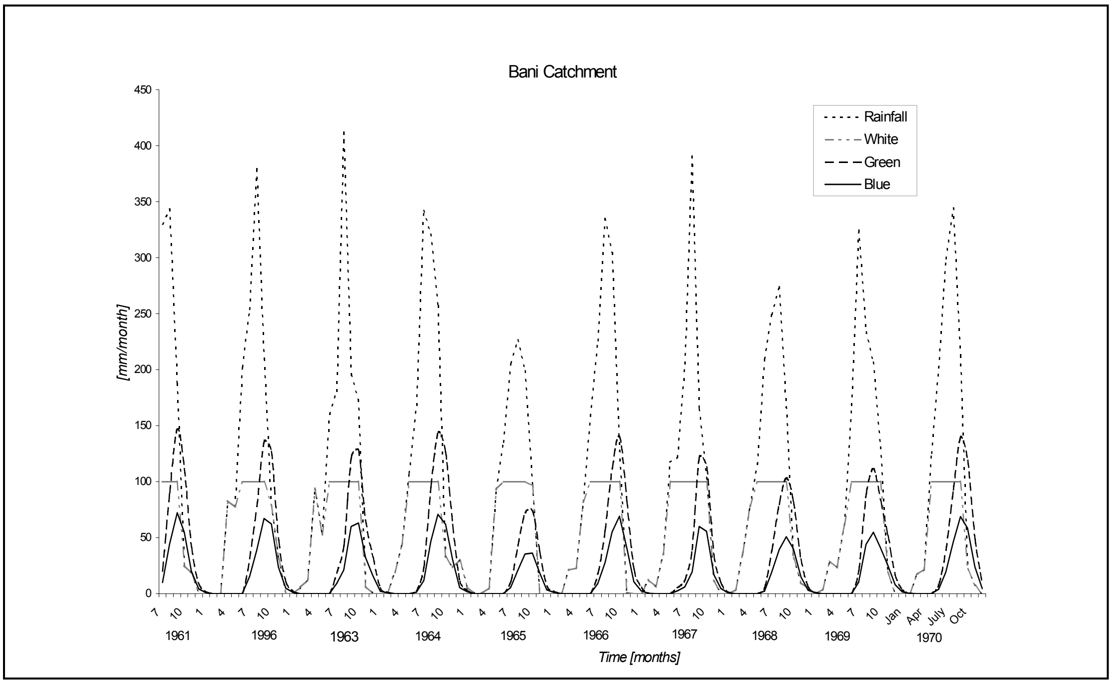


Figure 1.33: Rainfall and evaporation in the Bani catchment, distinguishing between green water and interception losses

Is evaporation a loss?

In most water balances, evaporation is considered a loss. Hydrological engineers, who are asked to determine surface runoff, consider evaporation a loss. Water resources engineers, who design reservoirs, consider evaporation from the reservoir a loss. For agricultural engineers, however, it depends on where evaporation occurs, whether it is considered a loss or not. If it refers to the water evaporated by the crop (transpiration), then evaporation is not a loss, it is the use of the water for the intended purpose. If it refers to the evaporation from canals or from spill, then evaporation is considered a loss, which reduces the irrigation efficiency.

The discussion on whether evaporation is a loss or not, depends on your perspective of the hydrological cycle. Applying the water balance to the earth's continents, Table 1.14 shows that of the 107000 Gm^3/a of rainfall on continents, only 46000 Gm^3/a stems from the ocean, and hence that the remaining 61000 Gm^3/a (57%) is supported by terrestrial evaporation. Without that evaporation, rainfall could not be maintained in the interiors of continental masses. So from that perspective evaporation is not a loss at all.

This situation occurs, for instance, in West Africa. Moist air is transported inland by the monsoon. In the first 500 km, the air moisture, which exceeds the carrying capacity of the atmosphere, precipitates on the tropical rainforests near the coast. Further inland, beyond the rainforest belt, lies the Savanna woodland, followed by the Sahel and the desert. In this part of the continent rainfall becomes gradually scarcer.

Savenije and Hall (1994) and Savenije (1995a) showed that the rainfall that falls on the Sahel, for a very large proportion (more than 90%), is supported by evaporation from the areas nearer to the coast. On average, moisture above the carrying capacity of the atmosphere requires 250 km (recycling length) to precipitate. If at that point it is not recycled (through evaporation) it disappears from the system through runoff or deep percolation. Recycling of moisture through evaporation sustains the rain that falls in the Sahel. Evaporation in the rainforest and the savannah woodlands, hence, is essential for the climate in the Sahel. In this case, evaporation is not a loss, it is the most important source of rainfall further inland.

On a river basin scale, it can be shown that the portion of the rainfall that is recycled from evaporation approaches 1-C_R (where C_R is the runoff coefficient), and that the "multiplier", the number of times that a recycled water particle triggers rainfall, is 1/C_R (Savenije, 1996a, 1996b). In semi-arid river basins, where a runoff coefficient of 10% is not abnormal, this implied that 90% of the rainfall has been triggered by recycled moisture and that, on average, the multiplier is 10. It follows that on continents it is essential to maintain evaporating capacity.

In the Sahel, evaporation is not a loss. What is a loss to the system is surface runoff. Not only does it extract water from the system, it also carries nutrients and soil particles. Reduction of runoff and, hence, increase of local evaporation is an important element of water harvesting. Measures that can be used to decrease runoff are reforestation, erosion control, watershed management, restoration of infiltration capacity, forest-fire prevention and prevention of over-grazing.

1.4 Water balances

The addition of the word integrated to the term water resources refers to three aspects:

- location of the resource: e.g. upstream, downstream, basin, sub-basin
- type of the resource: groundwater, surface water, rainfall harvesting, dew harvesting
- quality: water of bad quality is no resource unless it is treated.

It is not correct to consider the different aspects of water resources in isolation. The integration of location, type and quality is a necessary condition for water resources management

Figure 1.34 shows a picture of the well-known hydrological cycle. In this figure, the direct link between groundwater and surface water is apparent. If we add the aspect of water quality, the picture of integrated water resources is complete.

For Integrated Water Resources Management (IWRM), however, further integration is needed with regard to institutional, economical, financial, legal, environmental and social aspects. But with regard to the physical aspects of water we can limit ourselves to location, type and quality.

1.4.1 General water balance

The water balance is a special case of the general control volume equation, which is the basis of the continuity, momentum, and energy equations for various hydrologic processes. This equation is called the Reynolds Transport Theorem (see Ven te Chow *et al.,* 1988).

Reynolds Transport theorem

Reynolds theorem states that:
"the total rate of change of an extensive property of a fluid dB/dt is equal to the rate of change of an extensive property stored in the control volume, plus the net outflow of extensive property through the control volume".

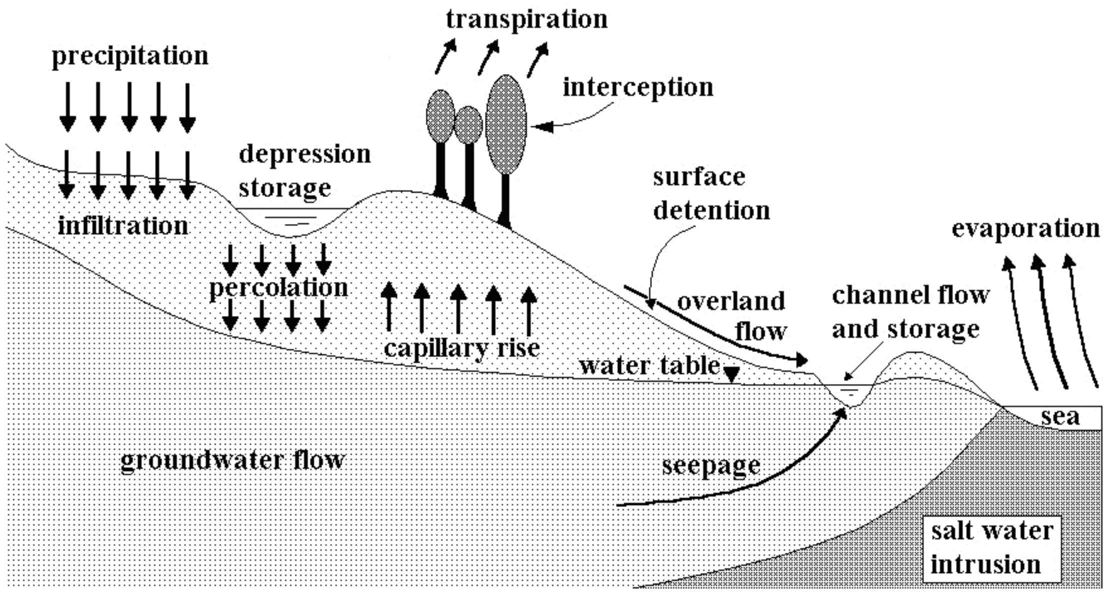


Figure 1.34: Descriptive representation of the hydrological cycle

$$\frac{\mathrm{d}B}{\mathrm{d}t} = \frac{\mathrm{d}}{\mathrm{d}t}\iiint \beta\rho \,\mathrm{d}V + \iint \beta\rho\left(U\,\mathrm{d}A\right) \qquad \text{Equation 1.27}$$

where *B* is the extensive property of the fluid which value depends on the amount of mass present (e.g. mass, momentum: $B = m$, $B = mU$), β is the quantity of *B* per unit of mass, the intensive property of the fluid, of which the value does not depend on mass: $\beta = \mathrm{d}B/\mathrm{d}m$. *B* and β can be scalar or vector quantities. Furthermore, ρ is the density of the fluid; the triple integral signifies the integration over the volume *V*; the double integral is the integration over the cross-sectional area considered *A*; **U·dA** is the vector dot product of the velocity vector of the fluid **U** with length *U*, and the vector **dA** with length d*A* perpendicular to the cross-section. If θ is the angle between **U** and **dA**, then $\mathbf{U}\cdot\mathbf{dA} = U\cos\theta\,\mathrm{d}A$. The first term of the right hand member is the rate of change of the extensive property stored in the control volume; the second term is the net outflow of extensive property through the control surface. When using the theorem, inflows are always considered negative and outflows positive.

Integral equation of continuity

The integral equation of continuity follows from Reynolds theorem and is the basis for the water balance equation. If mass is the extensive property being considered, then $B=m$, and $\beta = \mathrm{d}B/\mathrm{d}m = 1$. By the law of conservation of mass, $\mathrm{d}B/\mathrm{d}t = \mathrm{d}m/\mathrm{d}t = 0$ because mass cannot be created or destroyed. Substitution in Reynolds theorem yields:

$$0 = \frac{\mathrm{d}}{\mathrm{d}t}\iiint \rho \,\mathrm{d}V + \iint \rho\left(U\,\mathrm{d}A\right) \qquad \text{Equation 1.28}$$

which is the integral equation of continuity for an unsteady flow with variable density. If the flow has constant density, ρ can be divided out of both terms:

$$0 = \frac{\mathrm{d}}{\mathrm{d}t}\iiint \mathrm{d}V + \iint\left(U\mathrm{d}\,A\right) \qquad \text{Equation 1.29}$$

The integral $\iiint \mathrm{d}V = S$ is the volume of fluid stored in the control volume. Hence the first term is the time rate of change of the storage $\mathrm{d}S/\mathrm{d}t$. The second term, the net outflow, can be split into inflow *I(t)* and outflow *O(t)*. Because for inflow the direction of the velocity vector **U** and the area vector **dA** point in different directions (the velocity vector pointing in and the area vector pointing out), their vector dot product is negative; whereas for the outflow the vector dot product is positive. Hence:

$$\frac{\mathrm{d}S}{\mathrm{d}t} + O(t) - I(t) = 0$$ *Equation 1.30*

which is the basis for the water budget concept, widely used in the field of hydrology.

Equation 1.30 may be rewritten as:

$$I(t) - O(t) = \frac{\Delta S}{\Delta t}$$ *Equation 1.31*

where I is the inflow in [L^3/T], O is the outflow in [L^3/T], and $\Delta S/\Delta t$ is the rate of change in storage over a finite time step in [L^3/T] of the considered control volume in the system. The equation holds for a specific period of time and may be applied to any given system provided that the boundaries are well defined. Other names for the water balance equation are Storage Equation, Continuity Equation and Law of Conservation of Mass.

1.4.2 Specific water balances

Several types of water balances can be distinguished, like:

- the water balance of the earth surface;
- the water balance of a drainage basin;
- the water balance of the world oceans;
- the water balance of the water diversion cycle (human interference);
- the water balance of a local area like a city, a forest, or a polder.

The water balance of the earth surface

The water balance of the earth surface is composed by all the different water balances that can be distinguished. Data constituting the occurrence of water on earth are given in the Table 1.11, Table 1.12, and Table 1.13. The water balance of the interaction between the earth surface and the world oceans and seas is given in Table 1.14.

Table 1.11: Earth surfaces (Baumgartner, 1972)

	Area in 10^{12} m²	**Area in %**
Water surfaces	361	71
Continents	149	29
Total	510	100

Table 1.12: Earth land use (Baumgartner, 1972)

	Area in 10^{12} m²	**Area in % of total**	**Area in % of continents**
Deserts	52	10	35
Forests	44	9	30*
Grasslands	26	5	17
Arable lands	14	3	9
Polar regions	13	2	9
Oceans	361	71	

** in 1993 this number may be significantly lower*

Table 1.13: Amount of water on earth according to the survey conducted within the international geophysical year (Holy, 1982)

Water occurrence	10^{12} m^3	Amount of water	
		% of water	% of fresh water
World oceans	1.300.000	97	
Salt lakes/seas	100	0.008	
Polar ice	28.500	2.14	77.6
Atmospheric water	12	0.001	0.035
Water in organisms	1	0.000	0.003
Fresh lakes	123	0.009	0.335
Water courses	1	0.000	0.003
Unsaturated zone	65	0.005	0.18
Saturated zone	8.000	0.60	21.8
Total fresh water	36.700	2.77	100
Total water	1.337.000	100	

Table 1.14: Annual water balance of the earth according to the international geophysical year (Holy, 1982)

Region	Area	Precipitation		Evaporation		Runoff	
	10^{12} m²	m/a	10^{12} m^3/a	m/a	10^{12} m^3/a	m/a	10^{12} m^3/a
Oceans	361	1.12	403	1.25	449	-0.13	-46
Continents	149	0.72	107	0.41	61	0.31	46

Table 1.15: Annual water balance of World Oceans

Ocean	Surface area	P-E	Land runoff	Ocean exchange	P-E	Land runoff	Ocean exchange	
	10^{12} m²	mm/a	mm/a	mm/a	10^{12} m^3/a	10^{12} m^3/a	10^{12} m^3/a	m^3/s
Arctic	8.5	44	307	351	0.4	2.6	3	94544
Atlantic	98	-372	197	-175	-36.5	19.3	-17	-543466
Indian	77.7	-251	72	-179	-19.5	5.6	-14	-440739
Pacific	176.9	90	69	159	15.9	12.2	28	891318

The water balance of the oceans

Table 1.15 shows a very interesting result of applying the water balance to the world oceans, particularly with regard to the exchange of water between oceans. It can be concluded from Table 1.15 that from the Pacific about 440 000 m^3/s flows to the Indian Ocean and an equal amount to the Atlantic. An additional 94 500 m^3/s flows directly from the Arctic to the Atlantic. This water balance explains why there is an average flow from the Pacific to the Indian Ocean through the Indonesian Archipelago.

Anthropogenic contribution to seas level rise

Sahagian *et al*. (1994) drew very interesting conclusions from the information presented in Table 1.13 and Table 1.14. They demonstrated that massive withdrawal of fossil water from aquifers has an enormous potential for sea level rise, and that the groundwater withdrawal to date already explains 30% of the existing seas level rise (17 mm to date). A simple computation shows that the total amount of groundwater in the saturated zone corresponds with 8000/361=22 m sea level rise. They conclude, however, that the total removable volume would result in only about 3.9 m sea level rise.

Water balance of a drainage basin

The water balance is often applied to a river basin. A river basin (also called watershed, catchment, or drainage basin) is the area contributing to the discharge at a particular river cross-section. The size of the catchment increases if the point selected as outlet moves downstream. If no water moves across the catchment boundary indicated by the broken line, the input equals the precipitation *P* while the output comprises the evaporation *E* and the river discharge *Q* at the outlet of the catchment. Hence, the water balance may be written as:

$$\left(P-E\right)\cdot A-Q=\frac{\Delta S}{\Delta t} \qquad \textit{Equation 1.32}$$

where *S* is the change of storage over the time step *t*, and *A* is the surface area of the catchment upstream of the station where *Q* has been measured.

All terms must be expressed in the same unit. Precipitation and evaporation are usually measured in mm/d and river discharge in m^3/s. For conversion of one unit into the other, the catchment area *A* must be known. For example, conversion of m^3/s into mm/d for a catchment area of 200 Mm^2 (1 Mm^2 = 1 $(km)^2$) is as follows:
conversion of seconds to days:
1 m^3/s = 86400 m^3/d
conversion of m^3 to mm :
1 $m^3/200\ Mm^2 = 10^3/(200*10^6)$ mm
resulting in:
86400 m^3/d / $2.10^8\ m^2 * 10^3$ mm/m = 0.432 mm/d

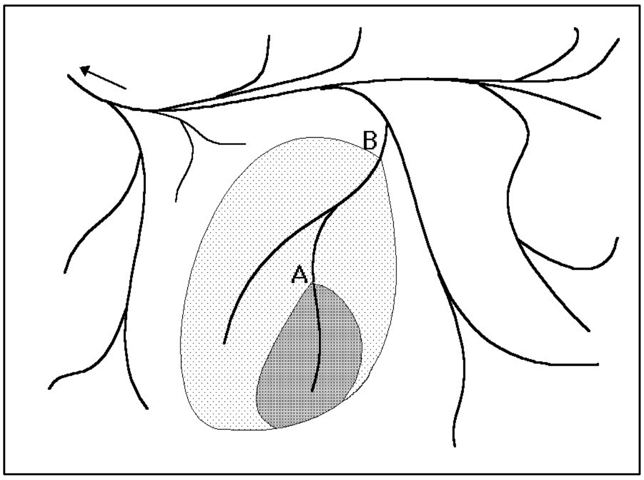


Figure 1.35: The catchment area increases as the control point moves down stream

$\Delta S/\Delta t$, the rate of change in the amount of water stored in the catchment, is difficult to measure. However, if the *account period* t, for which the water balance is established, is taken sufficiently long, the effect of the storage variation becomes less important, because precipitation and evaporation accumulate while storage varies within a certain range. When computing the storage equation for annual periods, the beginning of the balance period is preferably chosen at a time that the amount of water in store is expected not to vary much for each successive year. These annual periods, which do not necessarily coincide with the calendar years, are known as hydrologic years or as water years. The storage equation is especially useful to study the effect of a change in the hydrologic cycle.

The topographic divide between two watersheds is usually taken as the boundary of the catchment, the water divide. Figure 1.36 shows that the topographic divide applies to surface runoff, but may not necessarily coincide with the boundary for groundwater flow, the phreatic divide. Choosing in A the topographic divide as the watershed boundary, leakage of groundwater to a neighbouring catchment will occur. Especially in calcareous rocks where karst regions can be expected, subterranean channels may make it very difficult or impossible to determine the exact watershed boundary.

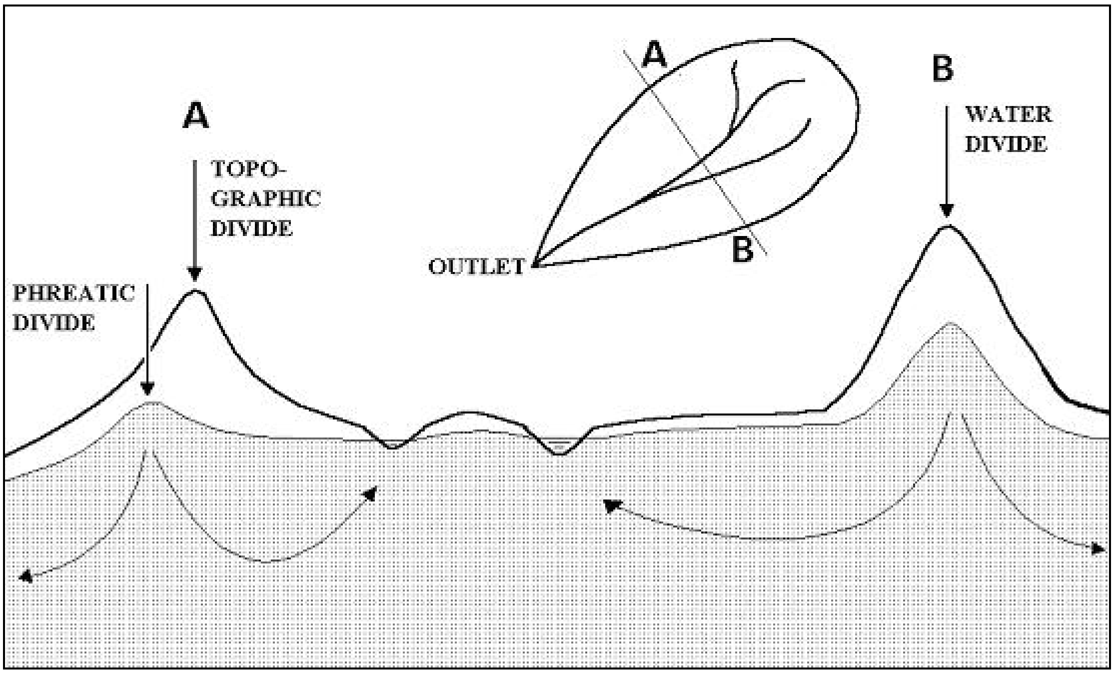


Figure 1.36: Watershed and cross-section showing a phreatic and topographic water divide which coincide in B and deviate in A

Table 1.16 gives the water balance of some of the world's largest drainage basins. Notice the large differences in the *Runoff Coefficient* (the percentage of rainfall that comes to runoff).

Table 1.16: Indicative average annual water balances for the drainage basins of some of the great rivers

River	Catchment	Rainfall size		Evaporation		Runoff		Runoff coefficient
	Gm²	mm/a	Gm^3/a	mm/a	Gm^3/a	mm/a	Gm^3/a	%
Nile	2803	220	620	190	534	30	86	14
Mississipi	3924	800	3100	654	2540	142	558	18
Parana	975	1000	980	625	610	382	372	38
Orinoco	850	1330	1150	420	355	935	795	70
Mekong	646	1500	970	1000	645	382	325	34
Amur	1730	450	780	265	455	188	325	42
Lena	2430	350	850	140	335	212	514	60
Yenisei	2440	450	1100	220	540	230	561	51
Ob	2950	450	1350	325	965	131	385	29
Rhine	200	850	170	500	100	350	70	41
Zambezi	1300	990	1287	903	1173	87	114	12

A colourful look at the water balance of a river basin

With the fluxes and stocks of Table 1.9, the catchment water balance can be further refined. The rainfall can be split-up into three compartments:

$$P = \left(\frac{\mathrm{d}S_s}{\mathrm{d}t} + I\right) + \left(\frac{\mathrm{d}S_u}{\mathrm{d}t} + T\right) + \left(\frac{\mathrm{d}S_w}{\mathrm{d}t} + \frac{\mathrm{d}S_g}{\mathrm{d}t} + Q\right) \qquad \textit{Equation 1.33}$$

where Q is the runoff per unit area in mm/month and stocks are expressed in mm. The first compartment of the right hand member represents the “white” water; the second compartment the “green” water; and the third compartment the “blue water”. After a quantity of rain has been allocated to these three compartments, over time the stocks are transferred into accumulated fluxes, whereby the dS/dt approaches 0 and the accumulated flux approaches the amount of water released form the stock. The time scales of these processes are typically the ratio of the life storage to flux: the residence times T of Table 1.9.

Depending on the time scale of the process to be studied, certain components of Equation 1.33 can be disregarded. If computations are made at daily time scales or more, the term dS_s/dt may be disregarded. Depending on the soil characteristics, generally the term dS_u/dt may be disregarded if the time step is in the order of one or two months. Also, if no significant water bodies are present in the catchment (and if it is sufficiently small for the discharge to reach the outfall within the time step), the term dS_w/dt may be disregarded at a monthly time step. What can generally not be disregarded at a monthly time step is the dS_g/dt (unless the catchment is very small and mountainous). Hence at a monthly time step, for small catchments and in the absence of reservoirs, Equation 1.33 may be simplified into:

$$\frac{\mathrm{d}S_g}{\mathrm{d}t} = P - I - T - Q \qquad \textit{Equation 1.34}$$

In case reservoirs are present, a term dS_w/dt should be added to cater for storage variation in water bodies. This term can generally be easily assessed from reservoir water levels.

1.4.3 Water balance as a result of human interference

Attempts have been made to incorporate the interference of man in the hydrological cycle through the introduction of the water diversion cycle, which includes water withdrawal and water drainage. This diversion cycle is exerting significant influence on the terrestrial water cycle, especially in highly economically developed regions with a dense population (See Figure 1.37).

The water diversion cycle including human interference results in the following annual average water balance equation (neglecting storage variation):

$$\begin{aligned} P &= E + C + Q \\ C &= U_s + U_g - R_s - R_g + H \end{aligned} \qquad \textit{Equation 1.35}$$

In which:
P = precipitation
E = T+ I+ O = total evaporation from the land surface (transpiration + interception + open water evaporation)
C = net water consumption due to water use
Q = runoff from land to ocean
U_s+ U_g = intake from surface and groundwater
R_s+ R_g = return flows to surface and groundwater
H = rainwater harvesting

Figure 1.38 is an attempt to workout the diversion cycle into more detail and to integrate it with the global water cycle of Figure 1.34, which includes blue, green and white water resources. In this respect it is important to note that re-use of return flows

(R_s and R_g) are no additional resources, but merely a way to make water use more efficient (minimizing drainage).

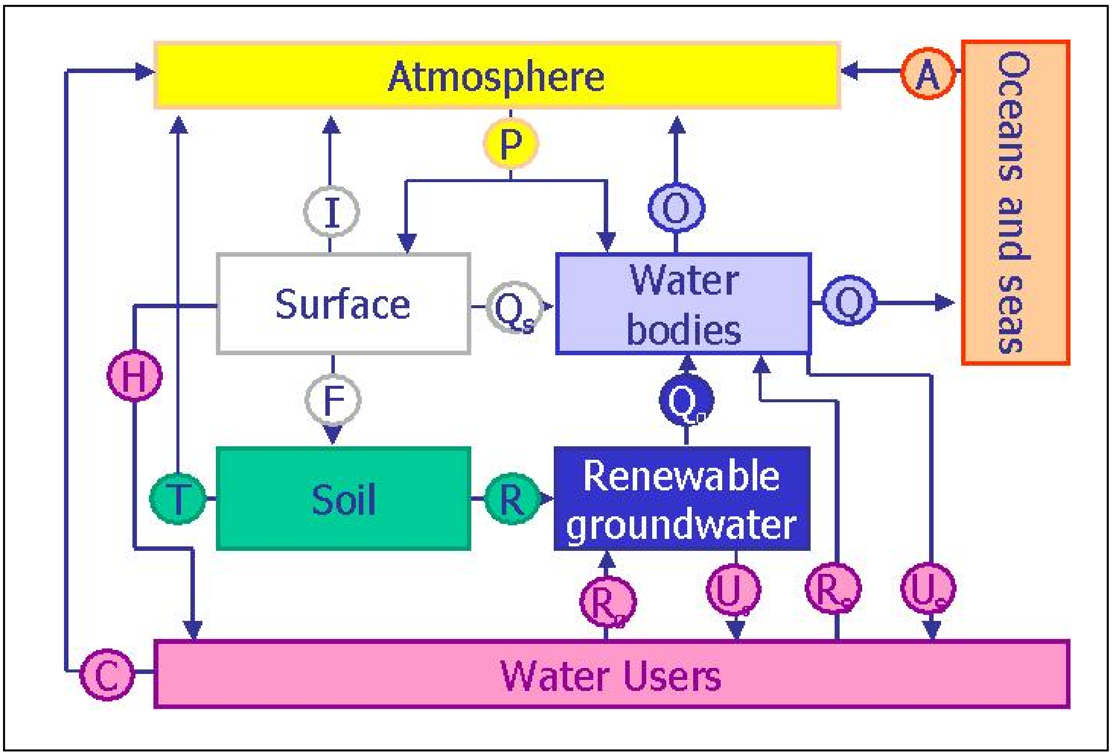


Figure 1.37: Human interference in the global water cycle

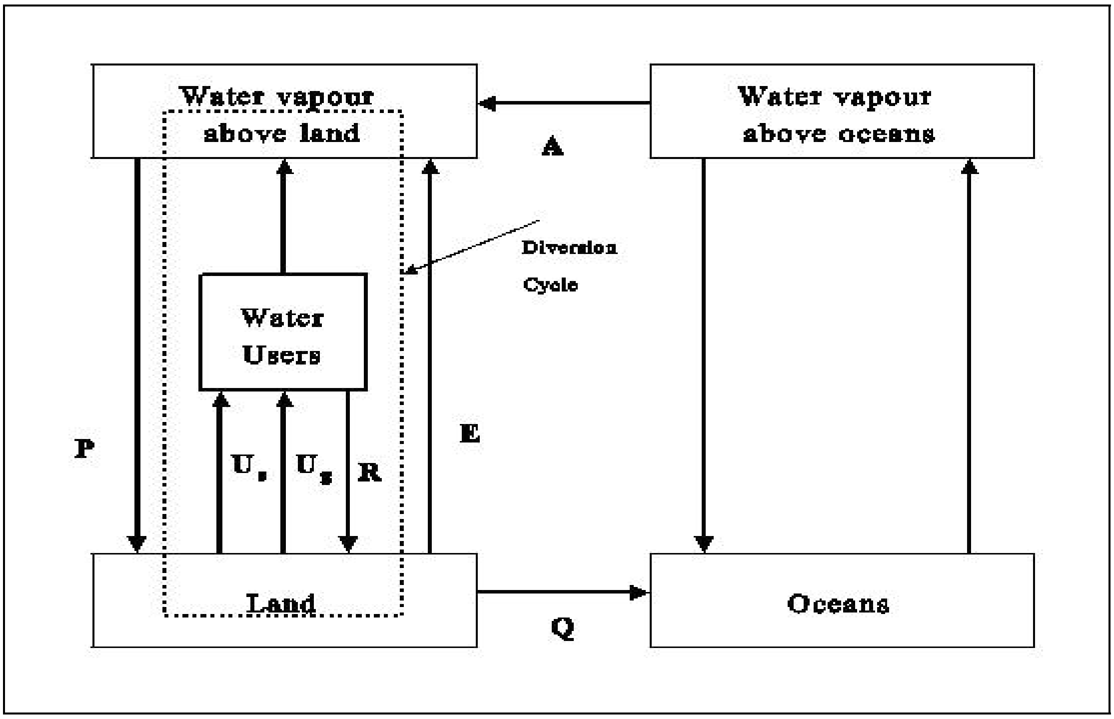


Figure 1.38: Scheme of the hydrological cycle with the diversion cycle (after Rodda and Matalas, 1987)

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).

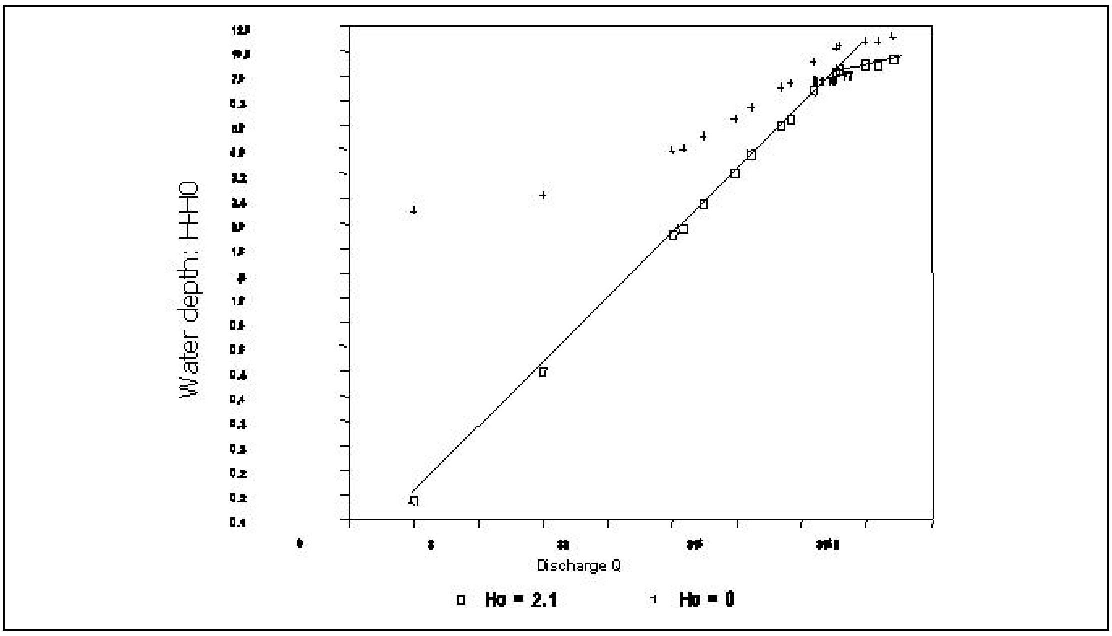


Figure 2.1: Rating curve in Limpopo river at Sicacate

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:

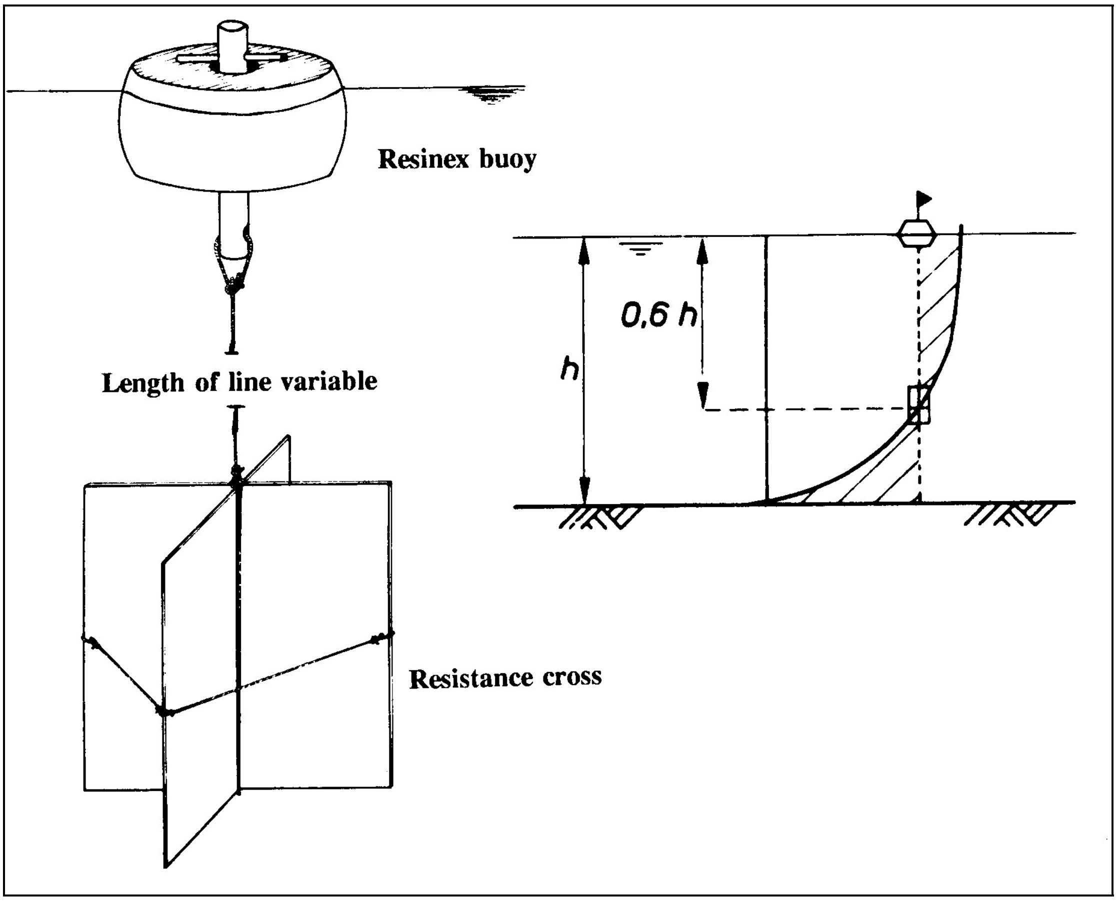


Figure 2.2: Float with resistance body, and location of the resistance body in the vertical

- 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.*

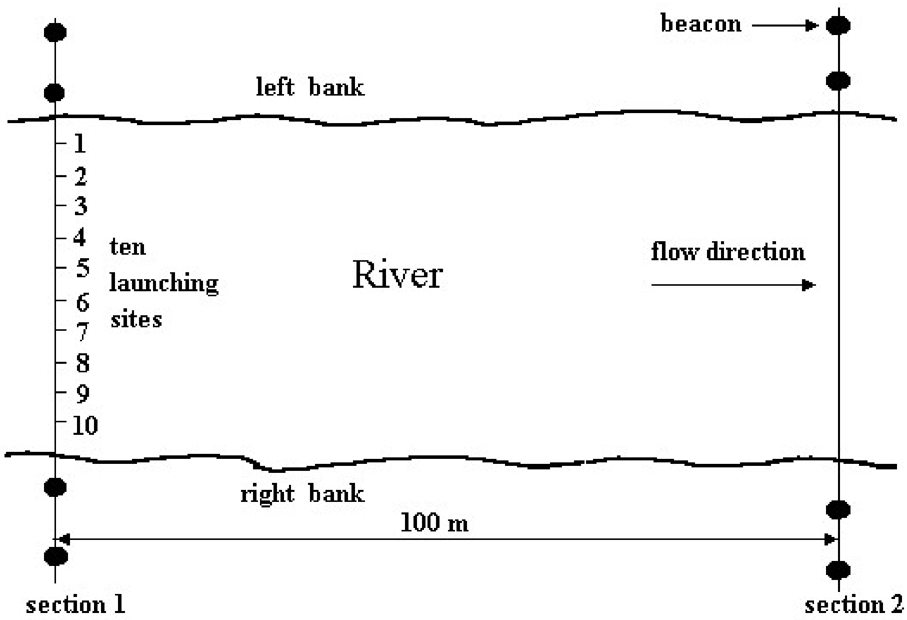


Figure 2.3: Layout of float measurement

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.

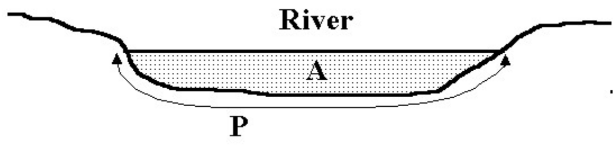


Figure 2.4: Cross-section of a river

The hydraulic calculations are based on Chézy's formula:

$$Q = CA\sqrt{R}\sqrt{S} = CK\sqrt{S}$$ *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:

$$K = \frac{\sum\left(A_i\sqrt{R_i}\right)}{N}$$ *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.

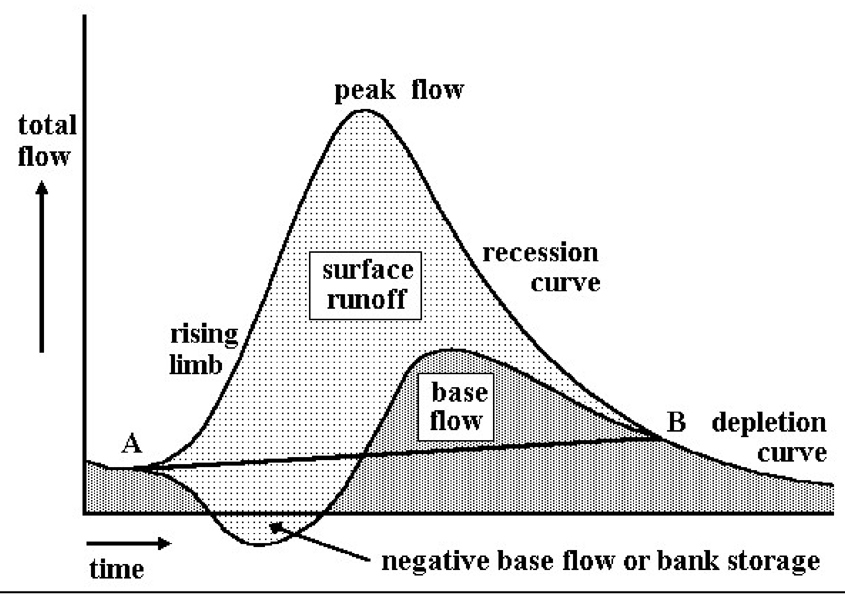


Figure 2.5: Components of hydrograph

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.

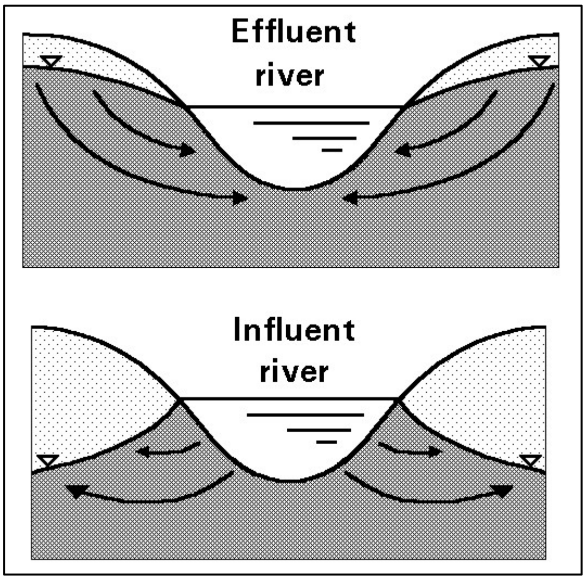


Figure 2.6: Influent and effluent streams

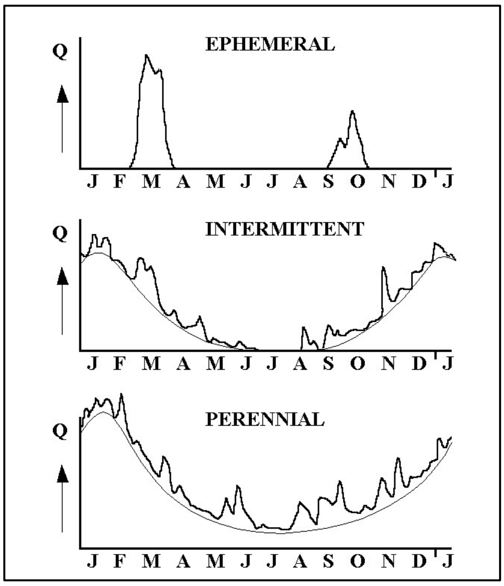


Figure 2.7: Classification of rivers

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.

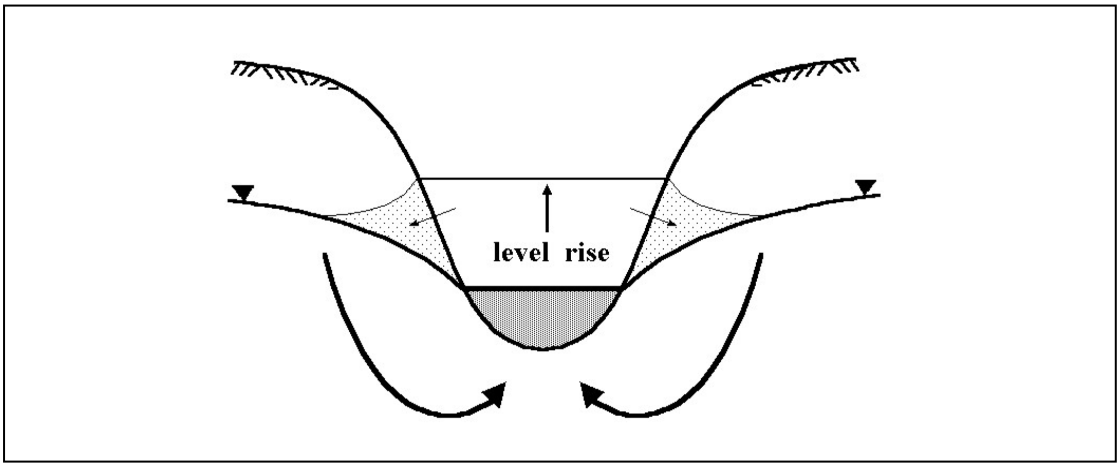


Figure 2.8: Temporary change of river, from effluent to influent due to a sudden rise of the water level

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.

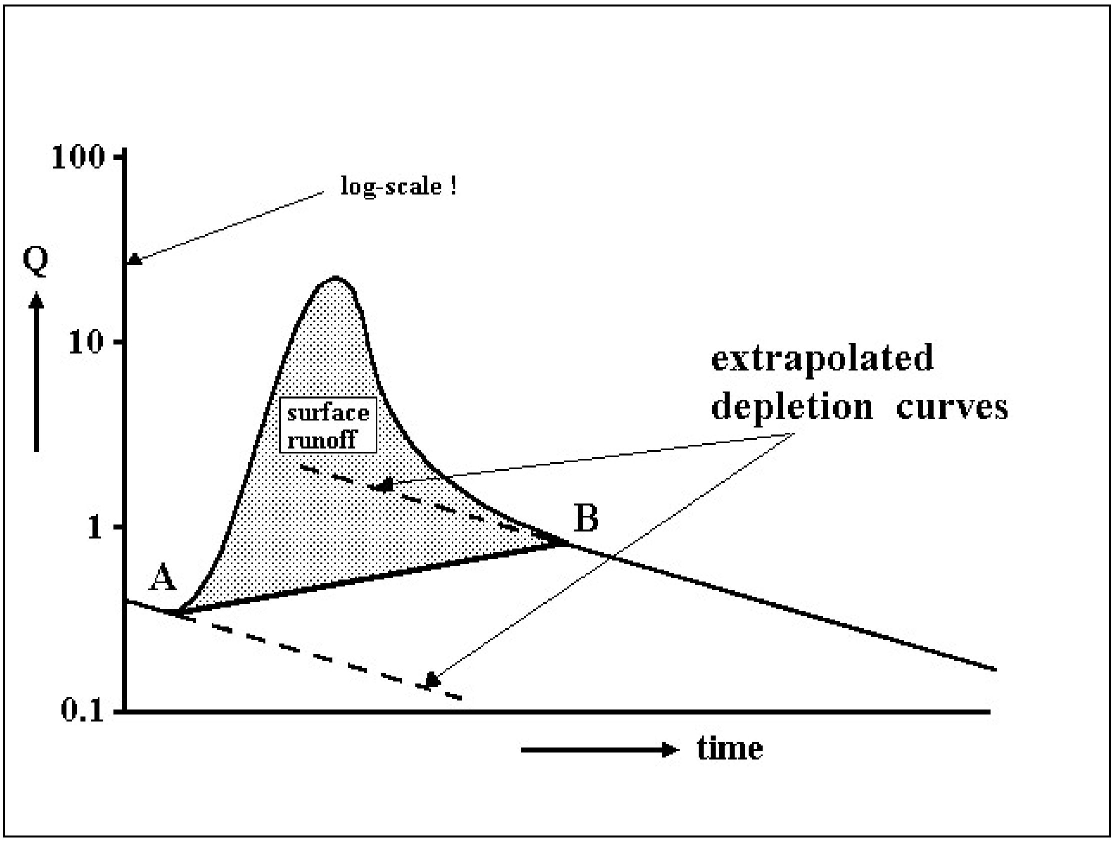


Figure 2.9: Hydrograph separation

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.

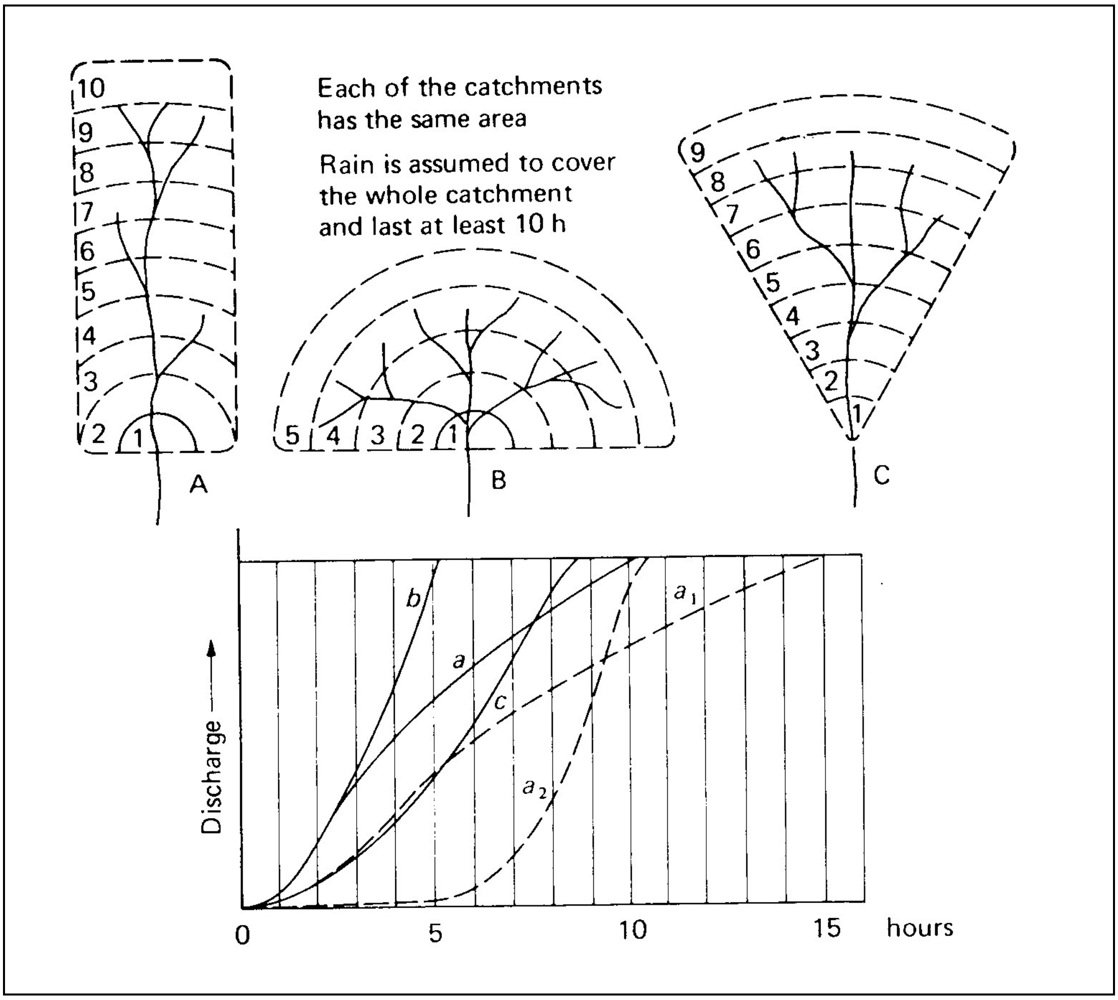


Figure 2.10: The effect of shape on catchment runoff (after Wilson, 1983)

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.

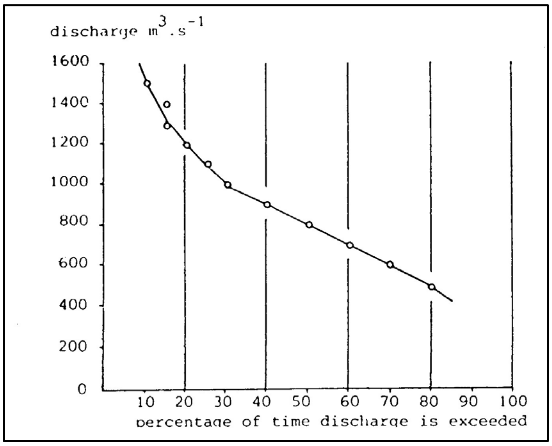


Figure 2.11: Flow duration curve for the example discussed in the text

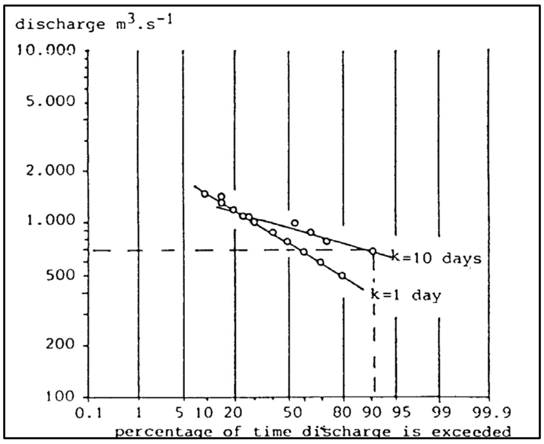


Figure 2.12: Flow duration curve on log-probability paper for flow durations of k = 1 and k = 10 days

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:

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

$$a = \frac{s_y}{s} \qquad \textit{Equation 2.13}$$

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:

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

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

N	y_m	s_y	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:

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

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.

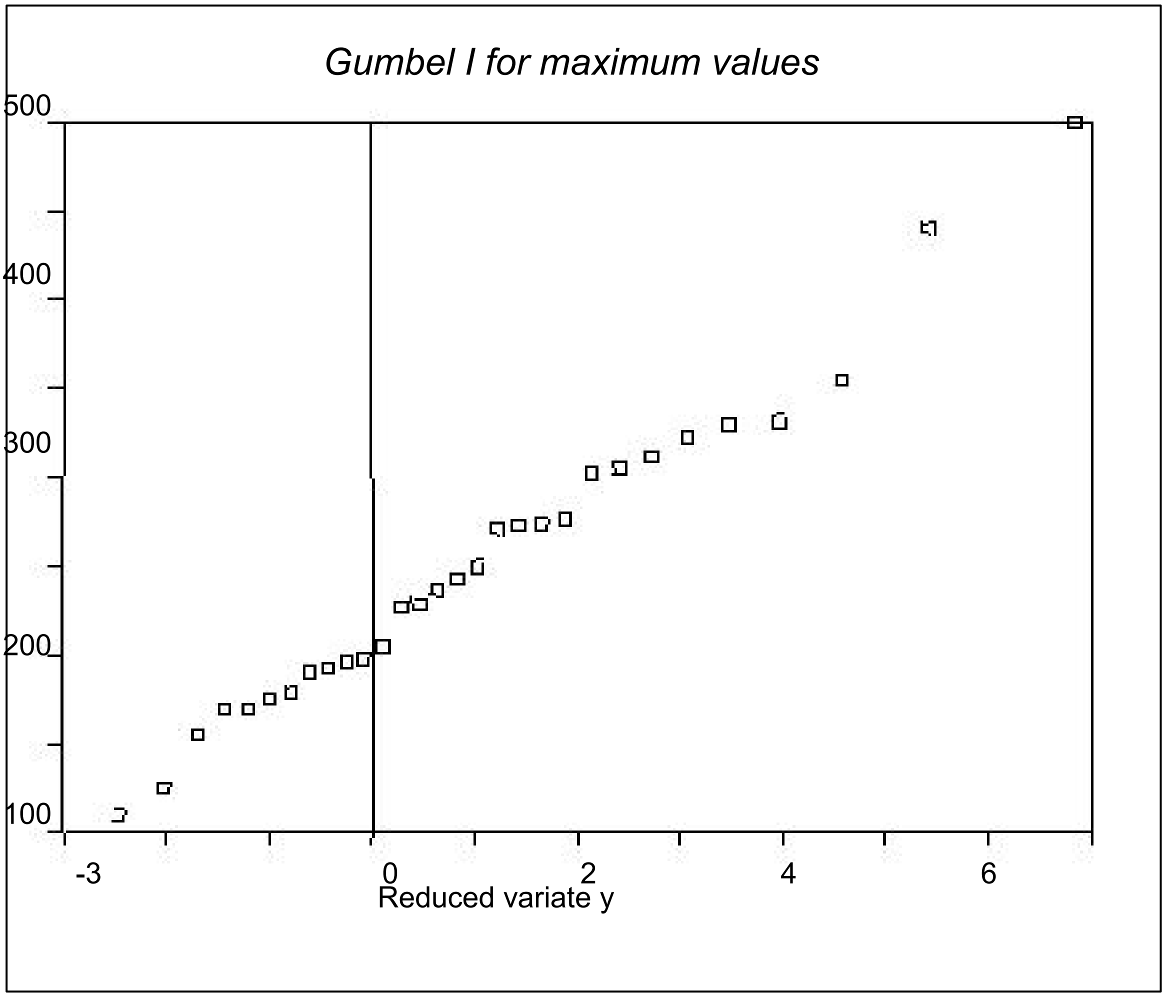


Figure 2.13: Gumbel probability distribution of annual maximum flood levels in the Sabie river at Machatuine

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:

$$p = 1 - \frac{i - 0.25}{n + 0.5}$$ *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:

$$y = -\ln\left(-\ln(p)\right)$$ *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:

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

and the reduced variate is given by:

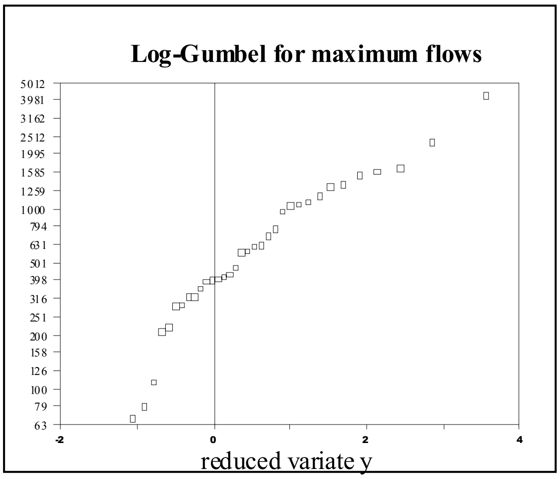


Figure 2.14: Log-Gumbel distribution for maximum flows in the Incomati river at Ressano Garcia

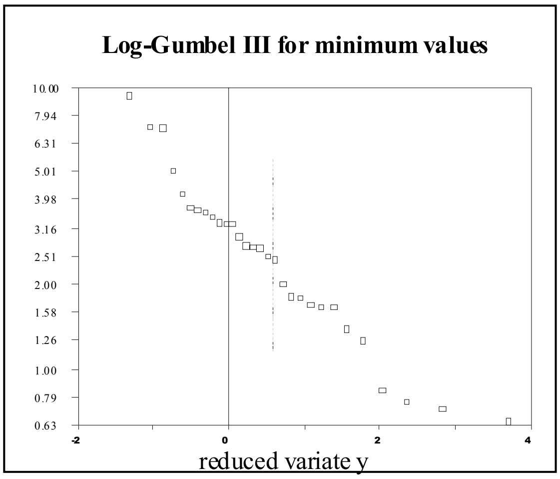


Figure 2.15: Log-Gumbel III analysis for minimum flow in the Sabie river at Machatuine

$$y = -\ln\left(-\ln(q)\right) \qquad \textit{Equation 2.19}$$

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.

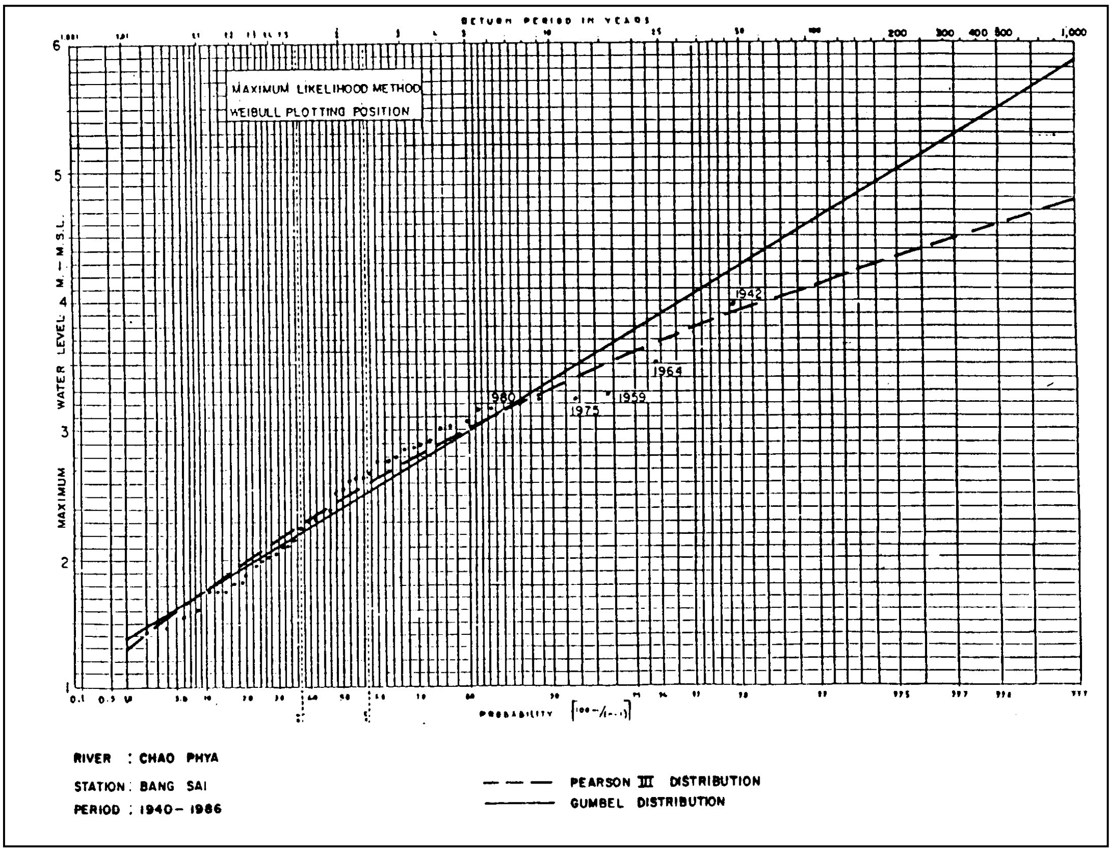


Figure 2.16: Gumbel and Pearson III distribution of annual flood levels in Bang Sai, Chao Phya river, Thailand (from Euroconsult, 1987)

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.

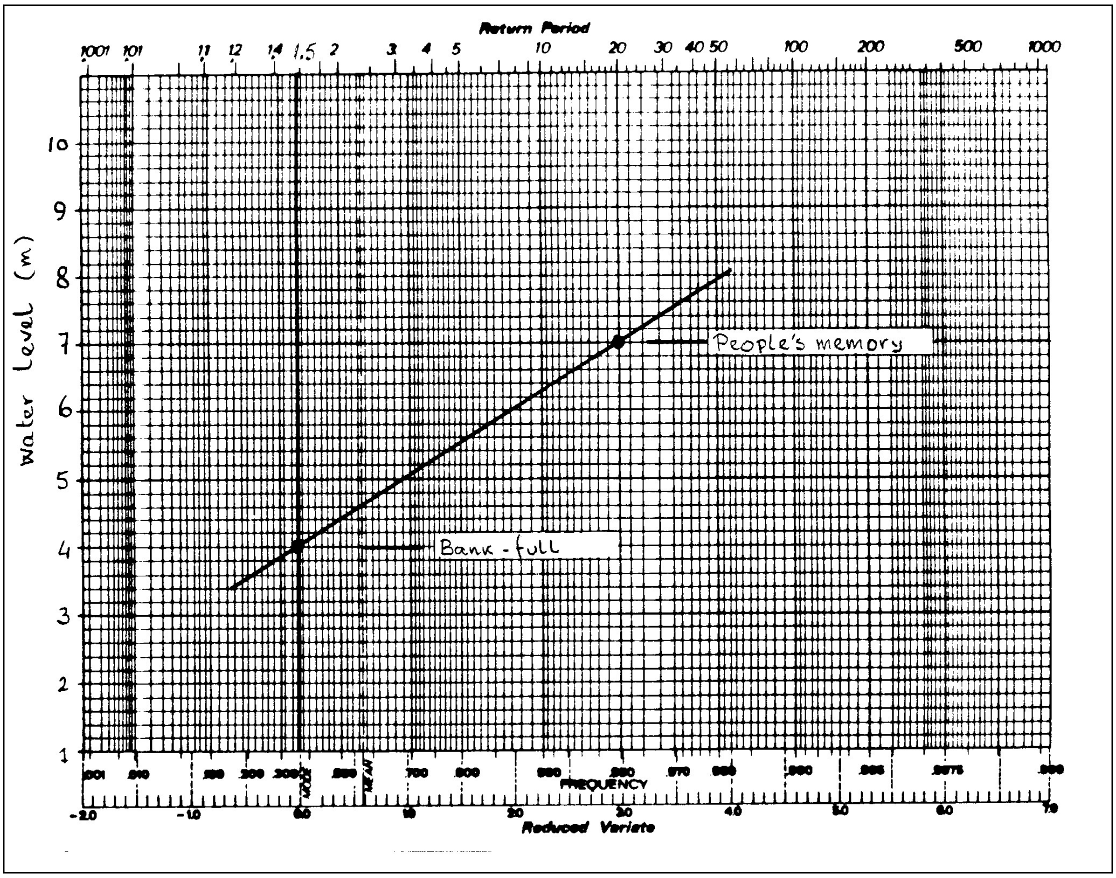


Figure 2.17: Gumbel distribution bases on merely field observation and interviews

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.

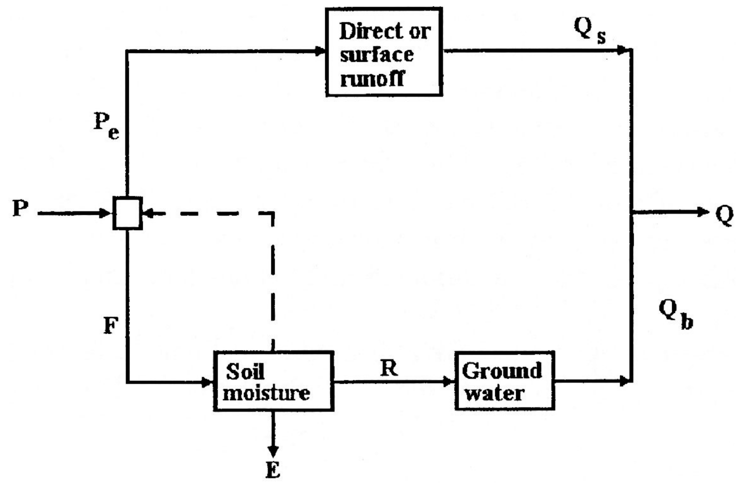


Figure 2.18: Simplified catchment model (Dooge, 1973)

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:

$$P_e = P - F - I$$ *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.

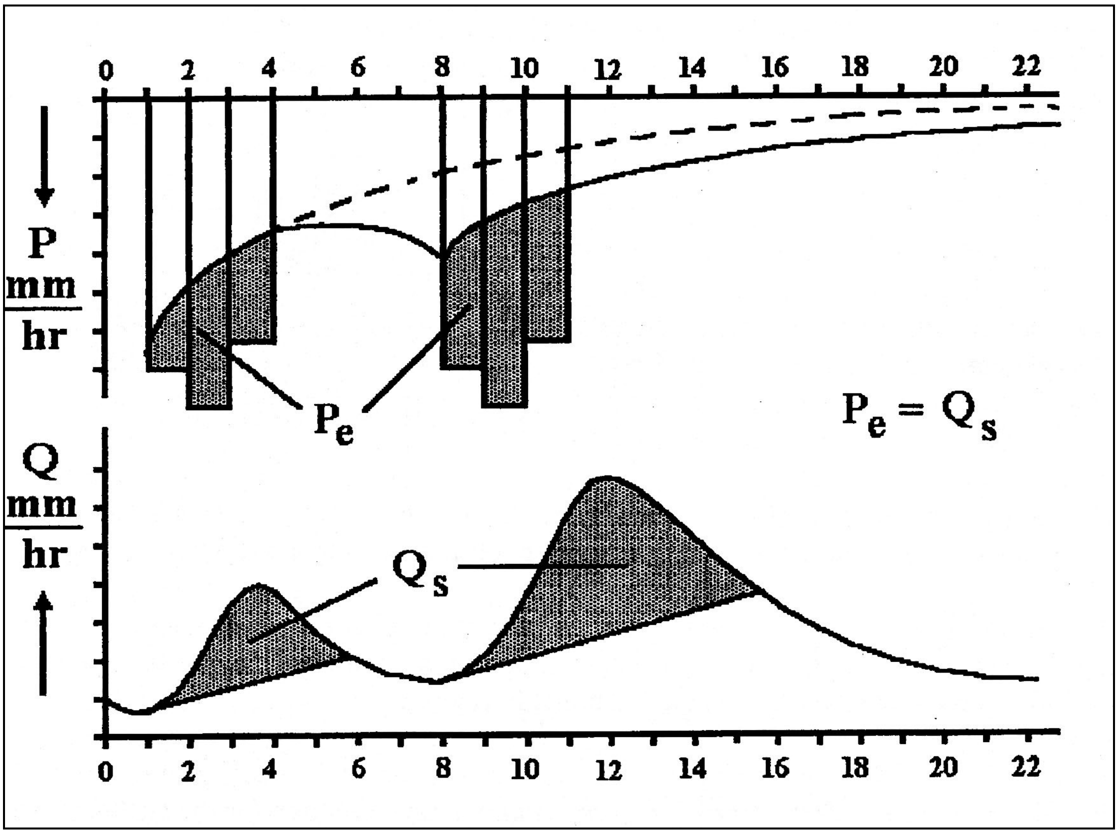


Figure 2.19: The effect of infiltration losses on the effective precipitation and direct runoff

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.

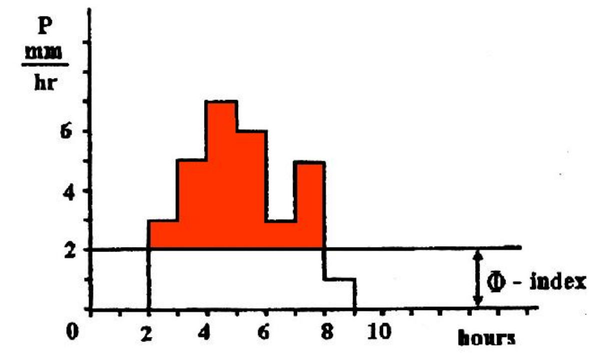


Figure 2.20: The Φ-index to indicate losses

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.

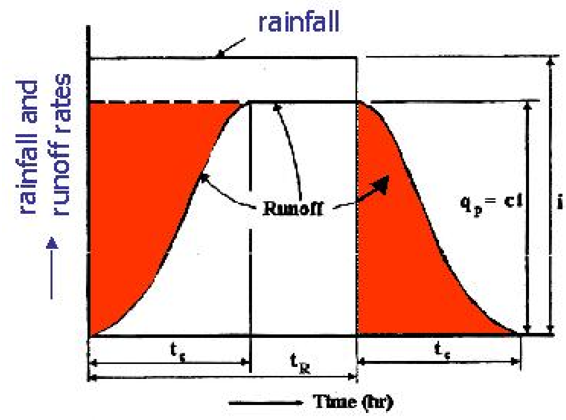


Figure 2.21: Runoff from uniform rainfall in the Rational Method (Gray, 1970)

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.

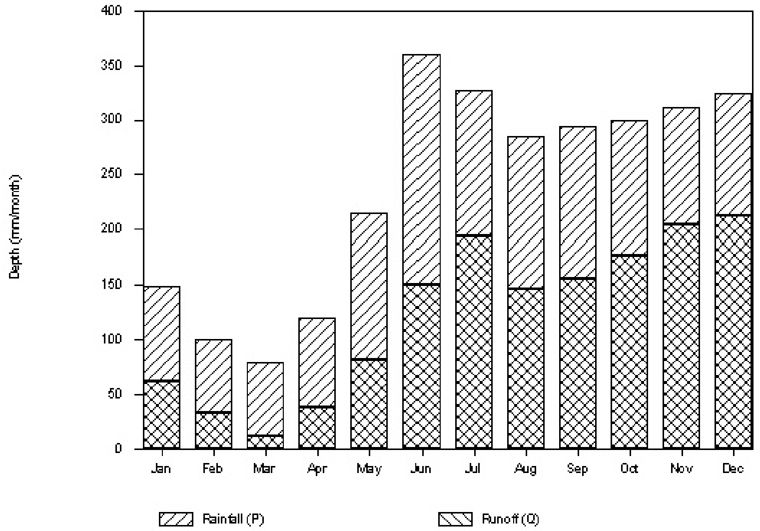


Figure 2.22: Monthly mean rainfall and runoff in the Cunapo catchment

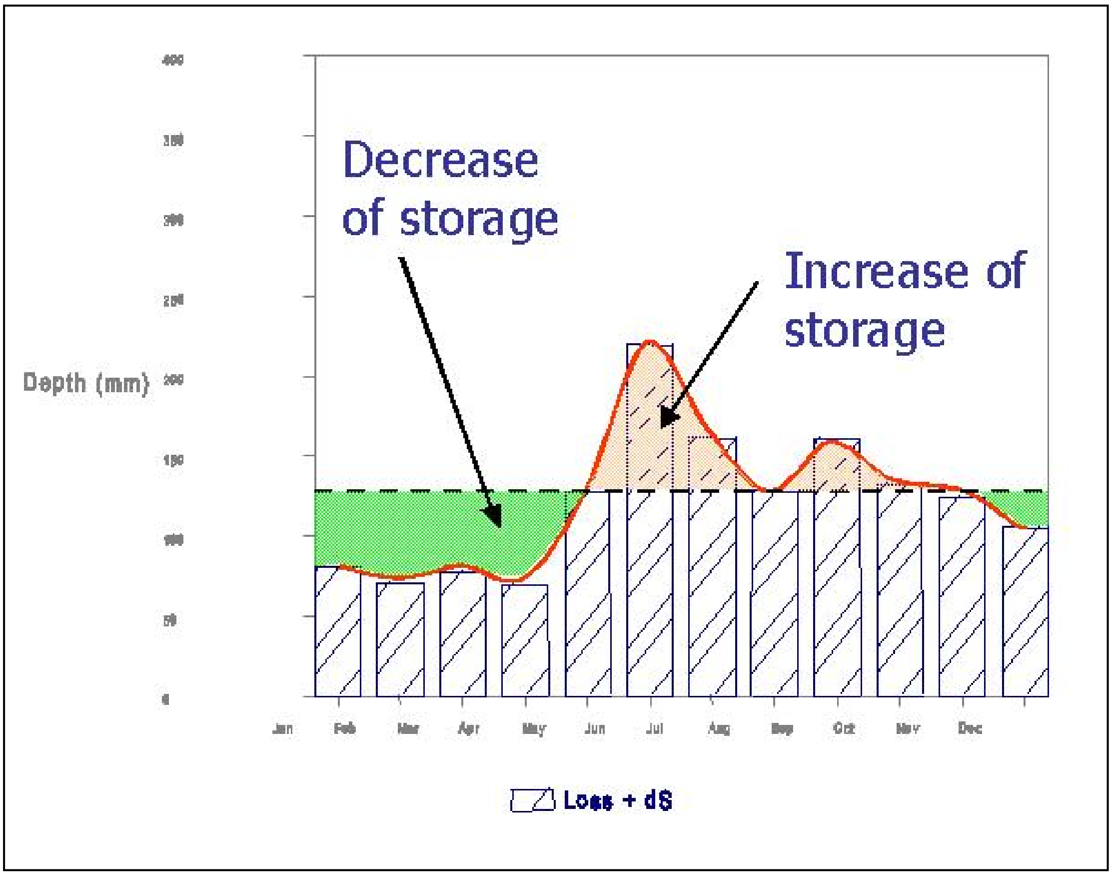


Figure 2.23: Mean monthly losses and change in storage in the Cunapo catchment

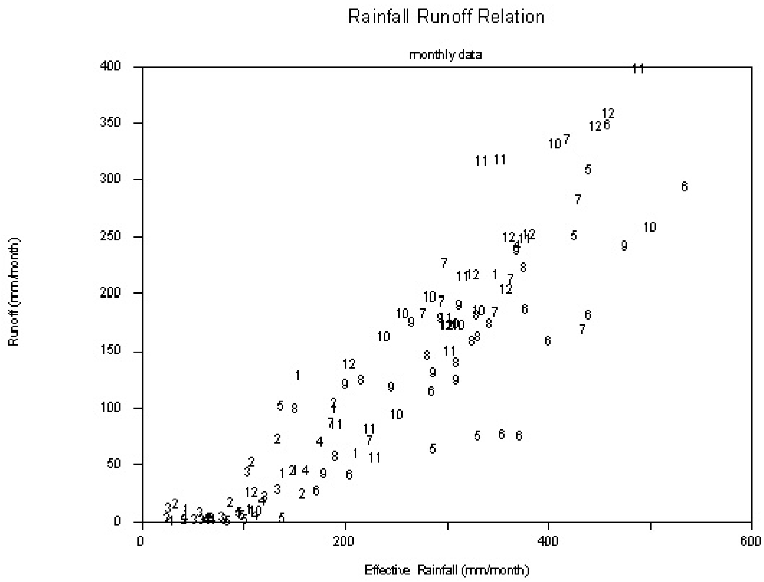


Figure 2.24: Rainfall plotted versus runoff in the Cunapo river basin

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:

$$C = \frac{\sum Q}{\sum \max(P - D, 0)}$$ *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:

$$\sum(b_i) \approx C$$ *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

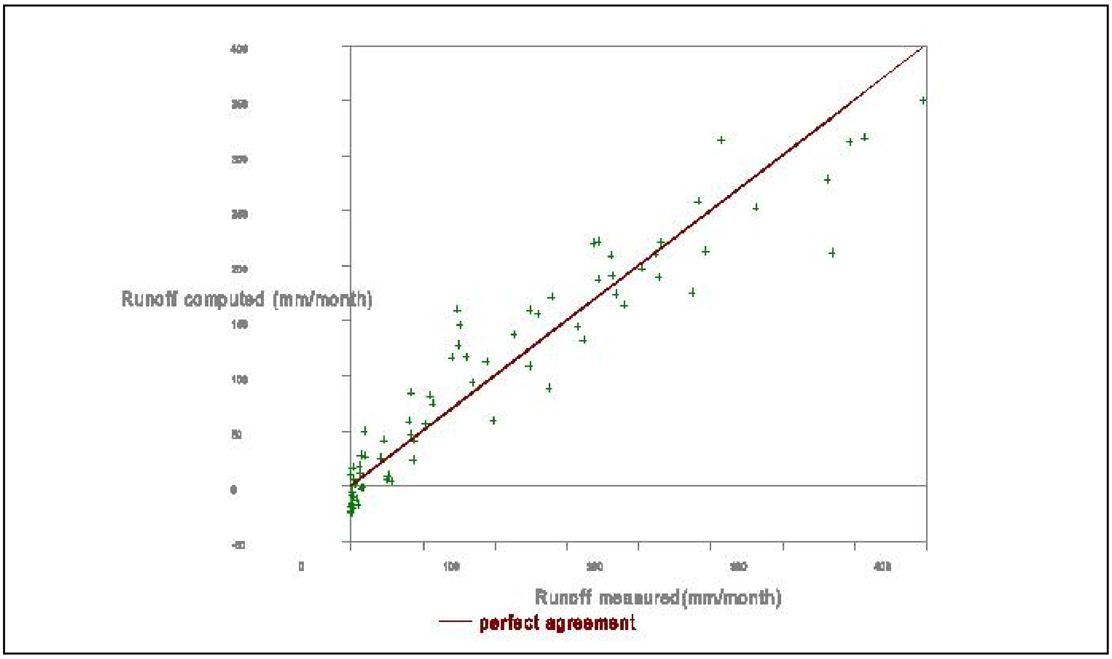


Figure 2.25: Measured versus computed runoff in the Cunapo catchment (D = 120)

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.

3 FLOOD PROPAGATION

3.1 Reservoir routing

The most important equation to describe the water balance of a reservoir is the water balance:

$$\frac{\mathrm{d}S}{\mathrm{d}t} = I - Q + A\left(P - E\right) \qquad \textit{Equation 3.1}$$

In finite differences form this equation can be written as:

$$S_1 = S_0 + \left(I - Q + A\left(P - E\right)\right)\cdot\left(t_1 - t_0\right) \qquad \textit{Equation 3.2}$$

where P is the rainfall, E is the evaporation, A is the surface area of the reservoir, S is the storage, I is the inflow and Q is the reservoir release (outflow). In Equation 3.2, the inflow, the rainfall and the evaporation are input data; the initial storage is an initial condition; the time is an independent variable. To determine the storage at a certain time t_1, the outflow and the surface area should be known. However, these depend on the water level in the reservoir, and thus on the storage to be computed. Equation 3.2, therefore, cannot be solved explicitly, but has to be solved iteratively.
For the solution of Equation 3.2, three extra equations are necessary to relate the outflow, the surface area and the storage to the water level. The following types of equations are widely applicable. They may have to be modified somewhat for application in a specific case.

$$A = A\left(H\right) \qquad \textit{Equation 3.3}$$

$$S = \int_{H_0}^{H} A\,\mathrm{d}H \qquad \textit{Equation 3.4}$$

$$Q = K\left(H - H_c\right)^c \qquad \textit{Equation 3.5}$$

Equation 3.3 is obtained from planimetering a topographical map. Often an exponential equation of the following type serves the purpose well:

$$A = A_0 \exp\left(b\left(H - H_0\right)\right) \qquad \textit{Equation 3.6}$$

where A_0 is the surface area at H_0. The equation plots a straight line on semi-logarithmic paper. But also a power function of the type:

$$A = A_0 + a\left(H - H_0\right)^b \qquad \textit{Equation 3.7}$$

can often be used. The equation plots a straight line on double logarithmic paper. Both Equations 3.6 and 3.7 are easily integrated to yield Equation 3.4.

Flood routing through a reservoir

In the case of a flood passing through the reservoir, the outflow hydrograph and the water levels in the reservoir can be computed. At the relative small time steps used for flood routing, the direct rainfall on the reservoir and the evaporation from the reservoir can be neglected.

The following procedure is commonly used in spillway design to determine the required dimensions of the spillway. Equation 3.5 is a spillway function. In the case of a free overflow spillway, the exponent c = 1.5 and the coefficient $K \approx 1.5*B$, where B is the spillway width; H_c is the crest level of the spillway. The equation can be modified to fit another spillway type, if required. The set of equations 3.2-3.5 can be solved iteratively:

1. Assume a certain spillway design by determining values for K, c and H_c. In most cases the simulation is started with a full reservoir:
$$H_0 = H_c$$

2. In a first approximation, Equation 3.2 is solved assuming that the outflow Q remains constant over the time step: $Q = Q(H_0)$ and that the effect of local rainfall and precipitation can be neglected in relation to the flood flows. The storage thus obtained is the first estimate of the storage S^*. The equation used is called the *predictor*:
$$S_1^* = S_0 + \left(I - Q(H_0)\right)\cdot\left(t_1 - t_0\right) \qquad [\mathrm{m}^3]$$

3. On the basis of S^*_1, H^*_1 is computed using the inverse of Equation 3.4:
$$H_1^* = H\left(S_1^*\right)$$

4. With this first estimate of the waterlevel at t_1, a new estimated storage at t_1 can be made, using the *corrector*:
$$S_1 = S_0 + \left(I - Q^*\right)\cdot\left(t_1 - t_0\right)$$
with
$$Q^* = K\left(\frac{H_1^* + H_0}{2} - H_c\right)$$

5. The corresponding reservoir level follows from $H_1 = H(S_1)$.

6. If necessary steps 4 and 5 are repeated (substitutions H_1 for H^*_1) until no further significant change in S_1 occurs.

7. Subsequently, the procedure is repeated in step 2 for the following time step:
$$S_2^* = S_1 + \left(I - Q(H_1)\right)\cdot\left(t_2 - t_1\right)$$
until the full flood wave has been simulated.

8. At the end of the simulation the maximum reservoir level and the maximum discharge are obtained corresponding to the assumed spillway design.

The iterative procedure described, based on the set of Equations 3.2 through 3.5, is easy to perform in a spreadsheet. Figure 3.1 is an example of the output of the spreadsheet model RESSIMFL.

Two observations can be made from studying Figure 3.1. Firstly, the inflow and the outflow hydrographs intersect at the point of maximum outflow; and secondly, the volume enclosed by the two curves left of the intersection is equal to the volume enclosed to the right of the intersection (assuming the water level is at the spillway crest at the start of the inflow hydrograph). The former volume is the part of the inflow, which is temporarily stored in the reservoir above the crest of the spillway, the latter volume is the release of that same amount. Before the point of intersection the storage increases; after the point of intersection the storage decreases.

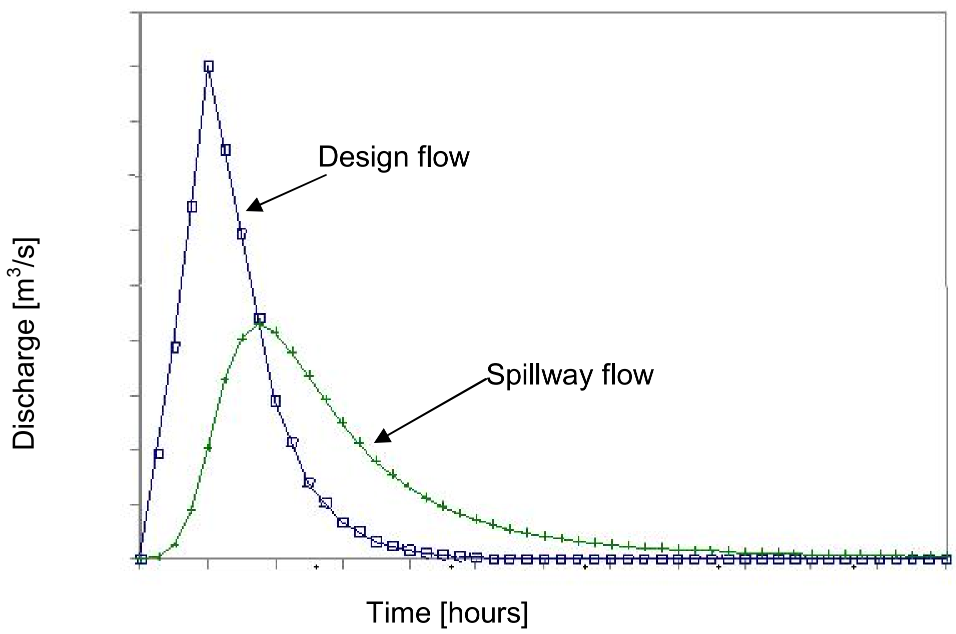


Figure 3.1: Inflow and outflow hydrograph of a reservoir

That the maximum outflow occurs at the point of intersection can be made clear by the following reasoning. It follows from Equation 3.1 that (neglecting the effect of rainfall and evaporation):

$$\frac{\mathrm{d}S}{\mathrm{d}t} = I - Q \qquad \textit{Equation 3.8}$$

At the point of intersection this results in:

$$\frac{\mathrm{d}S}{\mathrm{d}t} = 0$$

which because $S = S(H)$, and $\partial S / \partial H \neq 0$, results in:

$$\frac{\mathrm{d}H}{\mathrm{d}t} = 0$$

Thus the maximum water level in the reservoir occurs when inflow equals outflow. Since the outflow $Q = Q(H)$, it follows that:

$$\frac{\mathrm{d}Q}{\mathrm{d}t} = 0$$

Hence, the maximum outflow occurs at the maximum water level.

Reservoir yield analysis

The previous paragraphs refer to the routing of a single flood wave, for which the process time-scale is in the order of hours to days, depending on the size of the catchment and the reservoir.

For reservoir yield analysis flood waves are also crucial, as they contain most of the water that a reservoir is supposed to store for later use. However the process time-scale of reservoir yields is much longer than that of individual flood waves and is in the order of month to a year. Hence the time step in reservoir yield analysis generally varies from a week to month. Within such a month various smaller floods may have occurred. However at this process scale these variations are not relevant.

In reservoir yield analysis, the same equation (Equation 3.2) is used as for flood routing. In this case however, the effect of rainfall and evaporation can no longer be disregarded.

In yield analysis, the time series of *P*, *E* and *I* are known values. The variation of the storage *S* over time and the reservoir outflow, or release, *Q* are the unknown parameters. The reservoir release is composed of the draft, *D*, being the planned or envisioned release, and the spill over the spillway, *L*.

$$Q = D + L \qquad \textit{Equation 3.9}$$

The way the yield analysis is approached is by assuming a certain draft, possibly as a function of time, *D(t)*, on the basis of which the reservoir simulation is made. The spill, *L(t)*, follows from the reservoir operation.

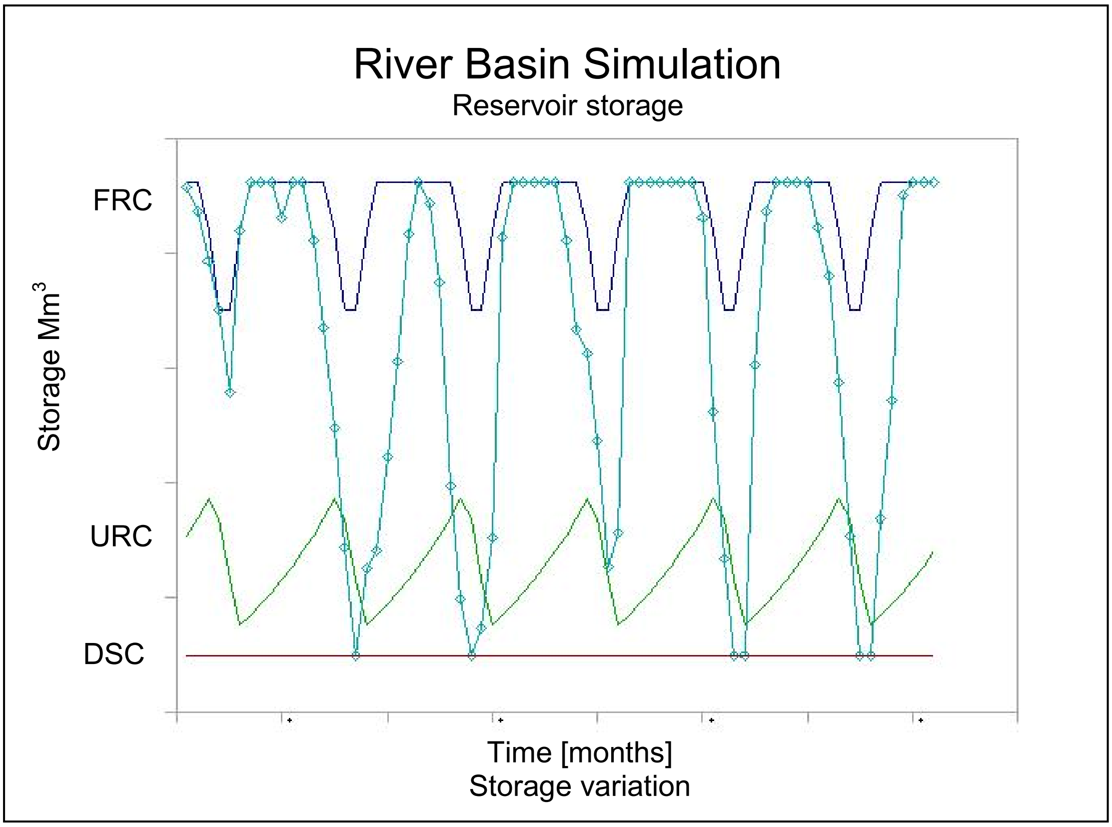


Figure 3.2: Reservoir operating rules (rule curves) and simulating storage variation by the model WAFLEX

The solution of Equation 3.2 is only possible if operating rules are used, that determine the release as a function of storage and a set of rule curves that can be functions of time:

$$Q = Q\left(S, RC_1\left(t\right)\right), RC_2\left(t\right), \ldots\ldots RC_n \qquad \textit{Equation 3.10}$$

Although, in principle, many different operating rules can be used, most reservoirs follow the basic operating rules of the following example. Figure 3.2 shows three operating rules:

- The Flood Rule Curve (FRC), which is a hard boundary (meaning it may not be crossed[1]). The FRC represents storage levels, which are a function of time, FRC(t). If the storage is more than FRC, all additional water is spilled (L*dt=S-FRC):
  If S > FRC, then Q = D + (S - FRC)/dt and S = FRC

- The Utility Rule Curve, URC(t), which is a soft boundary (it may be crossed). If the storage reaches, or crosses, the URC, the release from the reservoir is reduced by a certain rationing percentage r:
  If S < URC, then $Q = r * D$ and the water balance is redone with
  $Q = r * D$

- The Dead Storage Curve, DSC(t), which is a hard boundary. The storage may never drop below this level as a result of releases, only due to evaporation. The dead storage requirement is often for environmental or ecological reasons. If, as a result of the draft, the storage drops below the DSC, then the release is reduced in the following way:
  If S < DSC, then Q = Max (0, D - (DSC - S)/dt) and redo water balance with this release.
  As a result of evaporation it is possible that D - (DSC - S)/dt < 0. In that case the release is zero and the water balance results in storage below DSC.

The areas between the curves are generally called zones, and the drawing of rule curves is also called zoning of the reservoir storage. In Figure 3.2, zone 1 is the area under DSC; zone 2 is the area between DSC and URC; zone 3 is the area between URC and FRC; and zone 4 is the area above FRC. The line indicated by the symbols is the storage variation as simulated by the spreadsheet model WAFLEX (Savenije, 1995b).

[1] The FRC is only a hard boundary at the time scale used for reservoir simulation (a time step of a week, decade or month). During day-to-day operation, the FRC can be crossed temporarily during the spilling operation.

During the simulation, the following steps are followed:

1. Establish a draft pattern *D(t)* and assume that the release *Q=D(t)*. The inflow *I* is taken as the average over the time step $dt=(t_1-t_0)$.
2. Solve the water balance equation 3.2 in numerical form:

$$S_1 = S_0 + (I-Q)(t_1-t_0) + (P-E)(t_1-t_0)A(H_0) \qquad \textit{Equation 3.11}$$

   where $S_1 = S(t_1)$ and $A(H_0)$ is the inundated area at water level $H(t_0)$. Although it would be more correct to use the inundated area as a function of the average water level between t_1 and t_0, which would require an extra iteration in the computation, such a procedure is generally not necessary as the error made by the rainfall and evaporation term is expected to be small.
3. Check the operating rules. If necessary the release *Q* and the storage S_1 should be adjusted according to the operating rules.
4. Now that S_1 and *Q* are known, the computation for the next time step can be started similar to step 2:

$$S_2 = S_1 + (I-Q)(t_2-t_1) + (P-E)(t_2-t_1)A(H_1)$$

5. The above steps are repeated until the end of the data series is reached. At the end of the simulation the shortage of water is computed, as well as the amount of water spilled. On the basis of this information a decision can be taken to adjust the release pattern, *D(t)*, or to use other rule curves.

The order to be able to draw quantitative conclusions from the reservoir yield analysis, the starting value of the storage, S_0, should be equal to the end value, S_n. This can generally be achieved by one iteration where "lay" S_n is used as S_0, provided the reservoir volume is not too large in relation to the inflow.

3.2 Flood routing in natural channels

The volume of water in a channel at any instant is called channel storage *S*. The most direct determination of *S* is by measurement of channel volume from topographic maps. However lack of adequately detailed maps plus the need to assume or compute a water-surface profile for each possible condition or flow in the channel makes this approach generally unsatisfactory. Since Equation 3.2 involves only *S* ($S = S_1 - S_0$), absolute values of storage need not be known. Values of S can be found by solving Equation 3.2, using actual values of inflow and outflow (Figure 3.3). For flood routing, the effect of rainfall and evaporation on the storage in the channel reach can be disregarded. The hydrographs of inflow and outflow for the reach are divided into short time intervals, average values of *I* and *Q* are determined for each period, and values of *S* computed by subtracting *Q* from *I*.

Storage volumes are computed by summing the increments of storage from any arbitrary zero point.

When values of *S* computed as just described are plotted against simultaneous outflow (Figure 3.4), it usually appears that storage is relatively higher during rising stages than during falling stages. As a wave front passes through a reach, some storage increase occurs before any increase in outflow. After the crest of the wave has entered the reach, storage may begin to decrease although the outflow is still increasing. Nearly all methods of routing stream flow relate storage to both inflow and outflow in order to allow for these variations.

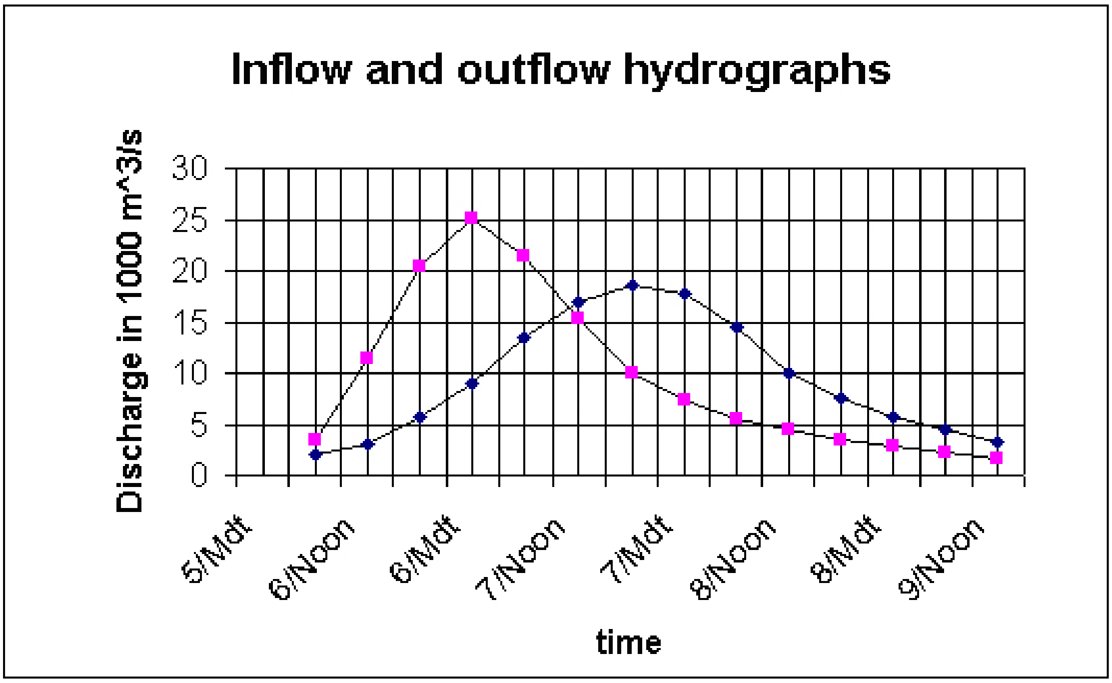


Figure 3.3: Inflow and outflow hydrographs for a reach of river, showing method of calculating channel storage

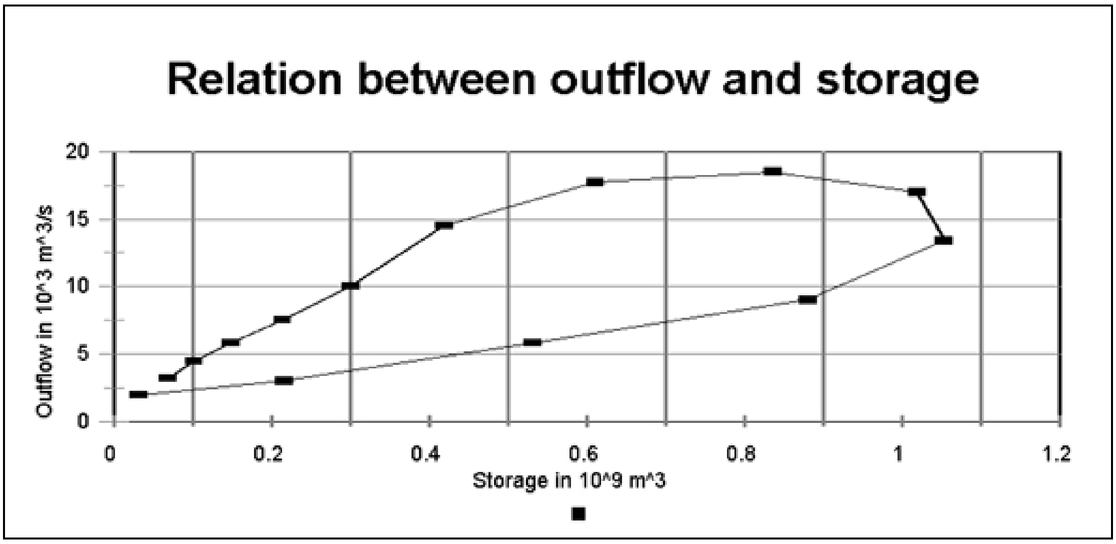


Figure 3.4: Relation between outflow and storage for the data of Figure 3.3

A very widely utilized assumption is that storage is a function of weighted inflow and outflow, which yields the Muskingum equation:

$$S = K\left[xI + (1-x)Q\right] \qquad \textit{Equation 3.12}$$

where *S*, *I*, and *Q* are simultaneous values of storage, inflow, and outflow over a reach Δ*x*. The dimensionless constant *x* indicates the relative importance of *I* and *Q* in determining storage, and *K* is a storage constant with the dimension of time. The value of *K* approximates the time of travel of the wave through the reach. If the celerity of propagation of the wave is *c*, and the length of the reach considered is Δ*x* , then *K*= Δ*x/c*. A flood wave in a river behaves as a mass wave with the equation:

$$c = \frac{\mathrm{d}q}{\mathrm{d}h} \approx 1.67v \qquad \text{Equation 3.13}$$

where q is the discharge per unit width, v is the cross-sectional average flow velocity and h is the cross-sectional average depth of flow. If the discharge obeys Mannings formula in a rectangular cross-section, then the wave celerity is about 60-70% higher than the average flow velocity v. Hence the wave celerity depends on the flow velocity and K is not constant: it is larger for larger floods.

determination of x

In theory, the constant x varies from 0 to 0.5. Cunge (1969) showed that x can be related to physical parameters:

$$x = 0.5\left(1 - \frac{q}{S_b cL}\right) \qquad \text{Equation 3.14}$$

where S_b is the bottom slope.

Since dS/dt = I-Q, differentiating Equation 3.12 yields

$$I - Q = \frac{dS}{dt} = K\left[x\frac{dI}{dt} + (1-x)\frac{dQ}{dt}\right] \qquad \text{Equation 3.15}$$

If I = Q, then at the point of intersection:

$$x = \frac{dQ/dt}{dQ/dt - dI/dt} \qquad \text{Equation 3.16}$$

which permits estimating x from concurrent inflow and outflow records. For a reservoir where Q = f(S), dS/dt and dQ/dt must be zero when I = Q. Therefore x for this case is zero. A value of zero indicates that the outflow alone determines storage (as in a reservoir). When x = 0.5, inflow and outflow have equal influence on storage. In natural channels x usually varies between 0.1 and 0.3.

Values of K and x for a reach are usually determined by trial. Values of x are assumed, and storage is plotted against xI + (1-x)Q. The value of x which results in the data conforming most closely to a straight line is selected (Figure 3.5). The travel time K is the slope of the line relating S to xI+(1-x)Q.

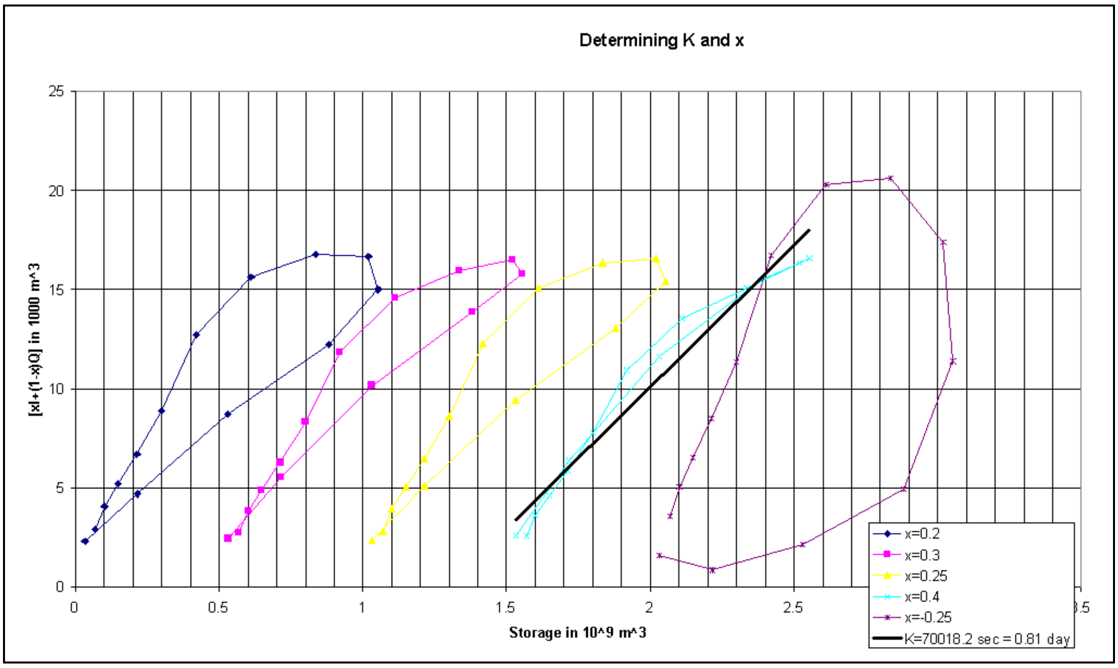


Figure 3.5: Method of determining K and x for the Muskingum method of routing

The Muskingum routing equation is found by substituting Equation 3.11 and solving for Q_2,

$$Q_2 = c_0 I_2 + c_1 I_1 + c_2 Q_1 \qquad \textit{Equation 3.17}$$

$$c_0 = \frac{-2x + \Delta t / K}{2(1-x) + \Delta t / K} \qquad \textit{Equation 3.18a}$$

$$c_1 = \frac{2x + \Delta t / K}{2(1-x) + \Delta t / K} \qquad \textit{Equation 3.18b}$$

$$c_2 = \frac{2(1-x) - \Delta t / K}{2(1-x) + \Delta t / K} \qquad \textit{Equation 3.18c}$$

$$c_0 + c_1 + c_2 = 1 \qquad \textit{Equation 3.18d}$$

The significance of Equation 3.18d may be seen if it is noted that, for steady flow ($I_1 = I_2 = Q_1 = Q_2$), Equation 3.17 can be correct only when the sum of the constants is unity. It is important that K and t be in the same units when used in Equations 3.18. When Q and I are in m^3/s and storage is computed in cubic meters, the units of K and t are seconds.

Table 3.1: Application of the Muskingum method

Date	Hour	I, m^3/sec	C_0I_2	C_1I_1	C_2Q_1	Q, m^3/sec
4/9	6 a.m.	1000	…	…	…	1000
	Noon	2400	*-408*	*530*	*640*	*762**
	6 p.m.	3900	*-663*	*1272*	*488*	*1097*
	Midnight	5000	*-850*	*2067*	*702*	*1919*
4/10	6 a.m.	4900	*-833*	*2650*	*1228*	*3045*
	Noon	4000	*-680*	*2597*	*1949*	*3866*

Note: Computed values are in Italic. Coefficients used are c_0 = -0.17, c_1 = 0.53 and c_2 = 0.64.
* The first computed outflow often drops when inflow increases sharply. This is simply disregarded.

The application of the Muskingum method is illustrated in Table 3.1. Values of c_0, c_1 and c_2 were computed by substituting K = 0.82 day, x= 0.3 (Figure 3.5), and Δt = 6 hr in Equation 3.16a, b, c. Values of I are tabulated, and the products c_0I_2 and c_1I_1 are computed. With an initial value of Q_1 given or estimated, the product c_2Q_1 is calculated and the three products added to obtain Q_2. The computed value of Q_2 becomes Q_1 for the next routing period, and another value of Q_2 can be determined. The process continues as long as values of I are known. This routing is easily performed in a spreadsheet.

Storage in a river reach actually depends on water depths. The assumption that storage is correlated with rates of flow is a valid approximation only when stage and discharge relations are closely correlated. Because of the hysteresis effect, this is not completely correct. In streams, with a complex slope-stage-discharge relation, more complex routing methods (and more data, particularly on geometry) are required to obtain satisfactory accuracy. The methods described in the preceding sections assume that the longitudinal profile of the water surface in a reach is the same every time a given combination of inflow and outflow occurs. This is also an approximation but is usually sufficiently precise if the reach is not excessively long. In general, the methods, which have been described, are satisfactory on the great majority of streams.

In making a Muskingum routing, one has to take into account the value of the Courant number: $N_{Cr}=c\Delta t/\Delta x$, which should always be less than 1. If the flood wave can travel through the reach Δx in a time less than the time step Δt, then computational instabilities may occur. Hence:

$N_{Cr} = \dfrac{c\Delta t}{\Delta x} \le 1$, or with $K=\Delta x/c$:

$$\Delta t \le \frac{\Delta x}{c} \le K$$

Kinematic routing

Kinematic routing involves the simultaneous solution of the continuity equation:

$$Q = I - L\frac{\Delta A}{\Delta t} \qquad \textit{Equation 3.19}$$

and a flow equation such as the Manning equation:

$$Q = KAR^{2/3}S^{1/2} \qquad \textit{Equation 3.20}$$

where A is the cross-sectional area, L the length of the reach, and hence $L\Delta A$ is the change in storage. In kinematic routing the energy slope S is taken as the bed slope S_b and an iterative solution is used until both equations yield consistent values of Q. A mean cross section of the reach is a required input. Kinematic routing is typically performed on a computer.

In the form described above, kinematic routing is subject to all of the assumptions of hydrologic routing and its principal advantage is an ability to deal with non-linear storage-stage relations on the basis of a measured cross section. The reliability of kinematic and hydrologic routing are roughly the same. Neither method works well on very flat slopes where second-order terms in the energy equation may exceed the bed slope, nor on very steep slopes where supercritical flow occurs.

The rate of convergence of the solution depends on how well Q_2 is estimated for the first trial. Many assumptions are possible such as $Q_2 = Q_1$ or $Q_2 = Q_1 + (Q_1 - Q_0)$. Another possibility is to use a very short routing period such that Q is small and eliminate the iteration.

Local inflow

The previous discussion has considered the routing of inflow entering at the head of a reach. In almost all streams there is additional inflow from tributaries, which enter the main stream between the inflow and outflow points of the reach. Occasionally this *local inflow* is small enough to be neglected, but often it must be considered. The conventional procedures are (1) add the local inflow to the mainstream inflow, and consider the total as I in the routing operation, or (2) route the main-stream inflow through the reach, and add the estimated local inflow to the computed outflow. The first method is used when the local inflow enters the reach near its upstream end, while the second method is preferred if the greater portion of the tributary flow joins the main stream near the lower end of the reach. The local inflow might also be divided into two portions, one part combined with the mainstream inflow and the remainder added to the computed outflow.

If the lateral inflow is known, one can use the four-point Muskingum method where there is a c_3 to be multiplied with the lateral inflow Q_L (see Ponce, 1979).

$$Q_2 = c_0 I_2 + c_1 I_1 + c_2 Q_1 + c_3 Q_L \qquad \textit{Equation 3.21}$$

$$c_3 = \frac{2\Delta t / K}{2(1-x) + \Delta t / K} \qquad \textit{Equation 3.22}$$

The hydrograph of lateral inflow may be estimated by comparison with stream flow records on tributary streams or by use of rainfall-runoff relations and unit hydrographs. In working with past data, the total volume of local inflow should be adjusted to equal the difference between the reach inflow and outflow, with proper allowance for any change in channel storage during the computation period. Since local inflow may be a small difference between two large figures, slight errors in the stream flow record may result in large errors in local inflow, even to the extreme of indicating negative local inflows.

There also is a so-called three parameter Muskingum method which allows for lateral inflow or lateral seepage loss as a percentage of the flow (O'Donnell, 1985):

$$\frac{\mathrm{d}S}{\mathrm{d}t} = I(1+\alpha) - Q \qquad \textit{Equation 3.23}$$

$$S = K\left[x(1+\alpha)I + (1-x)Q\right] \qquad \textit{Equation 3.24}$$

4 HYDROLOGY OF COASTAL AREAS

4.1 Introduction

In the context of this subject, coastal areas comprise the lowland areas bordering estuaries and deltas, or, more in general, the lower reaches of rivers, and coastal marshlands and lagoons, which have a predominantly marine character.

There are two reasons to pay special attention to the hydrologic and water control of such coastal areas as distinct from the upper portions of a river basin: A. the special nature of the hydrologic processes and B. the economical significance of coastal areas.

A1. In the upper portions of a river the water levels and flows are governed by the precipitation upstream of the station under consideration; in a delta on the other hand the water levels and flows are governed by the conditions at the two ends: the water levels and flows at the apex (the point where the river starts branching off into distributaries or branches) and the water levels of the recipient basin (sea, ocean, inland sea or lake).

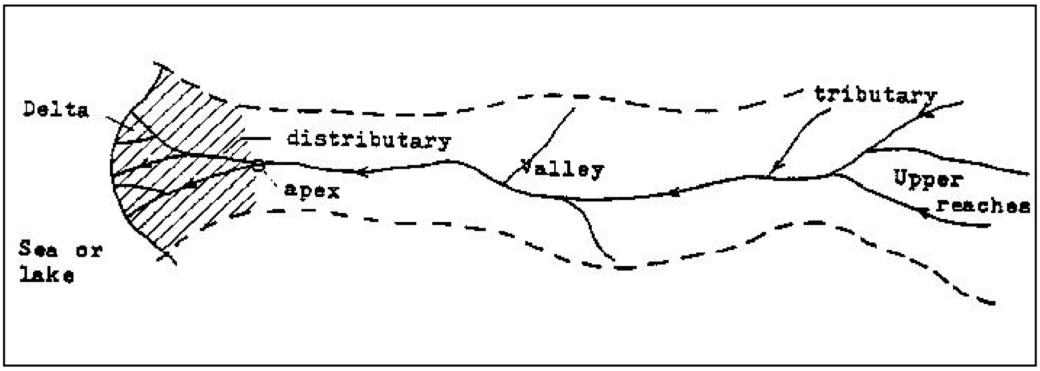


Figure 4.1: Delta

A2. In most cases the rainfall on the coastal area has little effect on the discharges and levels of the distributaries because the upstream discharge (or freshet) at the apex and the flows generated by the tides are much larger than the discharge generated by the run off from the coastal areas. The latter comes into the picture in the design of drainage systems of embanked areas.

A3. Coastal areas are characterized by small gradients (from 1.10^{-5} to 50.10^{-5}) and are exposed to extensive flooding which affects the flow processes.

A4. Coastal areas are also exposed to salt water intrusion from the sea into the open estuaries and from the ground water (section 4.4).

B The economic significance of coastal areas is demonstrated by the fact that more than 1/3 of the world population is living less than 100 km away from the coastlines and at an altitude less than 200 m above Mean Sea Level (MSL). 700 Million people are living less than 50 km away from coastlines. The largest cities of the world (Tokyo, New-York, London, Shanghai, Calcutta, etc.) are all located close to the sea. The low-lying coastal areas and especially the large deltas of Asia are important areas of agricultural production.

4.2 Astronomical tide and storm surges

4.2.1 The origin of astronomical tides

The sea level is subject to short period variations: the astronomical tides resulting from the attractive forces of the moon and the sun and the storm surges, mostly generated by strong winds.

Astronomical tides are perfectly predictable for any future real time and information can be found in "Tide Tables" for many coastal stations.

Most coastal stations show semi-diurnal tides with two high waters and two low waters during a day of 24 hours and 50 minutes. The two high waters and the two low waters show different heights with respect to MSL, the daily inequality. This is due to the declination (angle between the planes of the moon and the sun) with respect to the equator.

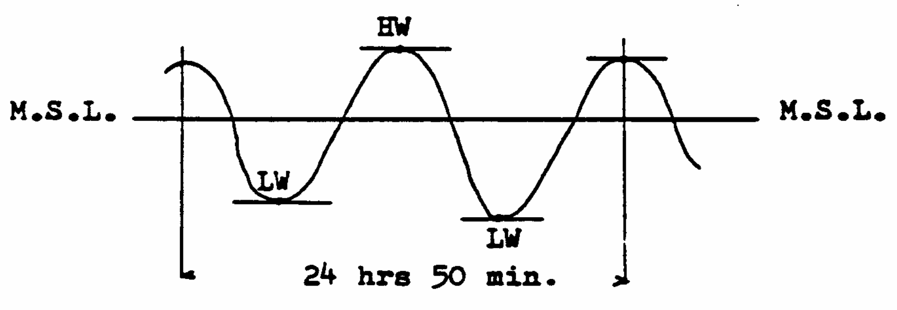


Figure 4.2

Diurnal tides (only one HW and one LW) occur in the Chinese Sea, the seas around Indonesia, the Gulf of Mexico and the Gulf of the St. Lawrence.

In some other places mixed tides occur: during a certain part of the year the tides are semi-diurnal and the other part diurnal.

The tidal range varies according to the position of the sun and the moon with respect to the earth: spring-tides at full moon and new moon and neap-tides during first and last quarter of the moon (1 or 2 days later: the age of the tide).

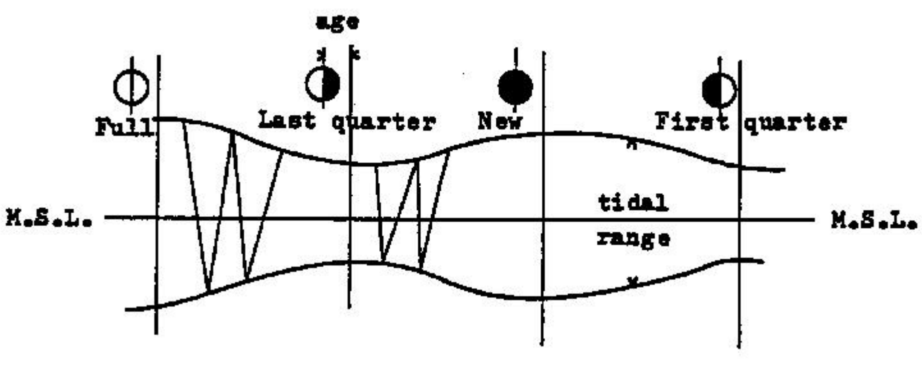


Figure 4.3

4.2.2 Storm surges

The magnitude of the tidal ranges varies considerably from one place to another; ocean tides are generally weak (a few decimetres), by resonance and reflection in seas in connection with the ocean high tidal ranges can occur with 15 m at spring-tide in the Bay of Fundy (Canada) as an extreme case.

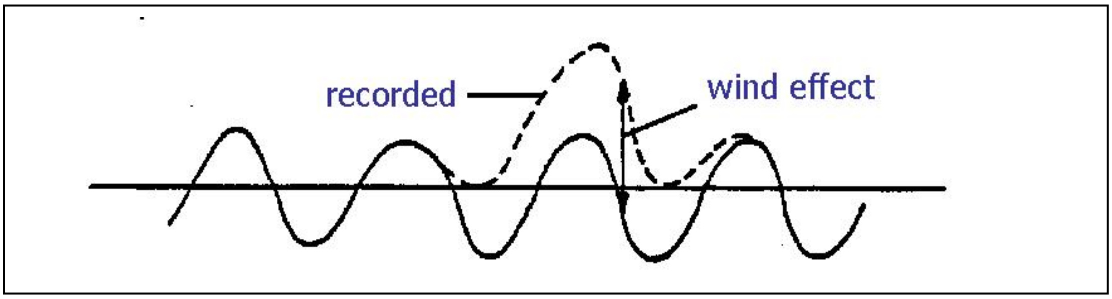


Figure 4.4: Wind effect

In many coastal stations the actually occurring sea levels deviate from the predicted astronomical tides (storm surges). This is mostly due to strong landward and seaward winds. Their effect is super-imposed on the tides. Seas and coasts where these effects are very pronounced (a few metres) are: the North Sea, the east coast of South America, the Bay of Bengal (wind effects of 6 to 9 metres), the west coast of the Pacific Ocean and the Gulf of Mexico. On many other coasts wind effects of a few decimetres may occur.

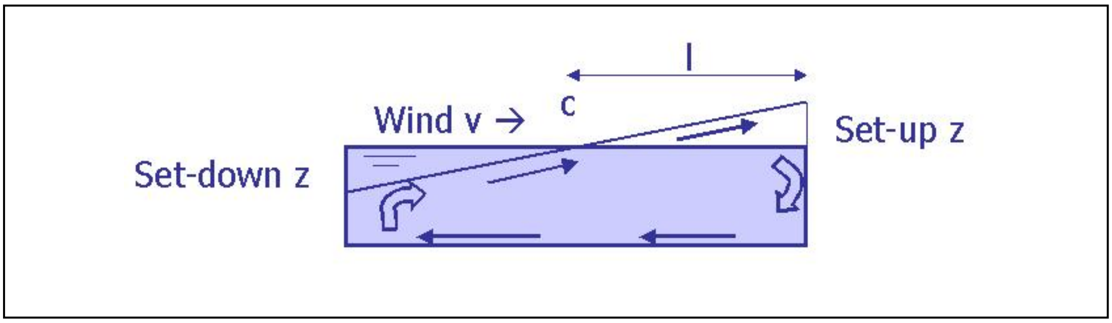


Figure 4.5: Set-up of the water level

Wind effects are generated by the 'drag' of the wind when blowing over a water surface. The friction between the wind and the water surface entrains the water and the water surface is no longer horizontal but assumes a slope.

If: l = distance in metres from the zero point C to the station P on the shore;
v = wind velocity in m/sec at 6 m height;
d = water depth in metres (including the set-up);
z = set-up of the water level

and $z << d$

then

$$z = 3.6 \cdot 10^{-6} \frac{v^2}{gd} \cdot l \qquad \text{Equation 4.1}$$

Under these conditions the neutral line NN (no set-up) has to pass through the centre of gravity *C* of the surface of the lake. The set-up in a station *P'* is *z* (in *P*) times cos α.

If *z* is not small with respect to d the water level is no longer a straight line and the zero point no longer coincides with the centre of gravity.

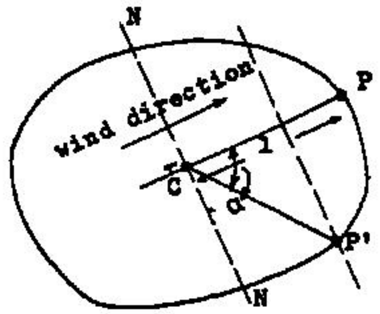


Figure 4.6: Neutral line

The set-up may be considerable: e.g. *v* = 50 m/sec, *l* = 600 kilometres and *d* = 100 m, *z* = 5.4 m. This may cause flood disasters in low-lying coastal areas: 1953 in The Netherlands, 1959 in Japan (Nagoya), and in Bangladesh (in 1970 more than 300 000 people drowned and in 1991 over 130 000 casualties).

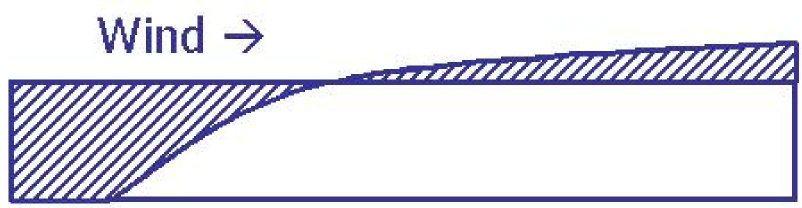


Figure 4.7

The formula for z refers to equilibrium conditions i.e. conditions when the wind has blown a sufficiently long time to displace the water in the basin. This may take say 2 days (North Sea).

Storm surges are generated by depressions moving in the atmosphere. In this respect distinction is made between:

- extratropical depressions of the temperate zones. These depressions cover large areas, move relatively slowly and generally affect the water levels during 1 - 3 days. Maximum hourly wind velocities (50 -100 years return period) are of the order of 25 - 30 m/sec;
- tropical depressions which according to the region are indicated as: (*v*: 40 - 50 m/sec 1 day or less);
  - cyclones (Bay of Bengal);
  - typhoons (Pacific coast of Japan, Taiwan, etc.);
  - hurricanes (Gulf of Mexico).

Abnormally high sea levels can also result from remote effects:

- differences in barometric pressure;
- tsunamis (long wave generated by submarine earthquakes, landslides and volcanic eruptions). These waves travel through the oceans and may hit coastal regions.

In monsoon climates with different wind directions during the different seasons the mean sea levels during each of these seasons may be affected by the prevailing wind during that season and differ a few decimetres from the mean sea level during other seasons.

4.3 Propagation of astronomical tides and storm surges into estuaries

4.3.1 Astronomical tides

Tides and storm surges propagate in open river estuaries and lower river reaches. To understand the hydrology of such area it is necessary to have at least a general understanding of the hydraulic processes involved.

The central problem "tidal hydraulics" (shallow water tidal waves) can be formulated as follows: given the variations of the sea level, the upland discharge and the geometry of the channels, to compute the water levels and discharges in the estuaries. Although considerable progress has been made in recent decades in solving this problem by the use of mathematical models, it is still necessary for the calibration and verification of the models to carry out observations in the field of water levels and discharges. Although it is not necessary for the understanding of the hydrologic conditions of coastal areas to know the methods of tidal hydraulics, it is only with these models that predictions can be made of changes in the flow regime caused by changes in the system (diversions, damming off of branches, deepening, etc.).

If the sea level would be constant the water levels would only be governed by the upland discharge and the flow in the estuary would be permanent in the entire lower reach. It can be noted that in the coastal strip the effect of the upland discharge on the water level is small.

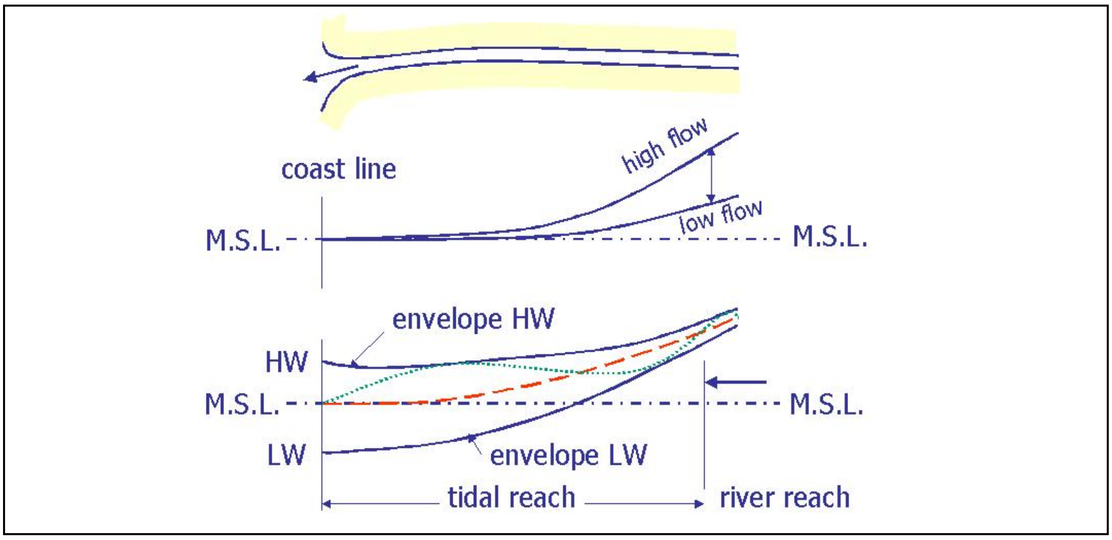


Figure 4.8

The situation is much more complicated in case of tidal variations at the sea boundary. In the tidal reach the water levels in a station also vary between two levels (for a given discharge) and the figure represents the two envelope curves of the high and low water levels in the estuary.

Since the propagation of the tide in the estuary takes time, the momentary situation may be as shown in the figure by the dashed line.

To understand in a qualitative way the intricate pattern of levels and flows in tidal river estuaries it is useful to firstly consider the simple case of a tidal basin (without inflow from the river), which is in open communication via a narrow but deep opening with a sea with tides. The tidal variation at sea propagates into the basin. In the opening an alternating current is generated with alternatively inflow (flood) and outflow (ebb).

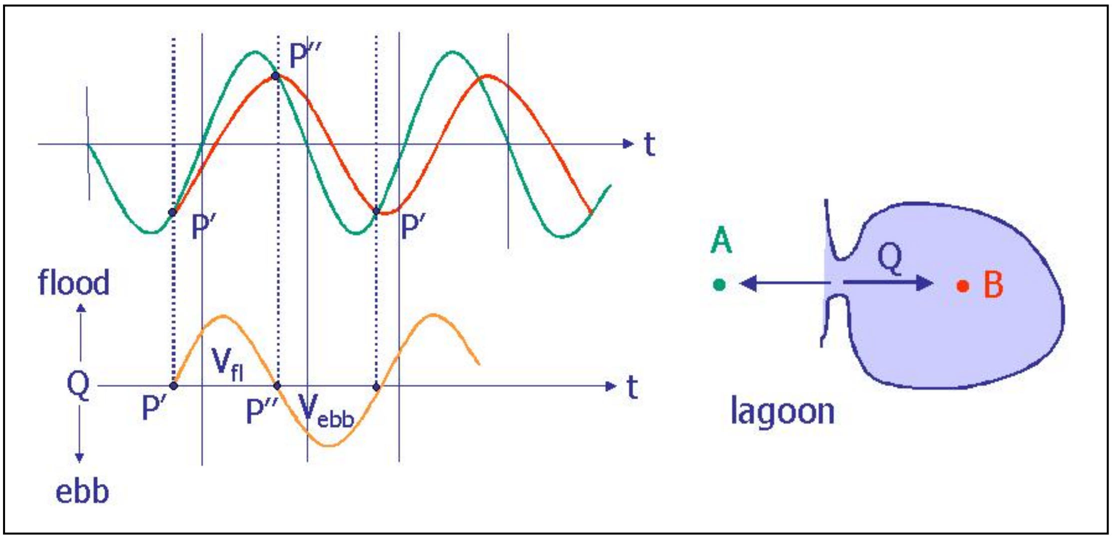


Figure 4.9: Lagoon concept

Because of the time lag in the propagation of the tidal wave, the water levels at A and B are not the same: during the flood the water level in B is lower than the one in A and the reverse is true during ebb. The result is a variation of the water level in B as shown in Figure 4.9 (it is assumed that in the tidal basin the water levels are horizontal; lagoon concept).

The points P' and P" indicate the moments of reversal of the current of slack water;
P' corresponds to low water slack (LWS), and P" corresponds to high water slack (HWS). In this example, where the lagoon concept is applied, HWS occurs at high water (HW) in B. This situation corresponds to a standing wave.

If subsequent tidal ranges are equal (no daily inequality), the two areas indicated by V_{fl} (flood volume) and V_{ebb} (ebb volume) should also be equal.

If a river is debouching into the tidal basin the effect of the inflow Q (supposed to be constant during the tidal cycle) can be superimposed on the flow generated by the tide. The ebb flow is reinforced and the flood flow is pushed back. For reasons of continuity (daily equality):

$$V_{ebb} - V_{fl} = \overline{Q} \cdot T \qquad \textit{Equation 4.2}$$

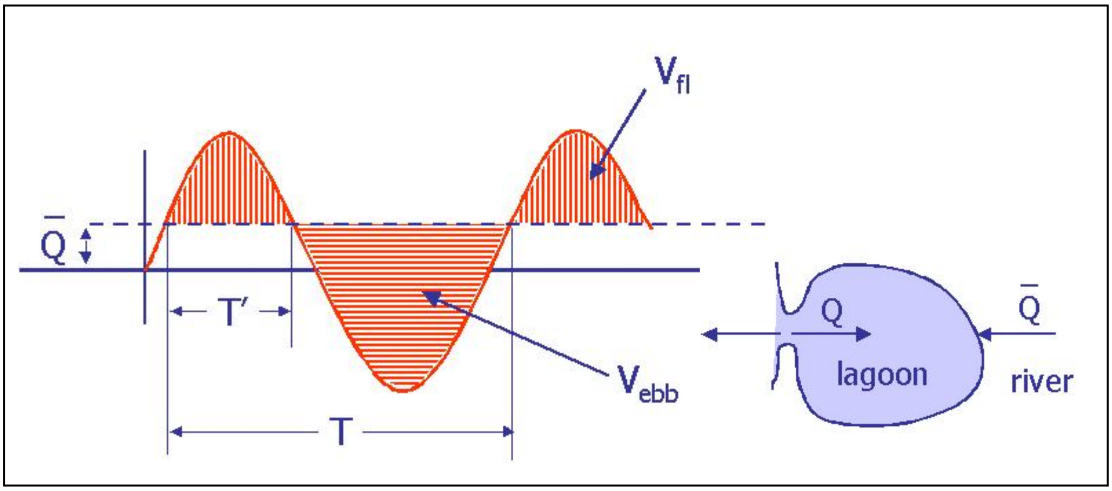


Figure 4.10

An important parameter in salt intrusion models is formed by the flood number or Canter-Cremers number *N*:

$$N = \frac{\text{volume of upland discharge during tidal period}}{\text{volume of the flood}}$$

$$= \frac{QT}{V_{fl}}$$

Equation 4.3

N is dimensionless and indicates the relative importance of the river flow with respect to the tide; if *N* = 0, there is no river flow; if $N \rightarrow \infty$, no tidal influence.

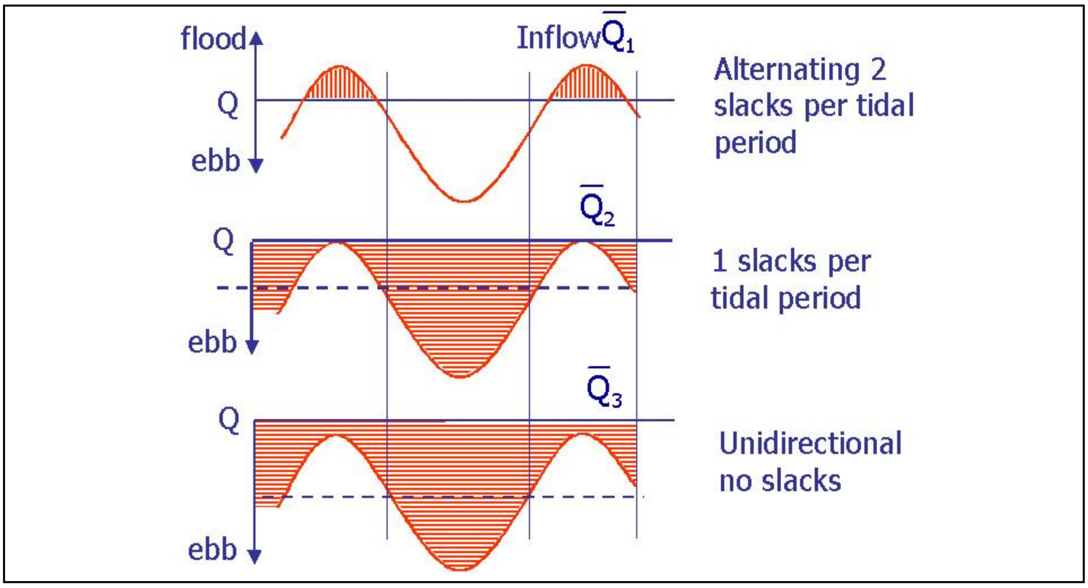


Figure 4.11: Amount of slack per tidal period

Depending on the magnitude of *Q*, the flow in the opening may be alternating or unidirectional as shown below. In the case with Q_2, there is only one slack water during a tidal cycle at which the flow does not change direction.

In an alluvial estuary, the transition between a sea and an alluvial river, the lagoon concept cannot be applied. The estuary is elongated and the time of travel can no longer be neglected. Neither can the friction be ignored.

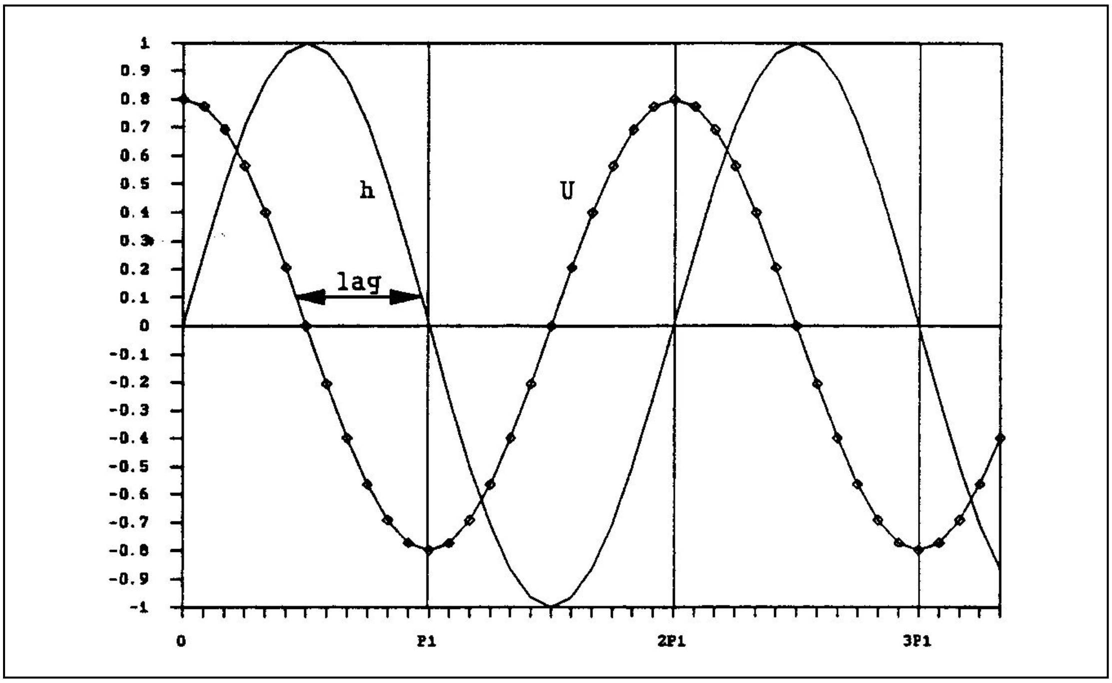


Figure 4.12: Standing wave

Three types of waves may be distinguished:

- a standing wave
- a progressive wave
- a wave of mixed type

Only the latter type of wave occurs in an alluvial estuary, which gradually tapers into an alluvial river. A purely standing wave requires a semi-enclosed body where the tidal wave is fully reflected. Since an alluvial estuary gradually changes into a river, this case does not apply. Standing waves only occur in non-alluvial estuaries or in estuaries where a closing structure has been constructed. In case of a standing wave, extreme water levels are reached simultaneously along the estuary.

Consequently, the "apparent" celerity of propagation c tends to infinity (as extreme water levels occur everywhere at the same time, it seems as if the celerity of propagation is infinitely large). Moreover, HWS coincides with HW (high water) and LWS coincides with LW (low water). The phase lag φ between the fluctuation of water level h and flow velocity U is $\pi/2$ (see Figure 4.12). In Figure 4.12, the positive direction of flow is assumed in upstream direction.

A purely progressive wave only occurs in a frictionless prismatic (constant cross-section) channel of infinite length. Alluvial estuaries do not belong to that category. In case of a progressive, water level and stream velocity are in phase: high water occurs at the same time as maximum flow velocity. The phase lag φ is zero and the wave celerity $c=c_0=\sqrt{(gh)}$ (see Figure 4.13)

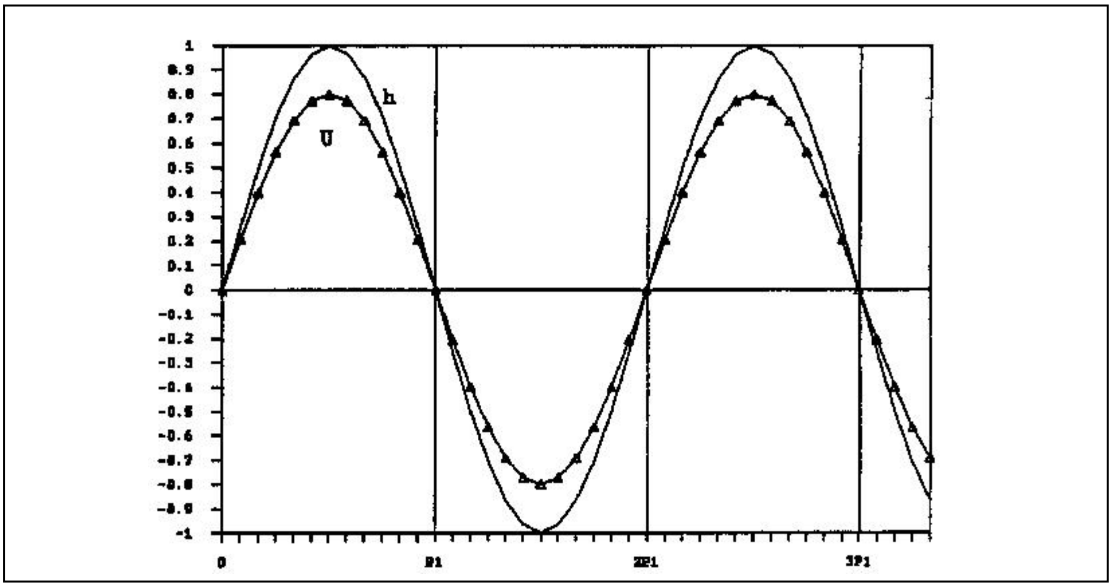


Figure 4.13: Progressive wave

None of these extreme situations occurs in an alluvial estuary. Hence, the tidal wave in an alluvial estuary is of a mixed type with a celerity inferior to c_0 and a phase lag φ which lies between 0 and $\pi/2$.

Another way to determine the phase lag is by looking at the lag time between HW and HWS (or LW and LWS). The resulting phase lag, indicated by ε in Figure 4.14, equals $\pi/2-\varphi$, and is an important parameter indicating the character of an estuary. ε can be easily computed by multiplying the time difference between the occurrence of HW and HWS (or LW and LWS) with $T=2\pi/T$. Like φ, ε varies between 0 and $\pi/2$ (see Figure 4.14).

As a result, in an alluvial estuary, HWS occurs after HW and before mean tidal level; and LWS occurs after LW and before mean tidal level. In an estuary with semi-diurnal tide, HWS occurs approximately 40 to 60 minutes after HW. In case of a diurnal tide the time lag is twice as large.

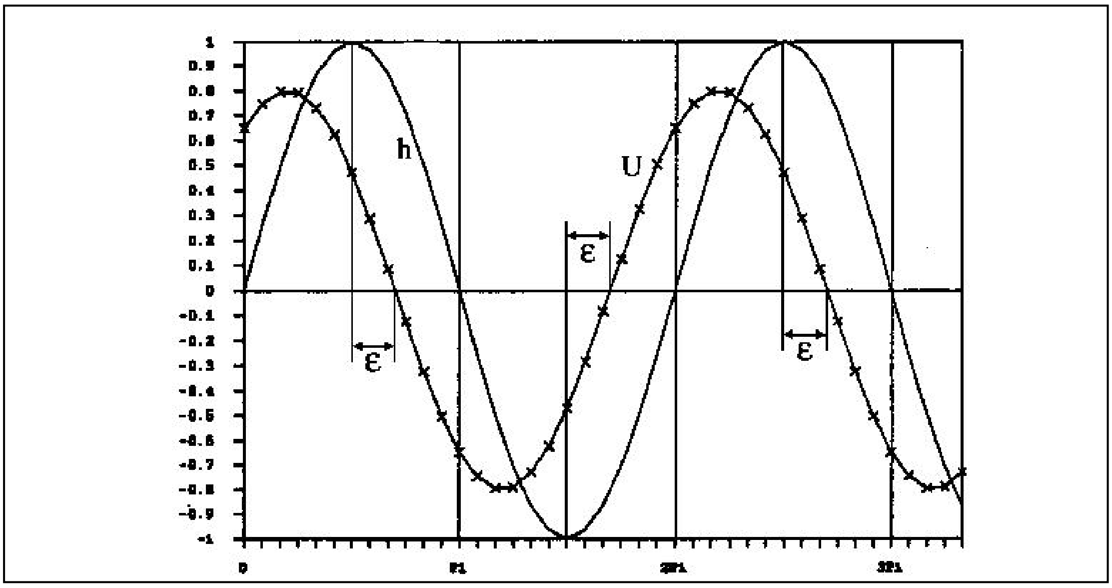


Figure 4.14: Wave of mixed type

Several researchers have approached the phenomenon of tidal wave propagation on the basis of prismatic channels of infinite length (e.g. Ippen, 1966; Van Rijn, 1990). This has led in some cases to incorrect conclusions. Van Rijn (1990) states that bottom friction and river discharge cause the phase lag between horizontal movement (current velocities) and vertical movement (water levels).

The effect of the river discharge is not a phase lag, but a vertical shift of the velocity-time graph, which causes HWS to occur earlier and LWS to occur later. The second cause mentioned, the friction, only has a minor effect on the phase lag. Computer simulation of an estuary with a constant depth of 10 m, a slight funnel shape (width reduction of 18% over 40 km), a tidal range of 3 m, and a Chezy coefficient of 60 $m^{0.5}/s$ showed that the velocity amplitude doubled if the Chezy coefficient was increased to an extremely high value of 1000 $m^{0.5}/s$ (frictionless flow) but that the phase lag φ only decreased from 0.3π to 0.22π and not to zero. A more important cause for phase lag is the shape of the estuary, which, depending on the convergence of the banks and the bottom causes partial reflection of the tidal wave and consequently a tidal wave of the mixed type.

Figure 4.15 shows a longitudinal profile of the water levels in an estuary at three different times t_1, t_2 and t_3, to illustrate the progressive character of the tidal wave. The mean level is slightly above MSL because the friction is higher during the ebb than during the flood.

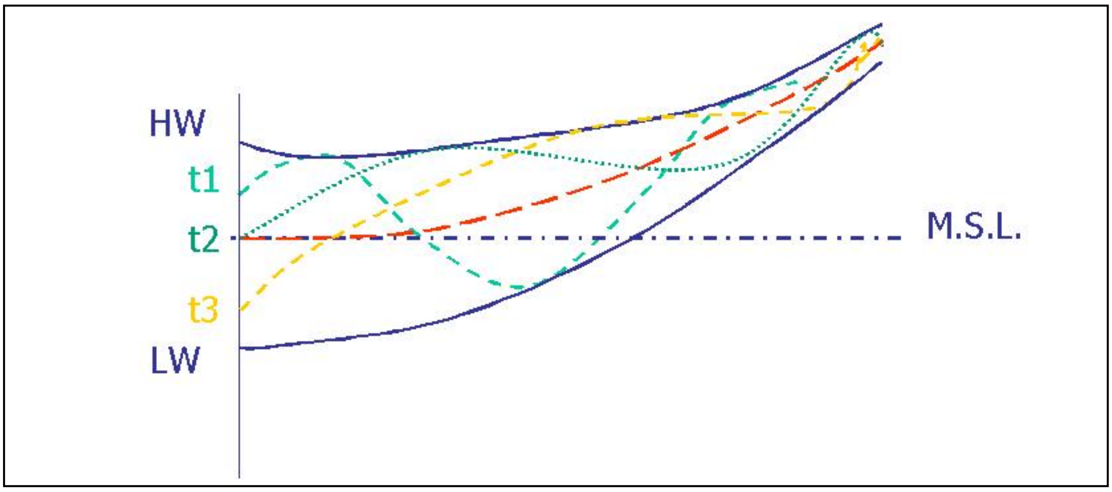


Figure 4.15

Figure 4.16 shows that the longitudinal profile of water levels can be divided into three reaches. The dotted lines represent the levels in the estuaries where current reversals occur. It is observed that the volume of water between the loci (envelopes) of slack water is equal to the flood volume V_{fl} upstream of any cross-section. Thus two methods exist for the determination of the flood volume ε:

- from integration of the discharge over the time between LWS and HWS
- from integration of the water levels between HWS and LWS over the area

Integration of the discharge between LWS and HWS yields:

$$\int_{LWS}^{HWS} Qdt = EA$$ Equation 4.4

Integration of *H'* over the surface area yields:

$$\int_{x}^{\infty} H'Bdx = \frac{HBb\cos\varepsilon}{1-b/\delta}$$ Equation 4.5

where *H'* is the tidal range between HWS and LWS, *E* is the tidal excursion of a water particle, *A* is the cross-sectional area, *B* is the width, *b* is the convergence length of the width ($B=B_0 \exp(-x/b)$), δ is the damping length of the tidal velocity amplitude ($v=v_0 \exp(-x/\delta)$) and ε is the phase lag between HW and HWS (equal to $\pi/2-\varphi$). Equating these two volumes leads to the following simple expression (Savenije, 1992, 1998):

$$Eh = \frac{Hb\cos\varepsilon}{1-b/\delta}$$ Equation 4.6

and subsequently:

$$\frac{H}{E} = \frac{h_0}{b}\frac{(1-b/\delta)}{\cos\varepsilon}$$ Equation 4.7

It should be noted that - for a constant *Q* - the value of ε differs according to the location in the estuary: ε is minimum in the mouth and infinite in P.

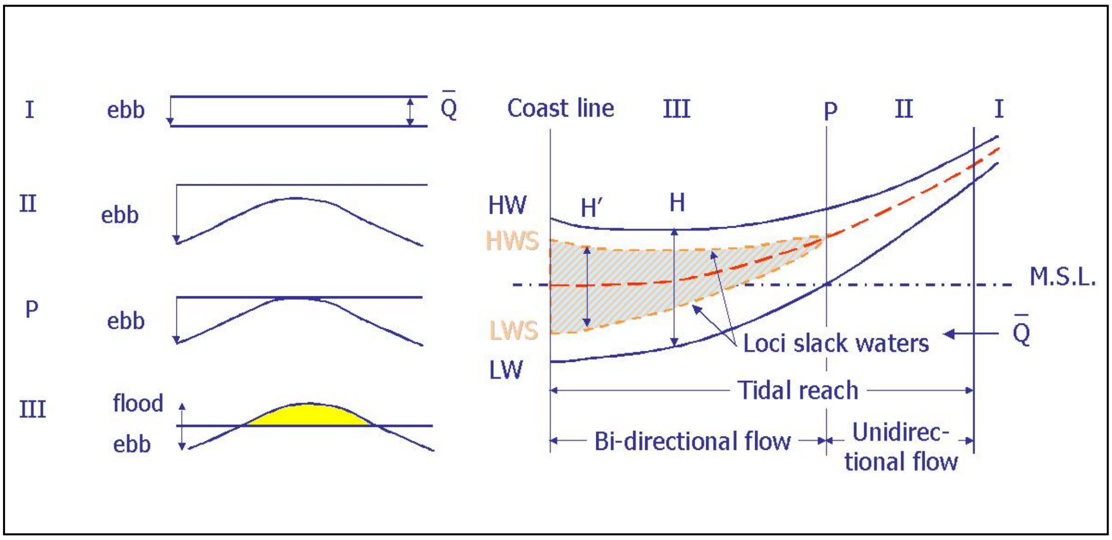


Figure 4.16

The distance over which the tides penetrate into an estuary (theoretically infinite) depends on the tidal range, the upland discharge and the channel characteristics (convergence length, roughness and especially the gradient). At low flows of the Mekong (2000 m^3/sec) the tides from the South China Sea penetrate as far as Phnom-Penh, 300 km from the mouth (gradient 1 to 3 cm per km). In the Gambia estuary the tides even propagate over a distance of 500 km, which is a distance in the same order of magnitude as the wavelength.

With increasing upland discharge the length of the tidal reach decreases: the river flow pushes the tides back.

Because of the effect of the tides on the discharges it is difficult to determine the upland discharge *Q* from measurements in the reach where the flow is alternating. In this reach *Q* has to be computed from the difference between flow and ebb volumes which both may show errors in the order of 5 to 10%. After subtraction of the ebb and flood volumes the relative error may well turn out to be more than 100%.

4.3.2 Storm surges

Storm surge levels also propagate into estuaries. Storm surges are non-periodic long wave of rather short duration especially when generated by tropical depressions. In non-embanked deltas they cause extensive flooding. Since the filling of the estuaries and especially of the land areas takes time and requires gradients, the highest water levels inside may remain much lower than the maximum water level at sea (case α). Only for storm surges of long duration (several days) may the areas be filled up with water to the same extreme level as at sea (case β).

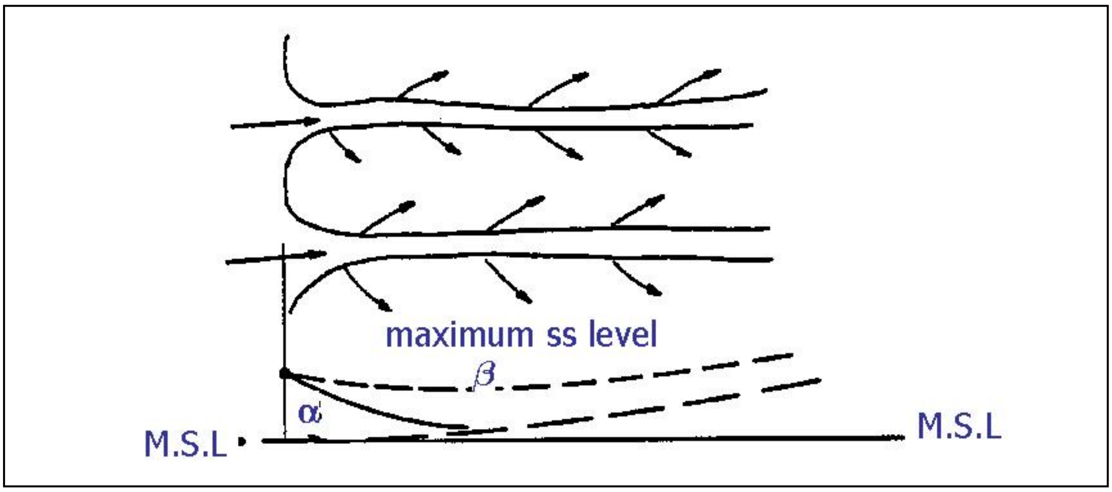


Figure 4.17

From this follows that embanking in the coastal strip brings about a rise of the water levels in the estuaries in case of storm surges of short duration, since in that case the land areas can no longer store water.

4.4 Salt water intrusion

4.4.1 Introduction

Deltaic and other low-lying coastal areas are exposed to the intrusion of salt both in surface water and groundwater. The result may be that the water in the rivers and in the canals cannot be used for consumption. Saline water can originate from various sources:

1. Salt water intrusion from the sea into open estuaries.
2. Seepage of brackish or saline ground water originating from old marine deposits.
3. Seawater entering through navigation locks between the sea and a fresh water canal.
4. Leakage of saline water through sluices and locks.
5. The salt load of a river (natural salinity or brackish effluents from agricultural drainage and industrial wastes).
6. The salt contained in rainwater and the saline spray in coastal regions (usually a minor amount).
7. Saline groundwater coming to the surface because of irrigation without proper drainage (e.g. Iraqi), or clearing of forest (e.g. Australia)

The first source of salt is the most general source in coastal areas.

4.4.2 Sea water intrusion into open estuaries

General

The penetration of seawater (relative density around 1.028) into rivers with a fresh water flow (relative density 1.0) is due to the 2.8% difference in density. If there are no tides and the estuary cross-sections are regular the intrusion assumes the shape of a sort of wedge (or tongue) with stagnant seawater. The fresh river water flows out on top of the seawater (see Figure 4.18).

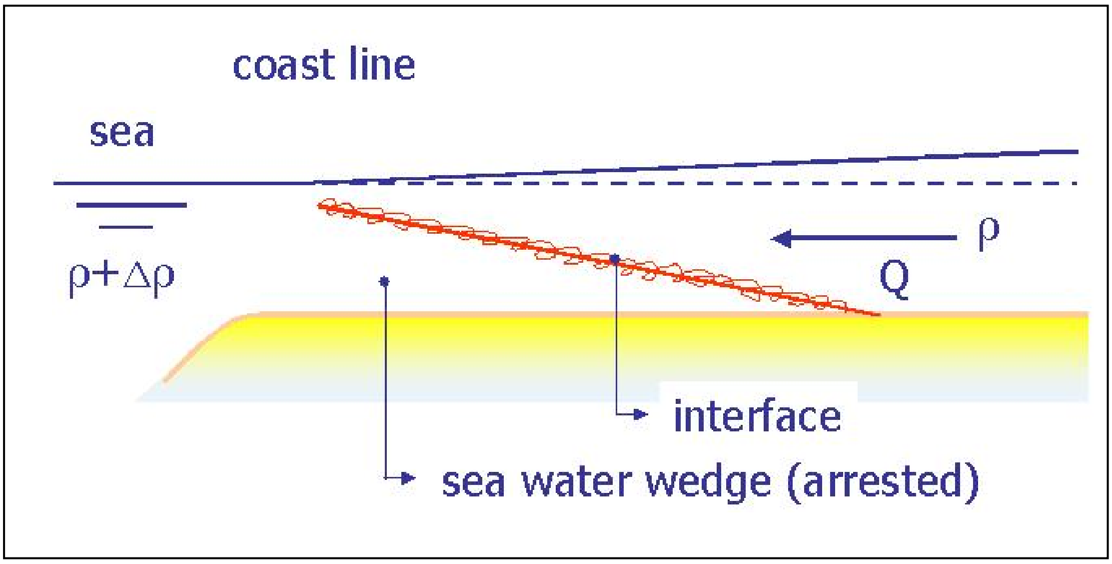


Figure 4.18

Saline wedges, however are very uncommon. They only occur in narrow estuaries, often man-made or non-alluvial estuaries, with a high river discharge and a small tidal range. Tides promote the turbulent mixing at the interface resulting into an estuary where incomplete or (almost) complete mixing of the waters near the bottom and near the surface occurs. Mixed estuaries are most common, particularly during periods of low river flow.

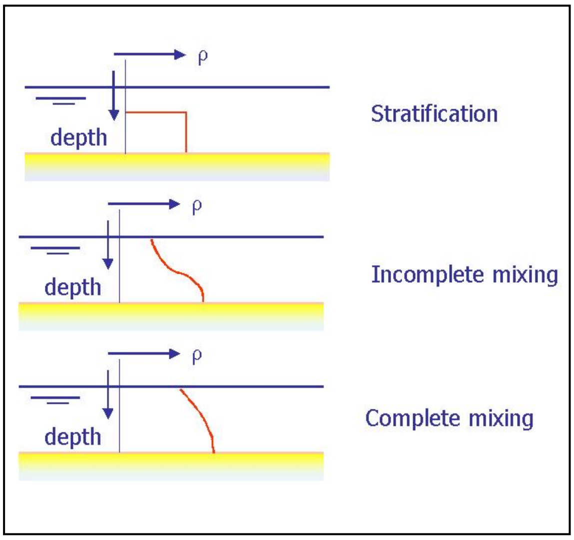


Figure 4.19

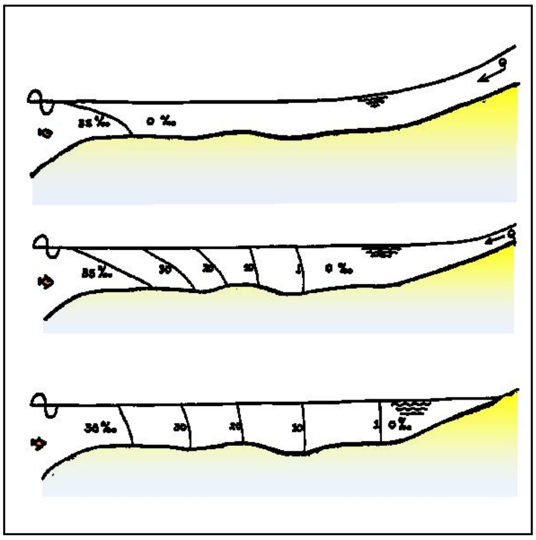


Figure 4.20: Mixing degree

The degree of mixing largely depends on the flood number *N*, also called the Canter-Cremers number:

$$N = \frac{QT}{V_{fl}} \qquad \textit{Equation 4.8}$$

Roughly it can be assumed that if:

$N \geq 1.0$: the estuary is stratified
$0.1 < N < 1.0$: the estuary is partly mixed
$0 < N < 0.1$: the estuary is well mixed

It can also be seen from Figure 4.19 and Figure 4.20 that there is no significant bottom slope in the part of the estuary where the tide dominates over the river discharge. Observations in estuaries have shown, however, that there is a slight increase of the water level in the upstream direction over the length of the salt intrusion.

Balance of forces

This phenomenon can be explained by a balance of hydraulic forces over the intrusion length of the salinity. This analysis is applicable to both the mixed and stratified situation (see Figure 4.21 and Figure 4.22)

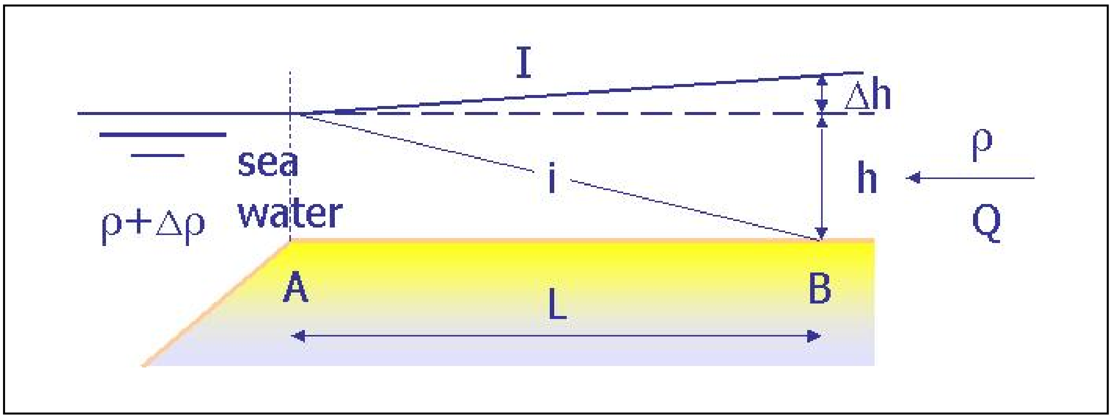


Figure 4.21 The balance of hydrostatic pressure over a reach of salt water intrusion (mixed or stratified)

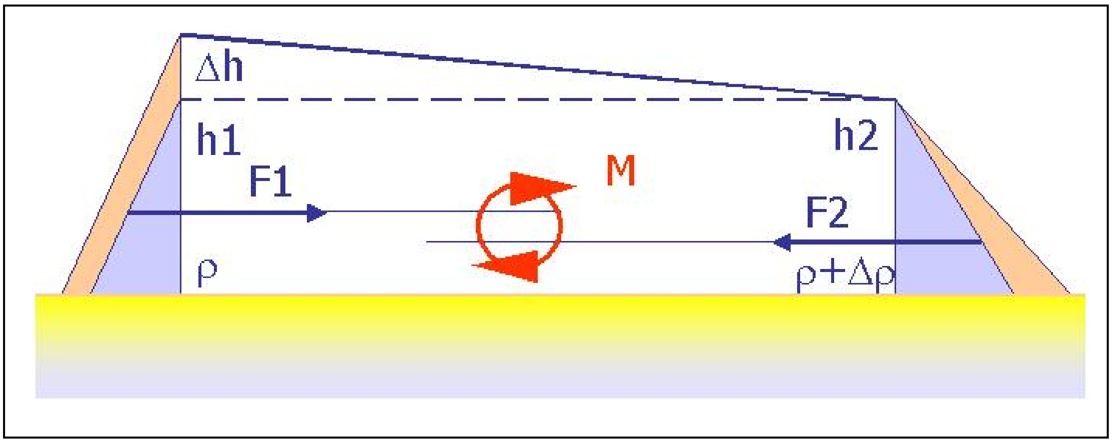


Figure 4.22 After the hydrostatic forces make horizontal equilibrium, a moment of forces remains driving vertical circulation and mixing

Since the sea water is not moving, and the flow lines in point B me be assumed to be parallel, the forces resulting from the hydrostatic pressures in A and B should be equal, or:

$$\frac{1}{2}gh^2(\rho+\Delta\rho)=\frac{1}{2}g(h+\Delta h)^2\rho \qquad \textit{Equation 4.9}$$

After disregarding higher orders of Δh, this yields:

$$\Delta h = \frac{\Delta\rho}{2\rho}\text{h} \quad \text{and} \quad \text{I} = \frac{\Delta h}{L} = \frac{\Delta\rho}{2\rho}\frac{h}{L} = \frac{\Delta\rho}{2\rho}i \qquad \textit{Equation 4.10}$$

Although the two forces cancel out, there is a residual moment M that results from the fact that the working lines of the forces don't agree (see Figure 4.22). The arm of the moment is $\Delta h/3$. Hence, the moment equals:

$$M = \frac{1}{2}gh^2\rho\frac{\Delta\rho}{2\rho}\frac{h}{3} = \frac{1}{12}g\Delta\rho h^3 \qquad \textit{Equation 4.11}$$

In a stratified estuary, this moment is counteracted by a shear stress in the interface, which keeps the interface in place. In a mixed estuary, this moment is counteracted by friction. The moment drives the vertical circulation and mixing process.

Stratified estuary

Thijsse (1952) developed a formula for the intrusion length for stratified flow based on the assumption that the flow of fresh water over the salt-fresh interface can be described by an adjusted Chézy equation. The flow of the fresh water is then governed by:

$$Q = C'A'\sqrt{h'I} \qquad \textit{Equation 4.12}$$

In this formula C' is referring to the conditions at the interface and A' and h' vary in the flow conduit. All three are virtually unknown and Thijsse replaced A' and h' by the known parameters A and h (full cross-section) and put the uncertainties in the value of C' to be determined from actual field records (canals in the Netherlands). Under these conditions a value of C' is 21.2 $m^{½}$/sec showed acceptable agreement with the observations and the formula reads:

$$Q = 21.2A\sqrt{hi\frac{\Delta\rho}{2\rho}} \quad \text{and since} \quad \text{i} = \frac{h}{L} \qquad \textit{Equation 4.13}$$

$$L = \frac{225A^2h^2}{Q^2}\frac{\Delta\rho}{\rho} = \frac{225B^2h^4}{Q^2}\frac{\Delta\rho}{\rho} = \frac{225h^2}{v^2}\frac{\Delta\rho}{\rho} \qquad \textit{Equation 4.14}$$

for a rectangular cross-section with width B and depth h.

The formula should be used with suspicion and not without due checking but it does show the great influence of the depth h (effect of dredging for estuarine harbours) and the freshening effect of the upland discharge Q.

Most estuaries only have stratified flow during extremely high flows. Estuaries normally have mixed salt intrusion. For these estuaries Savenije (1993) introduced in an analytical formula for the total salt intrusion length at HWS (L) on the basis of a number of dimensionless ratios. This theory is presented in the following paragraphs.

Mixed estuary

In a mixed (or partially mixed) estuary the salinity in a given station varies not only with the upland discharge but also with the tide.

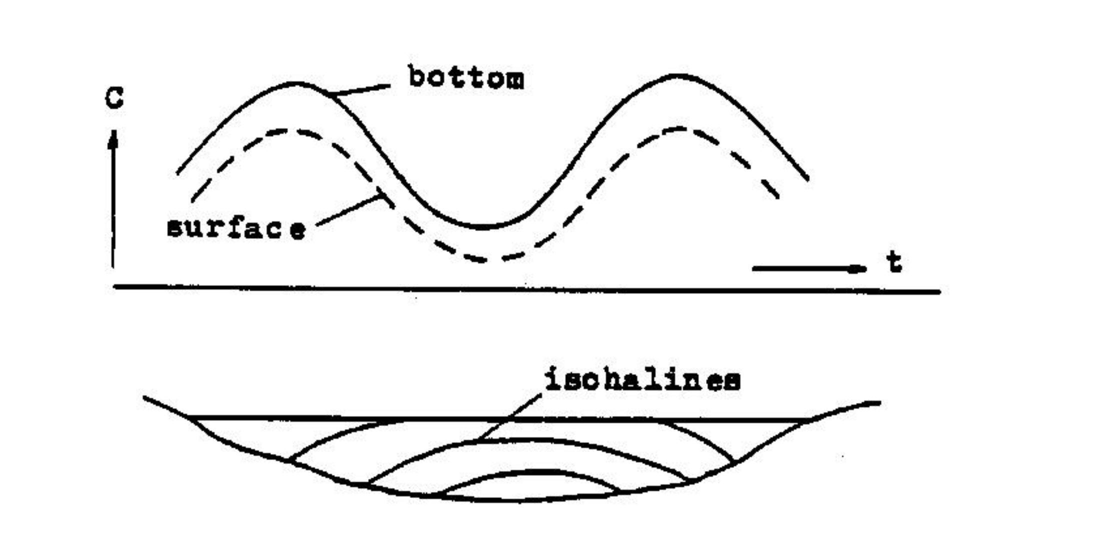


Figure 4.23

The concentrations in a certain station and at a certain moment can vary over the cross-section so that a sample taken from the bank does not always show the average salinity. During measurements one should be aware of this, so as to make certain that a representative sample is taken.

Salinity can be determined in a simple way by measuring the electrical conductivity (calibration with a limited number of samples with chemical analysis). Probes can be attached to a cable so that the vertical distribution can quickly be determined.

The average salinity in a cross-section *S* is a function of time *t* and space *x*. When plotted against the longitudinal *x*-axis, three characteristic situations can be distinguished: HWS, LWS and TA (tidal average). At HWS, when the direction of flow changes from upstream into downstream. In case of a progressive or mixed-type wave the point in time at which HWS occurs — some time after reaching high water (HW) — increases as the tidal wave moves upstream. Hence at each point along the estuary HWS occurs at a different time.

The best way to carry out a reconnaissance salt intrusion measurement is by moving boat at HWS; this is for the following reasons:

1. The moment that HWS occurs is easily determined. The observer measures the salinity, when the in-going current slacks. Although the same advantage applies to LWS, the accessibility at LWS is generally poor. Sometimes, especially in natural estuaries, it is very difficult to reach the waterside. Inaccessible mud flats often separate the river from the banks.

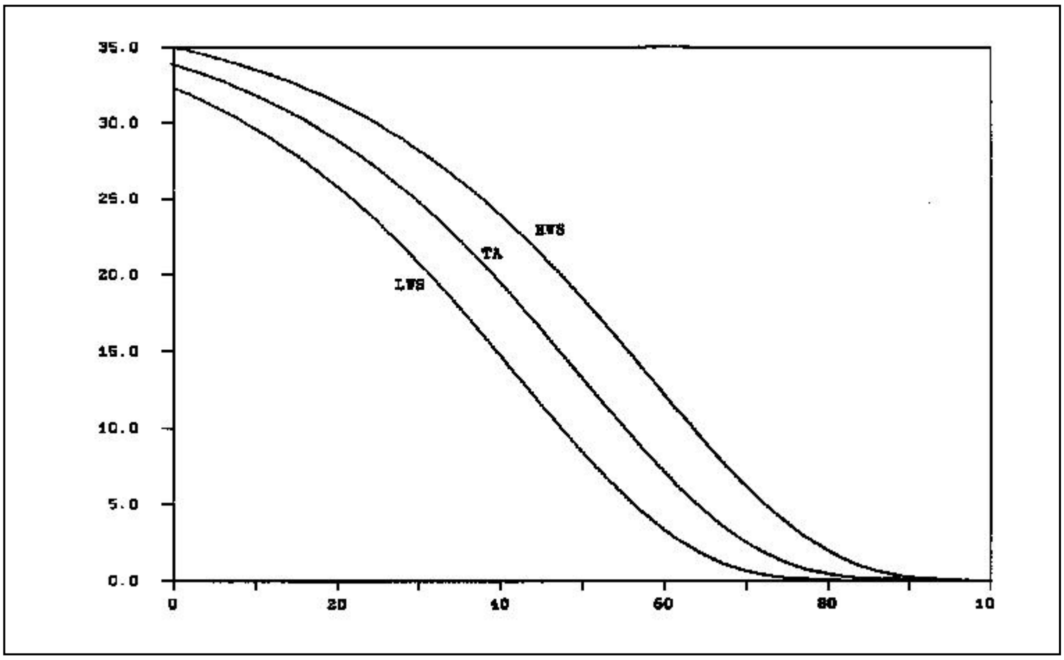


Figure 4.24: Envelope curves of salinity intrusion at High Water Slack (HWS), Low Water Slack (LWS) and mean tide (TA)

2. If the salinity at the downstream boundary is not known, it can easiest be estimated at HWS. At HWS, the salinity at the estuary mouth is generally equal or almost equal to the ocean salinity, which is not, or not much, affected by the fresh water discharge from the estuary.

3. At HWS the salt intrusion is at its maximum. Generally, it is the maximum intrusion that is of interest to planners.

4. A single observer in a small outboard driven boat can travel with the tidal wave and measure the entire salt intrusion curve at HWS.

5. In a moving boat it is easy to measure a full vertical in the center of the stream which is the best location to measure the salinity; if necessary a check in another vertical can be made to see if there is much lateral variation.

6. If the intrusion length is not too long, the observe can (and should) return to the estuary mouth and repeat the measurement for LWS.

Another method, which is often recommended in handbooks but which is not advisable, is to station one or two permanent observes at strategic points along the estuary to record continuous variations of the salinity. The information obtained by this method is not very valuable because from the shore it is difficult to determine the salinity at midstream and to measure the variation over the depth. In addition, it is impossible to install too many observers along the estuary so as to obtain a sufficient amount to draw accurate longitudinal profiles. Moreover, the most important information to be collected is the range over which the salinity varies; the moving boat method just measures this range and hence is much more efficient.

Analysis

In most alluvial estuaries (except exceptionally large estuaries such as the Gambia, or estuaries where there is no fresh water inflow during the dry season) a steady state model can be used to describe the salt intrusion.

For steady state, there should be an equilibrium in any cross-section between:

- the advective transport of salt by the river flow in downstream direction $Q(S-S_f)$, where S_f is the salinity of the river water;
- and the dispersive transport of salt in upstream direction under the effect of mixing which is proportional to the concentration gradient dS/dx.

The dispersive transport is generally assumed to be equal to

$$DA\frac{dS}{dx} \qquad \textit{Equation 4.15}$$

where: A = cross-sectional area
D = dispersion coefficient

Hence:

$$(S-S_f)Q_f = -AD\frac{dS}{dx} \qquad \textit{Equation 4.16}$$

If we assume, for the purpose of illustration, that *Q/DA* is constant, then

$$S-S_f = (S_0-S_f)\exp\left(-\frac{Q}{DA}x\right) \qquad \textit{Equation 4.17}$$

where S_0 is the sea salinity at x=0 (the estuary mouth).

In this simple case, the salinity plots as straight lines on semi-logarithmic paper.

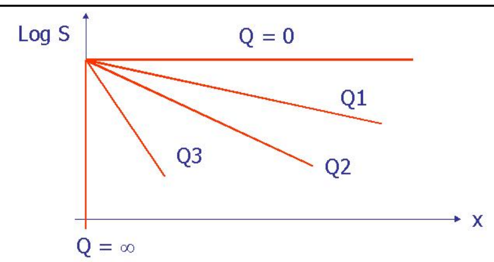


Figure 4.25

The following considerations make that the solution is not that simple:

1. The cross-sectional area A is not constant with x;
2. The value of D cannot be easily predicted; it differs from one estuary to another; moreover it varies with x;
3. Actual records show that the lines deviate from straight lines;
4. The concentration for x = 0 is not constant but depends on Q;

The objective is to have a formula derived from the available and usually small amount of data and with as much physical foundation as possible which would enable to predict for a wide range:

- the effect of upland discharge;
- the salinity at any station along the estuary;
- the effect of deepening or shoaling of the channels.

The geometric variation of the cross-sectional area can generally be solved by fitting an exponential function. Savenije (1986) showed that very good results are obtained by considering an "ideal estuary" with a horizontal bed (the depth h_0 is constant), exponentially varying width (easy to measure from a map), and hence an exponentially varying cross-section:

$$A(x) = A_0 \exp\left(-\frac{x}{a}\right) \qquad \textit{Equation 4.18}$$

where A_0 is the cross-sectional area at the estuary mouth and a is a convergence length to be obtained from fitting measured cross-sections to a line on semi-logarithmic paper.

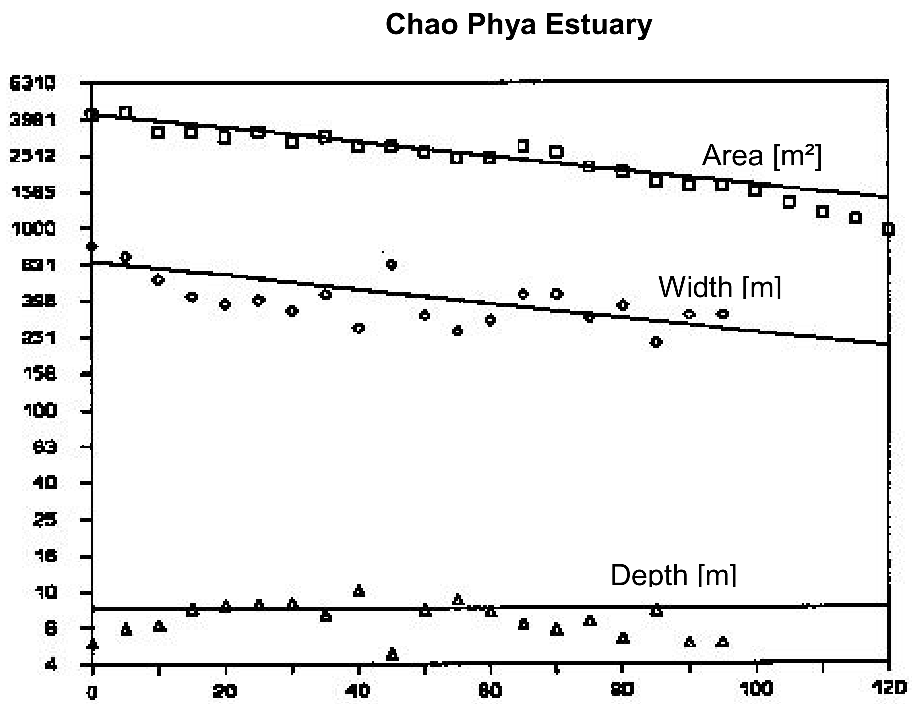


Figure 4.26

For the longitudinal variation of the dispersion coefficient different researchers have followed different approaches. Of the different types of relations tried in the literature, Prandle (1981) gave a good overview. He tried out the following relations:

$$D = D_0$$

$$D \propto \frac{\partial S}{\partial x} \qquad \textit{Equation 4.19}$$

$$D \propto \left(\frac{\partial S}{\partial x}\right)^2$$

which can be summarized as:

$$D \propto \left(\frac{\partial S}{\partial x}\right)^k \qquad \textit{Equation 4.20}$$

with k = 0 ,1, 2 respectively.

These relations appear to work only in very specific conditions. For instance, the first equation performs better in estuaries with a pronounced funnel shape, whereas the second and third equations perform better in narrow and prolonged estuaries.

However, Savenije (1992) showed that the following equation works well under a wide range of estuaries, of different depths, different convergence lengths and different widths.

$$\frac{D}{D_0} = \left(\frac{S}{S_0}\right)^K \qquad \textit{Equation 4.21}$$

This approach corresponds with the method of Van der Burgh (1972), who based on a large number of measurements in Dutch estuaries found that:

$$\frac{\partial D}{\partial x} = -K\frac{Q_f}{A} \qquad \textit{Equation 4.22}$$

where K is Van der Burgh's coefficient.

Combination of Equations 4.16, 4.18 and 4.22 yields:

$$\frac{S - S_f}{S_0 - S_f} = \left(\frac{D}{D_0}\right)^{\frac{1}{K}} \qquad \textit{Equation 4.23}$$

$$\frac{D}{D_0} = 1 - \beta\left(\exp\left(\frac{x}{a}\right) - 1\right) \qquad \textit{Equation 4.24}$$

Moreover, substitution that $x = L$ where $S = S_f$ yields:

$$\beta = \frac{Ka}{\alpha_0 A_0} \qquad \textit{Equation 4.25}$$

$$L = a \cdot \ln\left(\frac{1}{\beta} + 1\right) \qquad \textit{Equation 4.26}$$

where L is the intrusion length at HWS, and α_0 is a calibration coefficient equal to D_0/Q_f, where D_0 is the dispersion coefficient at the estuary mouth at HWS.

In addition empirical formulae have been derived by Savenije (1992) to predict the values of the calibration coefficients K and α_0:

$$K = 6.3 \cdot 10^{-6} \left(\frac{h_0}{a}\right)^{1.04} \left(\frac{E}{H}\right)^{2.36} \qquad \text{Equation 4.27}$$

$$\alpha_0 = 220 \frac{h_0}{a} \sqrt{\frac{ETgh_0}{-Q_f A_0}} \qquad \text{Equation 4.28}$$

Combination of these equations yields an expression for the intrusion length *L* at HWS. This relation is a combination of dimensionless ratios. These ratios are:

1. The densimetric Froude number F_d :

   $$\mathrm{F_d} = \frac{\rho}{\Delta\rho} \frac{\upsilon^2}{gh} \qquad \text{Equation 4.29}$$

   where υ is the tidal velocity amplitude. The ratio of $\Delta\rho$ to ρ is related to the residual moment of the hydrostatic forces, described in Figure 4.21, which relates to stratified flow. In mixed estuaries, the balance of forces is essentially the same and the resulting moment is the driving power behind the salt intrusion through mixing.
2. The Canter-Cremers number relating fresh water discharge to the tidal volume:

   $$N = \frac{QT}{V_{fl}} = \frac{QT}{EA_0} \qquad \text{Equation 4.30}$$

3. The Estuarine Richardson number, which is generally used as a better indicator than *N* to determine the degree of stratification:

   $$N_R = \frac{N}{F_d} \qquad \text{Equation 4.31}$$

4. A simpler form of the Canter Cremers number being the ratio of the tidal velocity amplitude υ to the fresh water velocity v_f
5. The ratio of the depth *h* to the convergence length of the cross-sectional area *a*
6. The ratio of *E* to *a*
7. The coefficient of Van der Burgh *K*

The resulting equation is:

$$L_{HWS} = a \ln\left(1 + \frac{220}{K} \frac{h}{a} \frac{E}{a} \frac{\upsilon}{v_f} \sqrt{40 N_R}\right) \qquad \text{Equation 4.32}$$

This equation is the most accurate equation available to date (see Savenije 1993) and is applicable in all estuaries, provided they are alluvial.

Preventive measures
The saline intrusion can be halted or reduced by:

a) Increasing the upland discharge (releases from reservoir). This requires relatively high amounts of fresh water, which cannot be used for other purposes like irrigation.

b) Decreasing the depth of the estuary. This is an expensive operation, which may only be suitable in special cases (Rotterdam Waterway).

c) In case of small tides and a stratified estuary a low submerged sill effectively halts the saline wedge without offering a significant obstacle to flood flows.

d) Damming off estuaries by building an enclosing dam equipped with sluices and gates to remove excess water to the sea (estuarine reservoirs, see section 4.5.2).

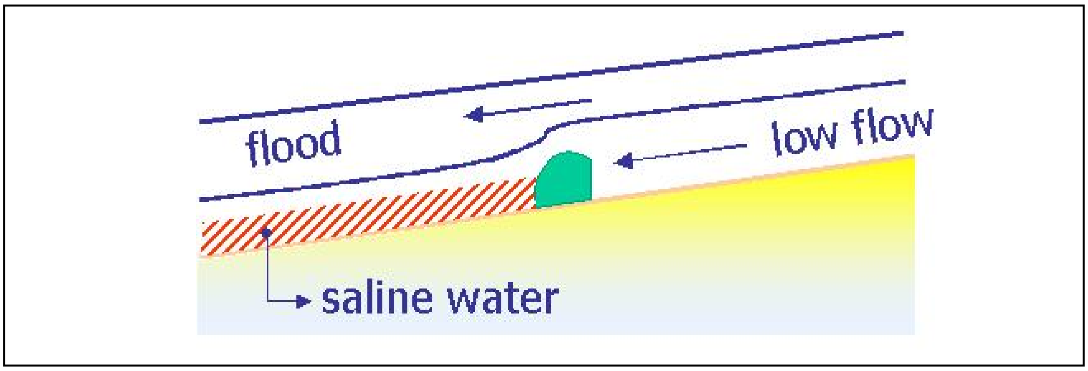


Figure 4.27: damming off estuaries by building an enclosing dam.

If preventive measures are not feasible other measures as well as repressive measures can be considered such as:

a) Shifting of intakes of fresh water (for water supply or irrigation) to points upstream of the saline reach.

b) Rinsing of flushing of canals exposed to saline intrusion with fresh water.

c) Over-irrigation in combination with adequate drainage to leach the soils.

4.4.3 Seepage of brackish ground water

The ground water under coastal areas is often saline or brackish especially under the coastal portions. The saline water originates from the marine transgressions at the time of deposition (primary aquifer salinization). If differences in the elevation of the surface waters are created (by reclamation and empoldering) a seepage flow is generated which carries the saline ground water to the surface ('oozing') thus forming a source of salt. In principle the rate of supply of salt is the product of the seepage rate and the concentration of the ground water. However the seepage flow changes the distribution of the salinity. In simple flow models (an aquifer covered by a semi-pervious top layer) the flow trajectories (or paths), the travel times and the isochrones can be computed and hence the changes in the salinity distribution. In this respect, reference is made to the lectures on ground water.

Fresh water aquifers under coastal areas may become saline because of intrusion of seawater. This may be caused by overdraft on fresh water pockets, reclamation of deep low-lying areas close to the sea and intrusion into aquifers of river water from the saline reach (secondary aquifer salinization).

Seepage of brackish ground water also contributes to soil salinization. The most important cause of soil salinization, especially in arid zone coastal areas), is the use of irrigation water in the absence of any drainage (primary soil salinization). The explanation lies in the fact that irrigation always contains some salt and that evaporation does not remove any salt. In this way a progressive increase of the salinity of the pore water occurs.

Shallow ground water (within say 2 metres below the surface) also acts as a source of salt (secondary soil salinization). The pore water moves upward by capillary forces and in the top layer the water goes into the atmosphere and salt remains in the top layer. Leaching with fresh water gives a temporary relief but produces a rise of the ground-water level, thus increasing the capillary ascend. The only remedy is to apply drainage in combination with leaching and removal of the saline leachate. By drainage (trenches or tile drains) the phreatic table is maintained at 1.2 to 1.5 m (even lower in the U.S.A. and U.S.S.R.) below the surface and a leaching of around 10% of the consumptive use is applied.

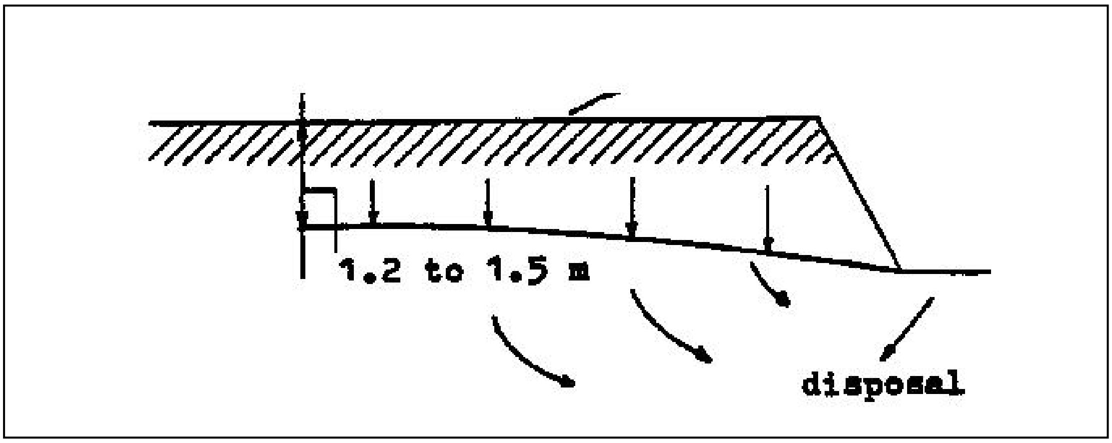


Figure 4.28

4.4.4 Sea water entering at navigation locks

Exchange processes

Distinction should be made between the amount of salt admitted by filling the lock chamber when the level of the sea is higher than the level of the fresh water canal and the amount of salt resulting from the exchange of water between the sea and the chamber filled with fresh water when opening the outer gates.

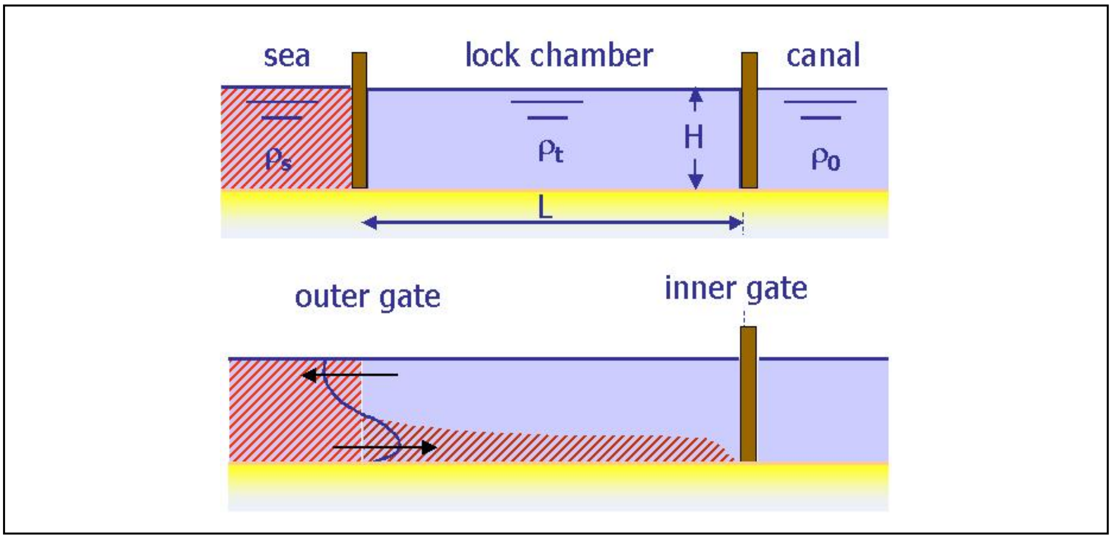


Figure 4.29

ρ_s = relative density of sea water
ρ_0 = relative density of fresh water
ρ_t = average relative density of water in the chamber at a time t after opening of the outer gates
H = depth of the chamber
L = length of the chamber

then the exchange at the time t can be characterized by the dimensionless factor:

$$u_t = \frac{\rho_t - \rho_0}{\rho_s - \rho_0} \qquad \textit{Equation 4.33}$$

if $\rho_t = \rho_s$: $u_t = 1$
$\rho_t = \rho_0$: $u_t = 0$

Experiments in the Netherlands at a number of ship locks of various dimensions have led to the following semi-empirical formula.

$$u_t = \tanh\left(\frac{t}{4L}\left(\frac{\Delta\rho}{\rho_0}gH\right)^{\frac{1}{2}}\right) \qquad \textit{Equation 4.34}$$

where $\rho = \rho_s - \rho_o$

If the dimensionless parameter $\frac{t}{4L}\left(\frac{\Delta\rho}{\rho_0}gH\right)^{\frac{1}{2}}$ is indicated by T:

$$u_t = \tanh(T) \qquad \textit{Equation 4.35}$$

Example

L	= 100 m
t	= 5 min = 300 sec = opening time of gates
$\Delta\rho/\rho$	= 0.02
H	= 4 m
T	= 300/400 $(0.02x9.8x4)^{0.5}$ = 0.664
u_t	= tgh T = tgh 0.664 = 0.58

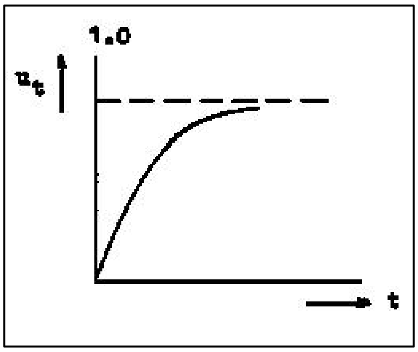


Figure 4.30

So the degree of exchange 5 minutes after opening of the outer gates is 58%. A similar process takes place when afterwards the inner gates are opened.

Preventive measures

There are several ways of preventing or reducing the entrance of sea water at navigation locks.

a) A drastic but expensive way is to pump back the saline water in the lock chamber to the sea and to replace this water by fresh water from the canal. The saline water is removed through openings in the floor of chamber and the fresh water is added on the surface.

b) An economic way of obtaining a reduction of 70 - 50% is found in the injection of air bubbles during the time that the gates are open thus hampering the exchange.

c) The saline water entered at a lock can be collected in a sump at the canal side of the lock and removed from there by pumping or gravity (Seattle U.S.A.).

d) To reduce the amount of salt entering the lock chamber can be subdivided so that smaller boats can be locked through without using the entire chamber. Also a second lock of smaller size can be provided.

4.5 Water control in coastal areas

4.5.1 Problems of quantity and quality

The main problems of water control in low-lying coastal areas are the problem of the salt-water intrusion and the problem of the floods.

The saline intrusion is maximum during dry periods with minimum river flow. This is exactly the period with a maximum need for fresh water for irrigation and other purposes.

The possibility of using water with a high salinity is indicated by the tolerance limits for the use of water as drinking water or industrial water or irrigation water.

According to the recommendations of the World Health Organization (WHO) drinking water should not contain more than 200 mg of Cl^- per litre, in special cases not more than 250 (sea water contains 18,000 to 19,000 mg Cl^- per litre and about 36,000 mg per litre of total salts).

The tolerance limit for process water in industries depends, of course, on the type of product. Water for food processing and breweries should not contain more than 100 to 150 mg Cl^- per litre.

The tolerance limit for irrigation depends on soil, crop and water and soil management and should be fixed in consolation with specialized agronomists.

Commonly accepted limits for the use of irrigation water for high yielding rice varieties are 350 mg Cl^- per litre in the early stages of growth (nurseries and seedlings) and 1500 in the later stages when the plants have become more resistant.

For irrigation of flowers and sensitive fruits and vegetables in hothouses with an artificial climate (Netherlands) limits of 100 to 150 mg Cl^- per litre have been set forth.

In many cases irrigation water with a much higher salinity is accepted. This is applied to salt resistant varieties on pervious soils and under conditions of over-irrigation to leach the soils. This implies adequate drainage facilities to remove the salt and to prevent any accumulation.

Early human settlements along river branches in deltas are often found beyond the saline reach. In the saline reach drinking water could be found in fresh ground-water pockets under dunes, former sandy beach ridges, natural levees and filled-in tidal creeks. Fresh water from excess rainfall during wet periods can be stored in tanks and ponds.

Nowadays desalinization of seawater is applied at increasing rate, especially in arid zones but also in temperate zones. In spite of the high costs (0.3 to 1.0 US \$ per m^3) it may be economical for drinking water and water to irrigate valuable crops (flowers, fruits and vegetables in arid zones).

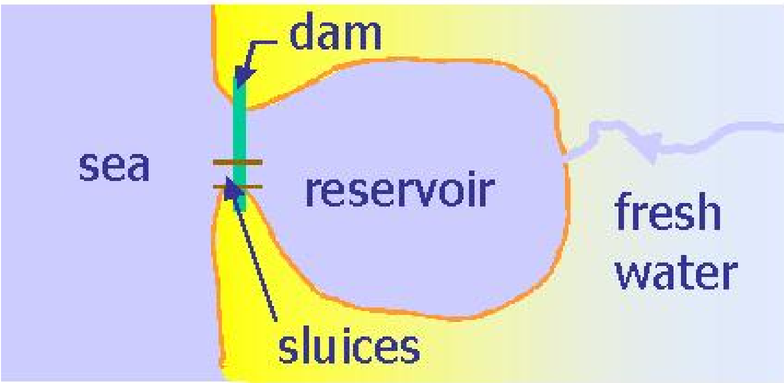


Figure 4.31

Possibilities for large-scale storage of fresh water in low-lying coastal areas can be created by damming off estuaries, coastal lagoons and tidal embayments with an inflow of fresh water. In this way an estuarine or coastal reservoir is formed which is separated from the sea by a dam equipped with sluices for removal of excess water from the reservoir. The originally saline water is replaced by the fresh water from the river. These reservoirs are dealt with in section 4.5.2.

The most generally applied means of flood protection in coastal areas consists of throwing up embankments or dikes. Embanking interferes in the natural hydrological conditions and may have a number of serious side effects and environmental impacts, which will be considered in section 4.5.3.

4.5.2 Coastal reservoirs

General design

In the context of these lectures, only the hydrological aspects of coastal reservoirs will be considered, not the structural aspects of dam building and closing operation.

Coastal reservoirs have been built in a number of countries (Japan, the Netherlands, Korea, Hong Kong, Bangladesh, Sri Lanka, England, etc.). These are multiple purpose reservoirs:

1. by shortening the coast line the salt water intrusion is reduced;
2. fresh water from the river can be stored;
3. a better defence of the adjacent low-lying areas against storm surges is obtained;
4. the drainage of these areas can be improved.

The technical feasibility of such reservoirs depend on:

1. The length of the period of desalinization.
2. The ultimate salinity of the water in the reservoir after the desalinization.
3. The water balance of the reservoir in connection with the regulation of the normal operational level.

The hydrologic design of coastal reservoirs requires the establishment after the water balance of a so-called salt balance in which incoming and outgoing amounts of salt are considered to determine the salinity of the water in the reservoir.

Water balance
In the water balance the following items have to be considered:

Table 4.1: Water balance

IN	OUT
River discharge Drainage on the reservoir Rain on the reservoir Decrease in storage	Evaporation Drainage to the sea Abstraction of water Increase in storage

The water balance is especially important during periods with river floods with a maximum of inflow of water to the reservoirs and during dry periods when water is withdrawn from the reservoir for water supply to the neighbouring areas.

In most cases the normal operational level of the reservoir is around MSL so that excess water can be drained off to the sea by gravity using the period of low water at sea (tidal drainage). During the period of high water at sea the sluice gates remain closed and the reservoir level rises due to the inflow of water to the reservoir.

On those coasts where under the effect of on-shore winds a set-up of the water level occurs, drainage may be hampered during periods with abnormally high sea levels. Then the inflow must be stored and a temporary high reservoir level can occur. Thus the area of the reservoir is an important factor governing the maximum level. This level can be reduced if during the wet season with the floods the normal operational level is maintained as low as possible (still above LW level) so that empty storage space is available to accommodate the floodwaters.

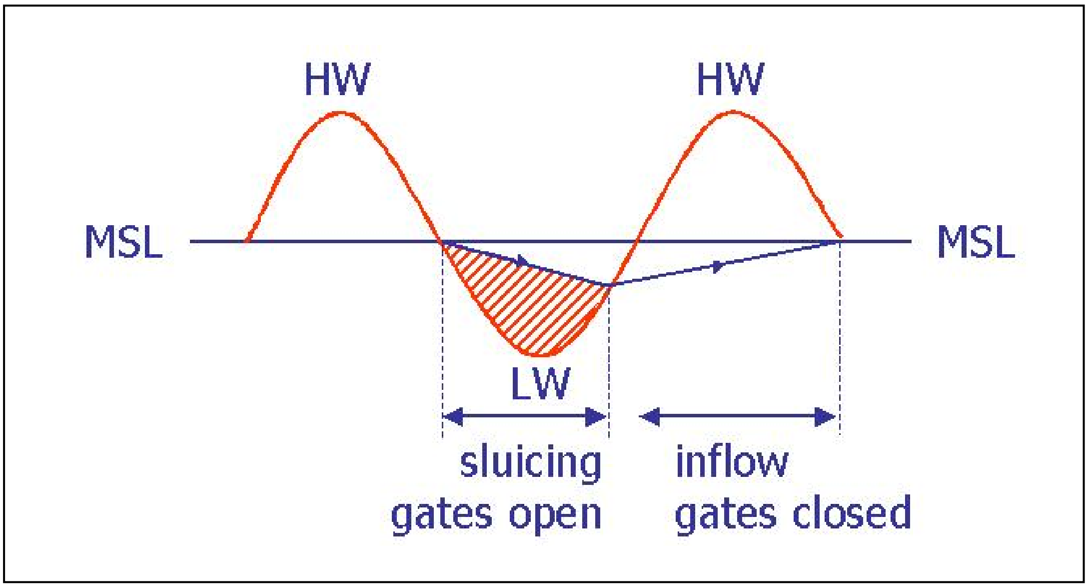


Figure 4.32

During dry periods the gates remain closed. The reservoir level is going down because of abstraction of water from the reservoir and evaporation. The lowest water level can be reduced if during the dry season the normal operational level is maintained at a high elevation so that a maximum amount of water is available to meet the demands. This leads to reservoir regulation somewhat similar to that of upstream reservoirs in the mountains.

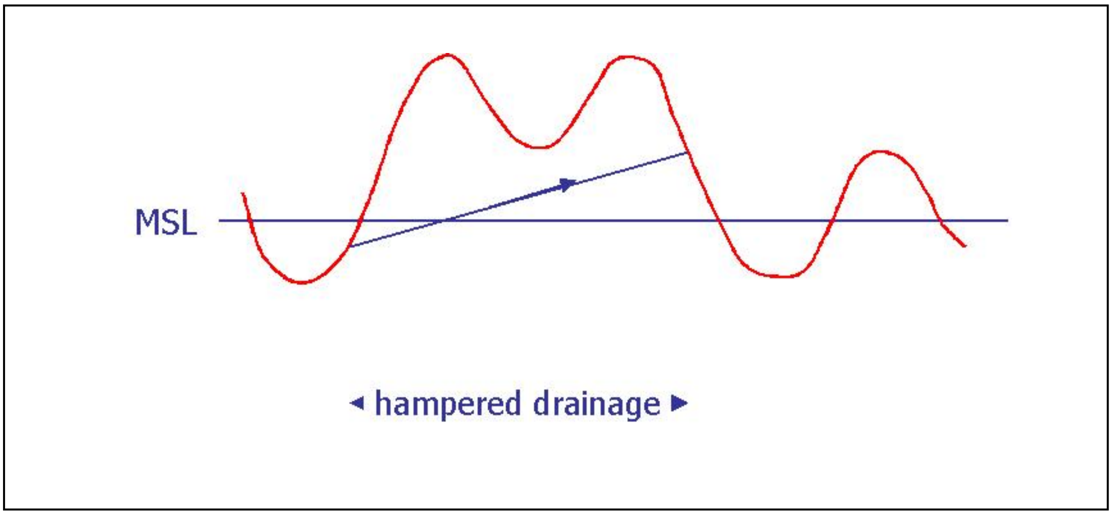


Figure 4.33

Salt balance

Coastal reservoirs are exposed to salt water intrusion and in the salt balance (after the initial period of desalinization) the following items should be considered:

Table 4.2

IN	OUT
Salt load of the river	Drained off to sea
Drainage of brackish water on the reservoir	Abstraction of water from the reservoir
Underground inflow of saline water	Increase in amount of salt stored
Diffusion of salt from the bottom	
Locking of ships	
Leakage of sluice gates	
Decrease in amount of salt stored	

No salt is removed from the reservoir by evaporation. In principle the ultimate salinity of the water in the reservoir under steady conditions is given by the equation:

$$c_\infty = \frac{\text{amount of salt reaching the reservoir}}{\text{amount of water drained off}} \qquad \textit{Equation 4.36}$$

A numerical example is given at the end of this section.

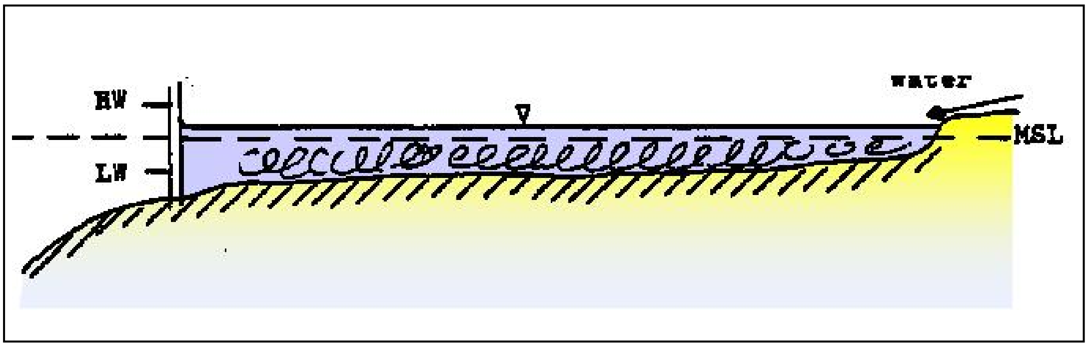


Figure 4.34

The initial desalinization of the reservoir after the closing of the dam can be achieved in two ways:

1. Removal of the saline water by pumping to the sea. The only case known is the Glover Cove Scheme in Hong Kong.

2. Gradual desalinization by draining off to the sea the mixture of the saline water with the fresh water supplied by the river. This is generally applied. The mixing is effectuated by winds and waves on the reservoir and turbulent motion. This works well in shallow reservoirs (up to 4 or 6 m) with a flat bottom.

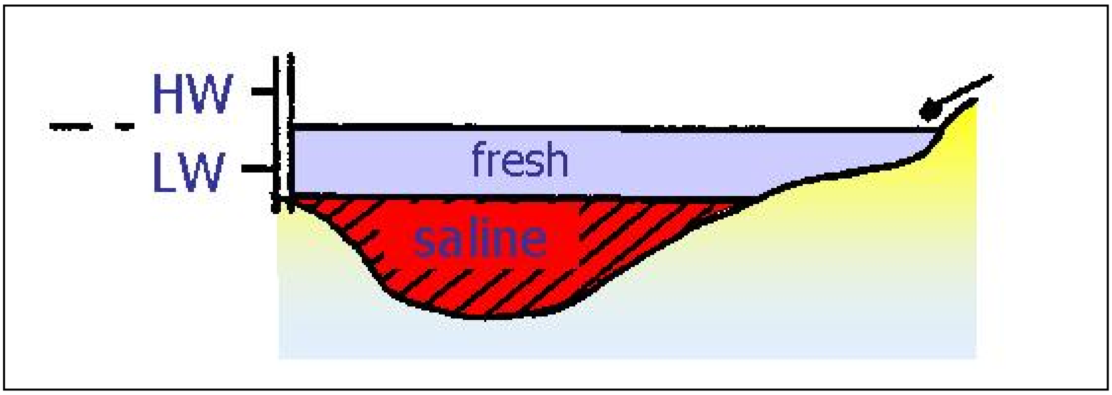


Figure 4.35: In the Hong Kong case saline water would remain stagnant in the deep pocket behind the dam.

The variations of salinity with time (including the desalinization) can be described by a fundamental equation, the mixing equation of theoretical physics (see Figure 4.36). This equation is the same for a container, a coastal reservoir or a column of soil:

- q = rate of supply of water with a concentration c'
- e = net evaporation rate (evaporation minus rainfall)
- d = drainage rate
- V_o = volume of water
- c = concentration of water in the reservoir $c = c(t)$

At the time $t = 0$ $c = c_o$.

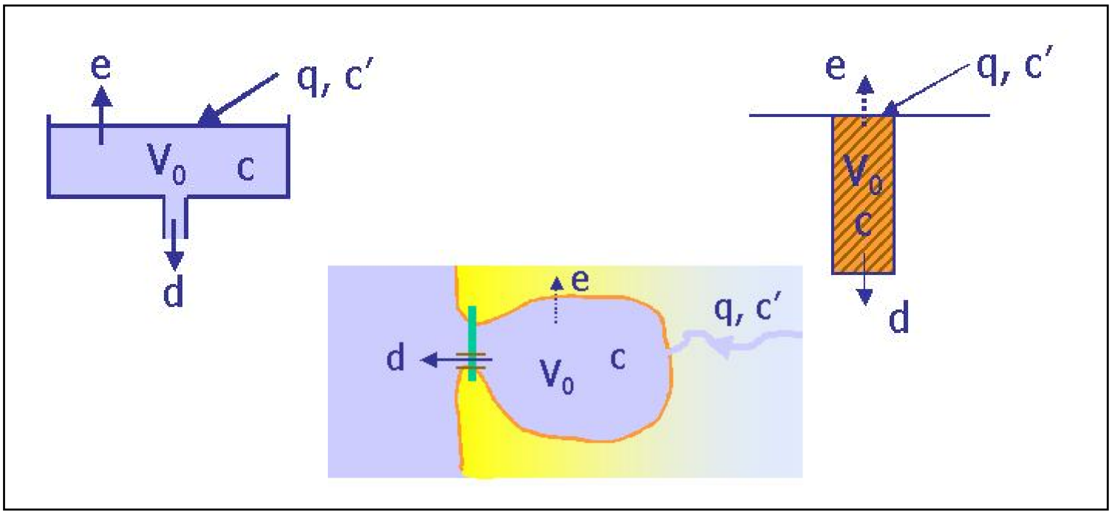


Figure 4.36

The rate of supply of salt is *q.c*'. If q is expressed in $10^6 m^3$ per month and *c*' in kg/m^3 *q.c*' is the rate of supply of salt expressed in 10^6 kg per month. In the case of the coastal reservoir *q.c*' stands for all inflows of salt mentioned in the salt balance.

To obtain a simple analytical solution it is supposed that *q*, *c*', *e*, *d* and V_o are constant and that only *c* varies with time. It is also assumed that there is complete mixing and hence a homogeneous distribution of the concentration.

Then for a time increment d*t*, the water balance reads:

$$q \cdot dt = e \cdot dt + d \cdot dt \qquad \text{Equation 4.37}$$

or

$$q = e + d \qquad \text{Equation 4.38}$$

and the salt balance:

$$qc' \cdot dt = -dc \cdot dt + V_0 dc \qquad \text{Equation 4.39}$$

being, respectively, the amount supplied, the amount drained and the increase in storage. Per unit time this yields:

$$V_0 \frac{dc}{dt} = -dc + qc' \qquad \text{Equation 4.40}$$

In order to solve this equation, first the reduced equation is solved:

$$V_0 = \frac{dc}{dt} = -dc \qquad \text{Equation 4.41}$$

Since, apart from constants, the function c must be equal to its derivative the function is an exponential one, hence:

$$c = K \exp\left(-\frac{d}{V_0} t \right) \qquad \text{Equation 4.42}$$

The non-reduced equation then has the form:

$$c = K \exp\left(-\frac{d}{V_0} t\right) + N$$ *Equation 4.43*

The value of *N* is found by substitution:

$$N = \frac{qc'}{d}$$ *Equation 4.44*

Substitution of the boundary condition $c=c_0$ when t=0 yields an expression for *K*:

$$K = c_0 - N$$ *Equation 4.45*

hence the total equation reads:

$$c - c_\infty = (c_0 - c_\infty) \exp\left(-\frac{d}{V_0} t\right)$$ *Equation 4.46*

c_4 = *N* = the ultimate salinity of the water in the reservoir as mentioned above.

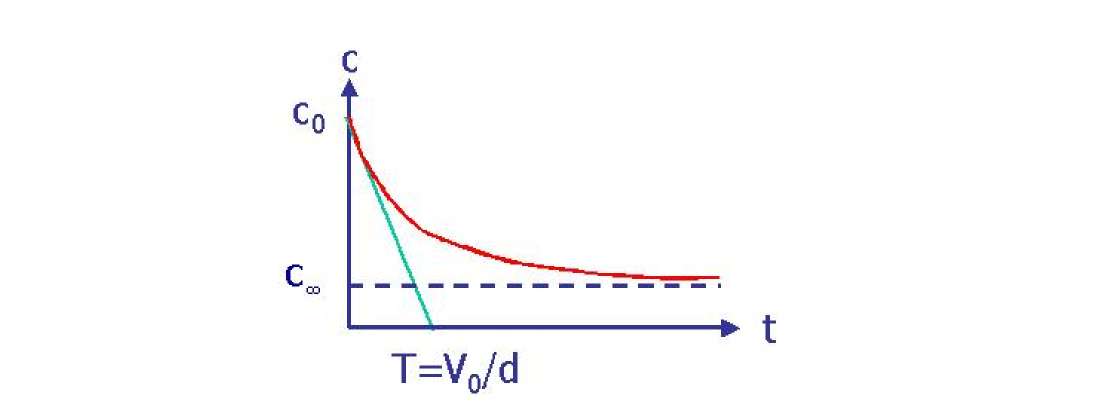


Figure 4.37

The equation yields the following special values:
$t = 0$ $\quad c = c_0$
$t = \infty$ $\quad c = (qc')/d$
$d = 0$ $\quad c = c_0 + (q.c')/ V_0 * t$
The last case is a direct integration of the differential equation with d = 0. It is the case of progressive increase in salinity in the absence of any drainage of salt (soil salinization where in arid climates irrigation water of otherwise low salinity is applied but where lack of drainage leads in the long run, to an accumulation of salt).

For $d \Rightarrow \infty$ (and hence $q \Rightarrow \infty$) $c = c'$, which is logical.
The solution shows that rate of desalinization depends primarily on the ratio d/V_0 (drainage rate in relation to the volume).

If d = drainage rate in 10^6 m^3/year
V_o = volume in 10^6 m^3
t = years

then desalinization expressed as a fraction of the original concentration proceeds as follows:

Table 4.3

After	1 year	2 years	5 years	10 years	20 years
$d = V_0$	0.37	0.135	0.006	0.000	0.000
$d = 0.5\ V_0$	0.61	0.37	0.08	0.006	0.000
$d = 0.1\ V_0$	0.90	0.82	0.61	0.37	0.135

If the period is e.g. 10 years or more the project may not be economically feasible.

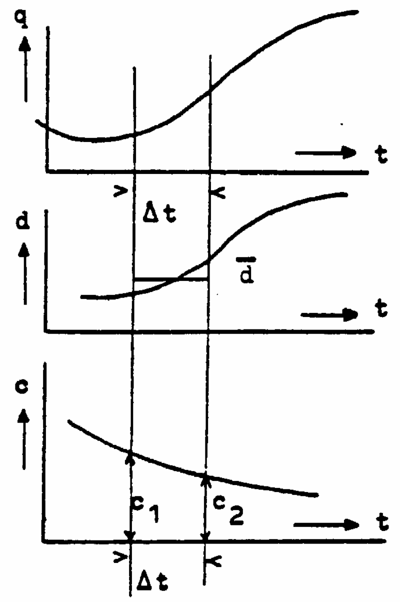


Figure 4.38

In general, however, *q, c', e, d* and V_o are functions of time (seasonal variations) and then discrete periods of time Δt (e.g. = 1 month) can be considered and the water and salt balance be drawn up for each period:

$$V_1 + q\Delta t = V_2 + (e + d)\Delta t$$ *Equation 4.47*

$$c_1 V_1 + qc'\Delta t = c_2 V_2 + \frac{c_1 + c_2}{2} d\Delta t$$ *Equation 4.48*

$$\frac{c_1 + c_2}{2}$$ *Equation 4.49*

From this set of equations c_2 can be determined for each time step:

$$c_2 = \frac{c_1 (V_1 - D/2) + T}{V_2 + D/2}$$ *Equation 4.50*

where $D = d\Delta t$ and $T = qc'\Delta t$

In this way the concentration c_2 at the end of each interval can be computed step by step.

Operation of coastal reservoirs

The requirements of drainage of excess water from floods and supply of water during dry periods leads to reservoir regulation with rule curves for the normal operational level (NOP):

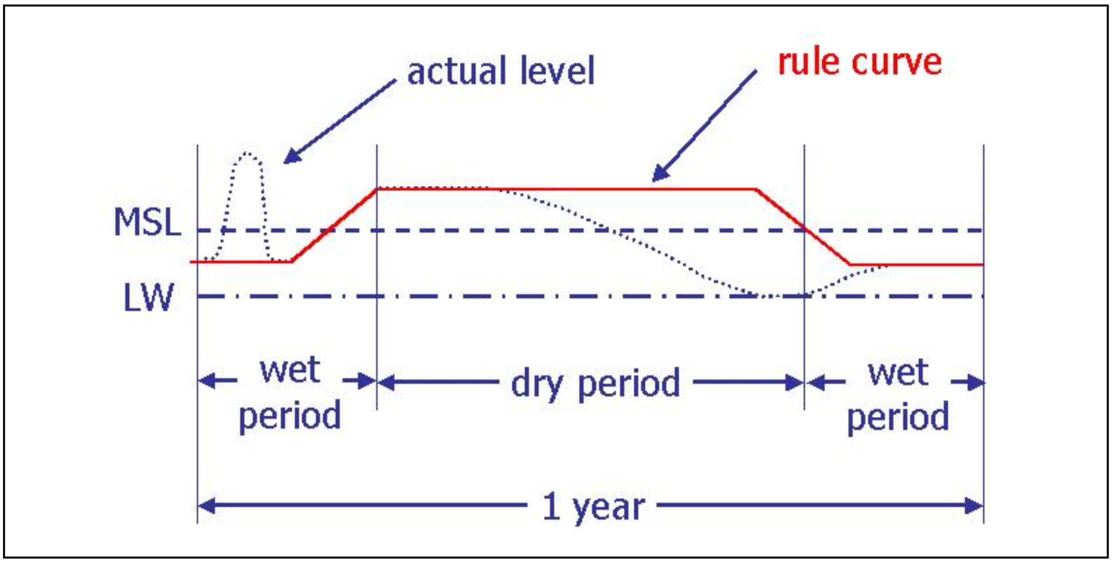


Figure 4.39: Operation rule curves

This works well in a climate with distinct wet and dry seasons. If there is the possibility of the incidence of a flood during the irrigation season (typhoon in Japan) then the operational level will always be low in order to have empty storage space.

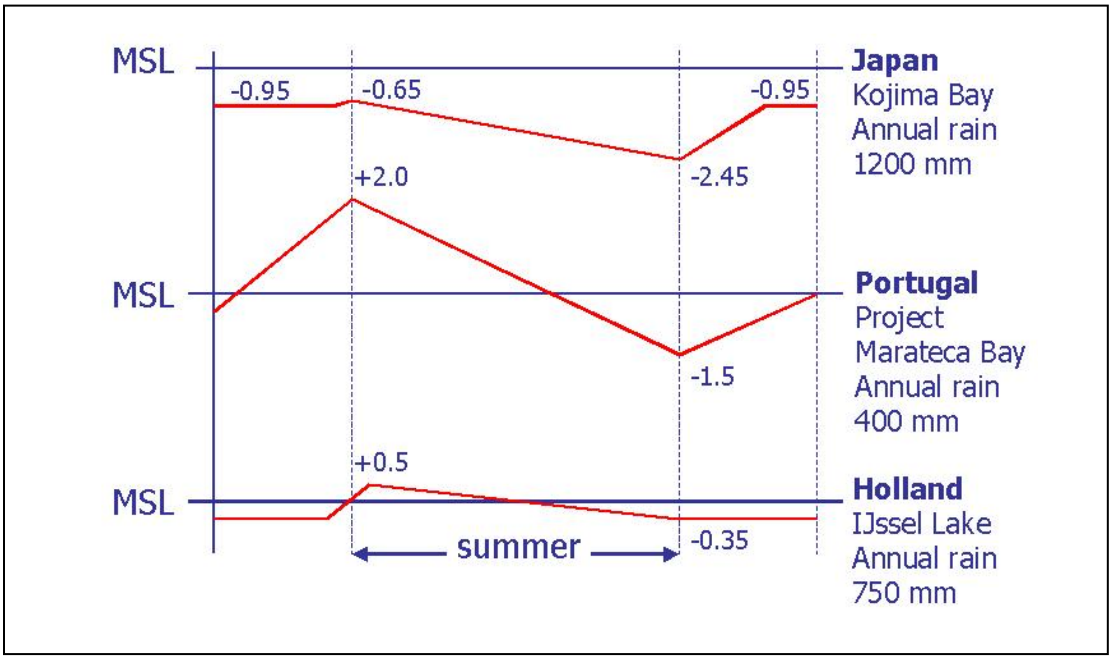


Figure 4.40

Case histories

The tables below show the water and salt balance after desalinization for the IJssel Lake in the Netherlands. The figures refer to the final size of the reservoir (area 1200 km^2) and to conditions of an average year.

Table 4.4: Water balance (in 10^9 m^3 per year)

IN		OUT	
River discharge (IJssel)	10.14	Evaporation	0.80
Drainage high ground	3.30	Abstraction irrigation water	2.00
Drainage polder areas	2.60	Abstraction flushing water	2.00
Rainfall on the lake	0.90	Drainage to the sea	12.14
Locking, leakage	p.m.		
	16.94		16.94

Table 4.5: Salt balance (in 10^6 kg Cl^- per year)

IN		OUT	
Salt load river	1097	Total evacuation	2448
Drainage high grounds	130		
Drainage polder areas	900		
Rainfall on the lake	9		
Locking, leakage	212		
Diffusion	100		
	2448		2448

From this Table it follows that the ultimate salinity of the reservoir for a year with average conditions is:

$$c_\infty = \frac{2448\times10^6 \text{ kg } Cl^-}{(12.14+2.00+2.00)\times10^9 \text{ m}^3} = 152\times10^{-3}\frac{\text{kg } Cl^-}{\text{m}^3} = 152 \text{ mg } Cl^-/l \qquad \textit{Equation 4.51}$$

The initial desalinization was a matter of 5 years (1932-1937) with at that time V_o = 13 x 10^9 m^3 and d = 11.4 x 10^9 m^3/year.

It is clear that the low ultimate salinity is due to the high amount of water, which is drained off to the sea and the relatively small salt load, factors which are characteristics for temperate climates.

Thus the transformation of the brackish Zuiderzee into the fresh water IJssel Lake was quite a success. As an entirely different case stands the failure of the Braakman, a small coastal reservoir in the south-west of the Netherlands which did not become fresh after damming off and initial (partial) desalinization.

As appears from the water and salt balances below the cause of the high salinity lies in an inflow into the reservoir of ground water with salinity almost equal to that of seawater together with a small drainage to the sea. The ground-water flow did not exist before the enclosure but was generated when after damming off a normal operational level of M.S.L. - 0,6 m was adopted, which is 0.6 m lower than the average level before. This was done to reclaim some tidal foreland without embanking.

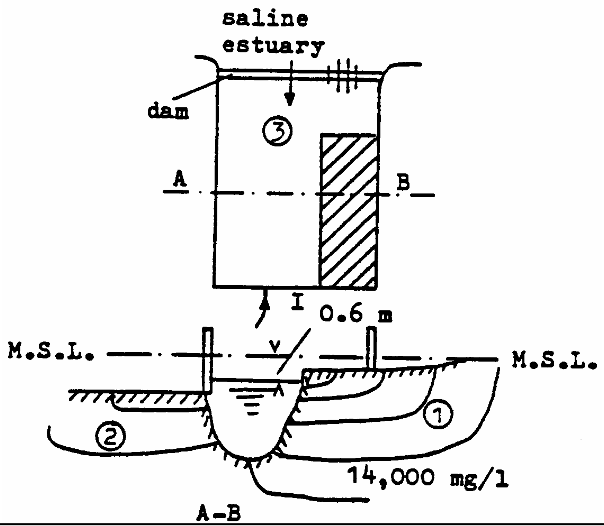


Figure 4.41

The reservoir area is 150 ha $V_o = 5 \times 10^6$ m^3.
The catchment is 1500 ha.

Table 4.6: Water balance for an average year in 10^6 m^3

IN		OUT	
Inflow I from the south	3.00	Loss by seepage (2)	1.64
Rain on reservoir	1.05	Drainage to the sea	2.77
Ground-water inflow (1)	1.09	Evaporation	0.90
Seepage under dam (3)	0.18		
	5.32		5.32

Table 4.7: Salt balance for an average year in 10^6 kgs Cl^-

IN		OUT	
Inflow I from the south	0.3	Loss by seepage (2)	6.68
Rain on reservoir	0.02	Drainage to the sea	11.54
Ground-water inflow (1)	15.34		
Seepage under dam (3)	2.56		
	18.22		18.22

From these tables it follows that the mean salinity in an average year is:

$$\frac{18.22\times10^{6}\ \text{kgs Cl}^{-}}{(1.64+2.77)\times10^{6}\ \text{m}^{3}}=4.1\frac{\text{kg Cl}^{-}}{\text{m}^{3}}=4100\ \text{mg Cl}^{-}/\text{l} \qquad \textit{Equation 4.52}$$

This water is unfit for most uses.

As can be seen from the balances the inflow (1) of ground water from the East plays a relatively small role in the water balance (20% of total inflow) but forms 85% of the total salt load. The generation of this flow after enclosure had been overlooked in the hydrological design.

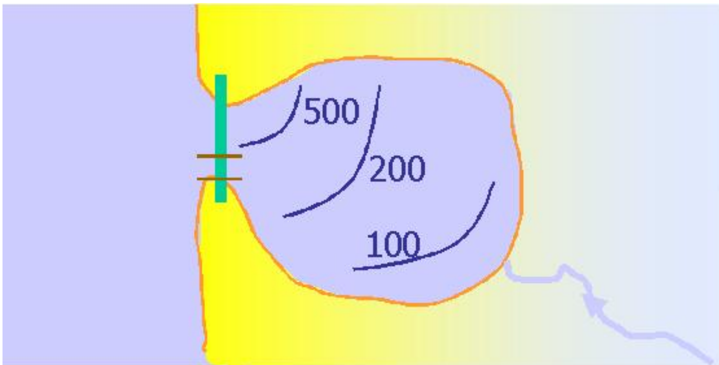


Figure 4.42

The assumption of complete mixing is an approximation. Usually higher concentrations occur near the sources of salt. The concept of the water and salt balance is still valid (continuity) but the computed salinity refers to the concentration of the water that is drained off and not to the average concentration and concentration in any point can be derived from the salinity distribution before enclosure.

4.5.3 Effect of embanking on the hydrological conditions

Flood protection by embanking is a very ancient, simple, and, in most cases, very economical way of preventing flooding. Hence its almost universal application. In many cases embanking provides the only practicable way of flood protection. However, embanking may entail a number of side effects, which may be of such a magnitude that this means of flood protection is not technically feasible. It is often a controversial issue. Embanking has repercussions in the following nine fields:

A. Hydrological effects
1) The longitudinal overland flow.
2) The over-bank storage.
3) The flooding of the strip between the channel and the dike.

B. Morphologic effects
4) The necessity of river training.
5) The natural process of delta building up.
6) The position of the river bed.

C. Environmental effects
7) The hygienic conditions of the land areas.
8) The water management in the land areas.
9) The cropping pattern and farm management.

Failures of flood protection schemes with embankments are often due to the fact that these effects had not been predicted or had been deliberately neglected.

The following considerations refer to the non-tidal reach in a delta and the flood pain.

1. Embanking eliminates the longitudinal overland flow

Before embanking the floodwater is discharged both by channel flow (1.0 to 2.5 m/sec) and by overland flow (0.1 to 0.3 m/sec). Owing to the small depth and high rugosity the velocity of the latter is small but it occurs over a considerable width.

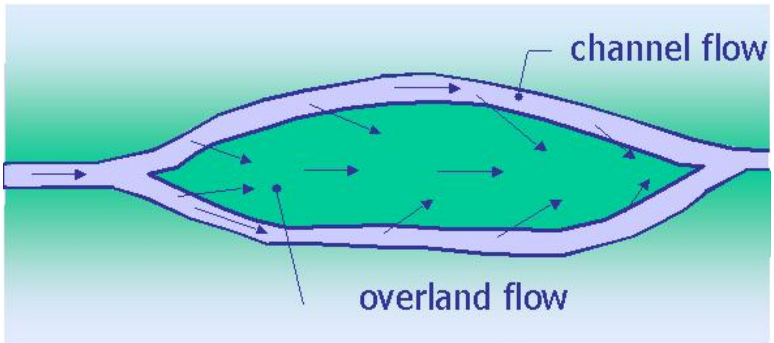


Figure 4.43

Embanking produces a constriction of the flow to the cross-section between the dikes. The channel flow increases and the flood levels are going up. If the flows are known the rise can be roughly estimated starting from a downstream control point, like the M.S.L.

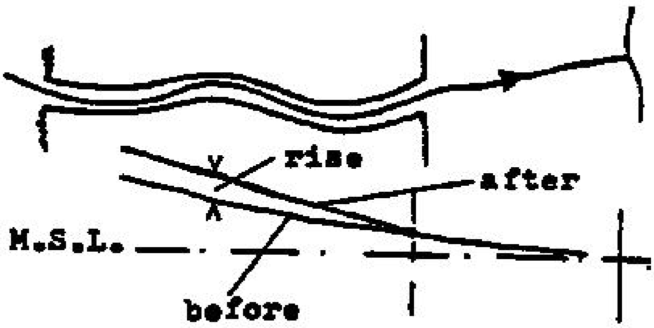


Figure 4.44

The rise can be reduced by setback or retired embankments at some distance away from the channel but this means sacrificing the best (level) soils.

2. *Embanking eliminates the over bank storage*

When during a flood the riverbanks are overtopped water flows in a lateral direction to fill the flood plains. This flow reduces the downstream channel flow from upstream A to downstream B. Thus the flow in B is smaller than in A as long as the water level on the land is still rising.

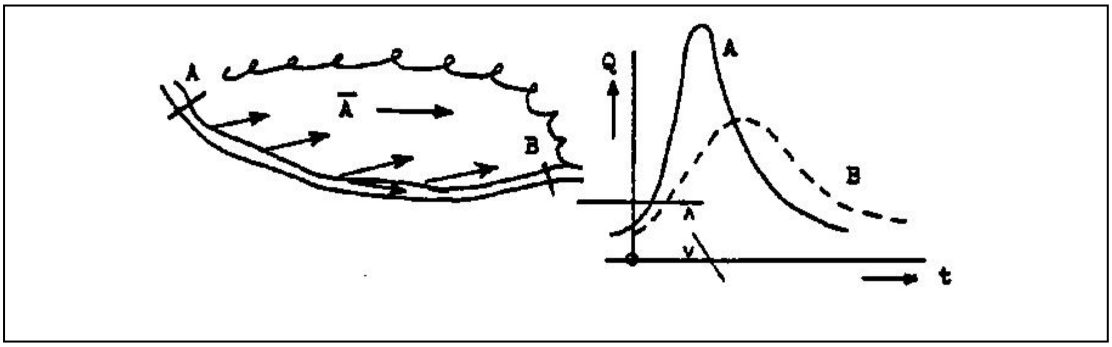


Figure 4.45

The lateral inflow Q_{lat} at any time is given by:

$$Q_{lat} = A\frac{dh}{dt} \qquad \textit{Equation 4.53}$$

where:

A = area of the flood plain
h = depth of flooding of the land

When the flow is receding the hydrograph in B becomes more similar to the one in A. The result is an increase of the peak flow not only between A and B but also downstream of B.

The effect is particularly significant in case of flash floods (rapid rise and sharp peak). When the flood is gentle and has a flat peak the effect is smaller because the flood plain is then filled to almost the same level as the maximum level in the river.

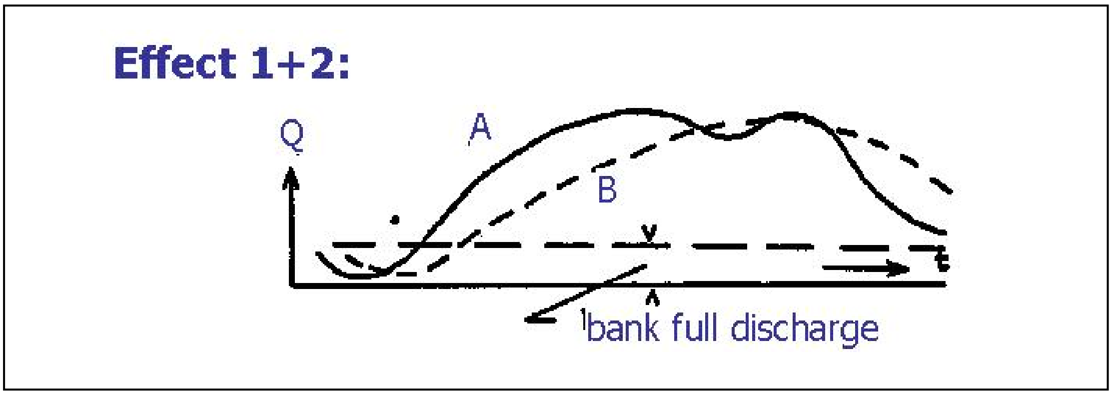


Figure 4.46

The hydrological effects 1 and 2 can roughly be estimated as indicated above. To make a more correct assessment physical or mathematical models can be applied. They require for their calibration extensive data on levels (channels and land) and discharges of floods under the original conditions.

The magnitude of the two effects together may be considerable like in the Pampanga valley and delta where according to a physical model test in case of a major flash flood a rise (upstream) of 2.5 to 3.5 m would occur.

3. Embanking causes more frequent flooding of the strip between the channel and the alignment of the dike

This is precisely where, because of higher elevation and better soils, most people are living and where fruit trees can grow and vegetables be raised.

4. Embanking may render river training necessary

Embanking, because of higher flows, tends to increase river meandering and hence bank erosion. Local protection is often inadequate and river training (groynes upstream) may be necessary. This is many times more expensive than the construction of dikes. Local setback dikes (as 2nd line of defence) are often applied when river training is not feasible (lack of stones).

5. Embankments exclude the deposition of silt and halt therefore the further building-up of the land areas

The much mentioned "fertilizing effect" of the silt is often small in terms of chemical nutrients. The rate of deposition in the lowest parts of the back swamps is often small.

6. Embankments may produce a rise of the river bed

Before embanking the lateral flow carried water as well as sediments to the flood plain. After embanking the channel between the dikes has to carry more material in downstream direction than before. This may entail silting-up. On the other hand the sediment-carrying capacity of the river has also increased (higher flow) and whether actually a silting-up or a scouring will occur depends on which of the two factors prevails.

It should be noted that under natural conditions a rise of the river bed occurs anyhow as a result of delta extension into the sea and hence lengthening of the river course.

7. Embanking affects the hygienic conditions of the land areas

One of the beneficial effects of river floods is the periodic flushing or rinsing with clean and fresh water. All kind of wastes, dirt, human disposal and sometimes salinity accumulated during the dry season is effectively removed. After embanking the water becomes stagnant and special provision may have to be taken to get the required periodic washing out.

8. Embanking affects the water management of the land areas

After embanking a drainage system with sluices or pumping stations and canals has to be installed to remove excess water from local rainfall. This may entail over-drainage and reduction of water conservation so that less water is present in the area at the beginning of the dry season. Supplementary during the dry season by flooding is also eliminated.

A new water management system has to be created with new perspectives for more intensive land use and crop diversification but also a new and unknown exploitation of the farmlands with higher costs and higher returns.

4.5.4 Drainage of level areas

Embanked areas in coastal areas and low-lying coastal areas where the water levels inside can be controlled are called 'polders'. The areas are very flat and horizontal (level areas).

Removal of excess water from local rainfall on these polders often poses serious problems because of the high levels of the receiving waters and the absence of any natural land gradients.

The land in coastal areas has been built up by the river (and the sea) and the general configurations shows natural levees or ridges along the river channels consisting of sediments deposited during the floods and the back swamps or depressions which occupy most of the areas in between.

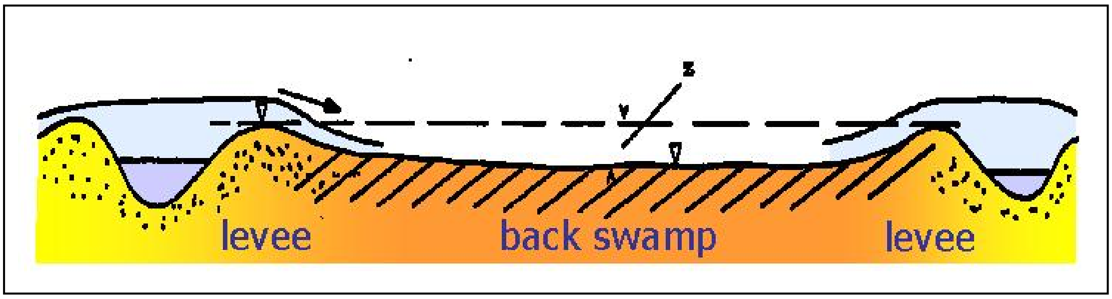


Figure 4.47

Upstream the difference z in elevation between the crest of the natural levee and the back swamp may be as much as 3 to 4 metres. This difference decreases in downstream direction and is almost absent near the coast line (tidal flooding). The natural levees are sandy; in the back swamps heavy soils occur. In the coastal strip the texture is more homogeneous and heavy.

In the river reach the crests of the natural ridges are built up to a level around the annual flood or slightly higher. In the tidal reach the level is around mean high tide. The back swamp is lower, especially upstream.

During floods higher than the annual flood the whole area would be flooded. Embankments can prevent this but the problem of the removal of excess water from local rainfall remains because of the high river level.

It is possible, of course, to remove excess water by pumping but this is expensive. If the floods have short duration, excess water can temporarily be stored in the polders and released later but this is not possible if the high flood levels persist for several weeks.

As can be seen from the longitudinal profile there is a strip near the coast where the land elevation of the back swamp is higher than the low water level of the river even when there is a flood so that gravity drainage is possible (tidal drainage). The distance d must be above a certain minimum.

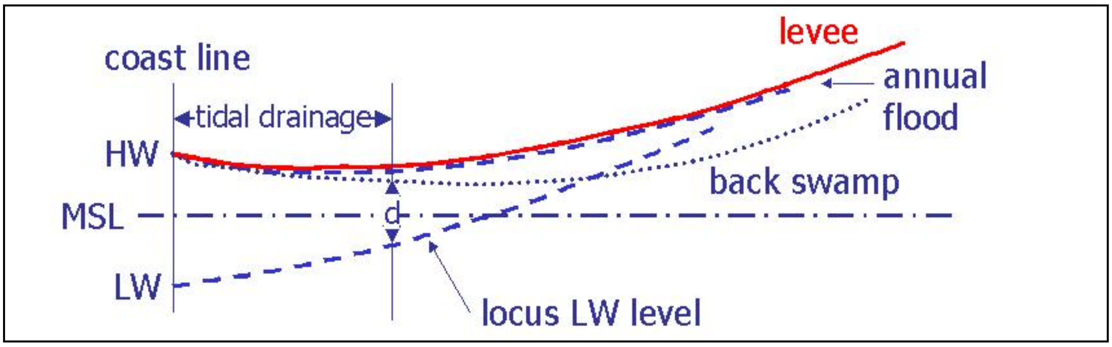


Figure 4.48

Tidal drainage is an intermittent drainage using the period that the water level of the recipient basin is lower than the water level of the canal at the inner side of the sluice. During the high water the gates remain closed and the water level inside may rise due to inflow of drainage water.

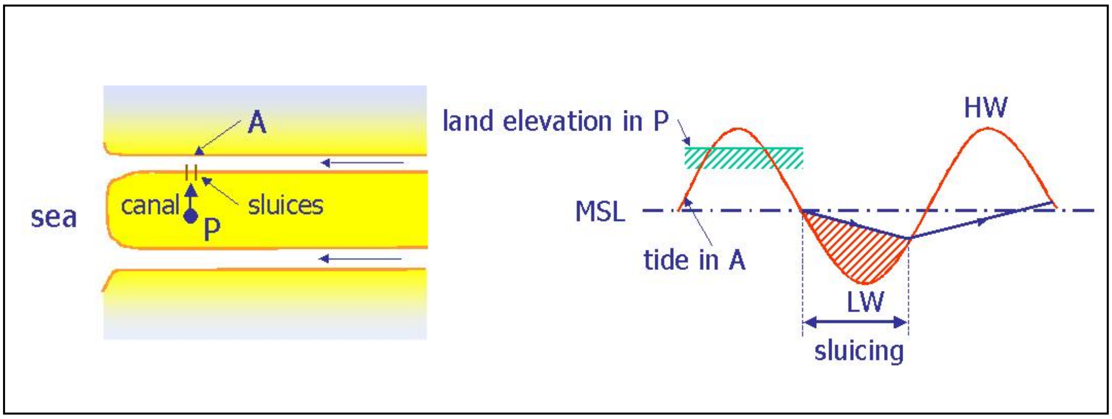


Figure 4.49

The flow through the sluice may be critical or sub-critical depending on the depth of the sill with respect to the tidal level.

In P the water level in the minor drainage at the time of maximum flow should be 1.0 to 1.2 m below ground elevation for dry soil crops and 0.2 to 0.4 m for paddy. The gradient for the flow in the drainage canals from P to A is taken as small as possible, e.g. 5×10^{-5}. For a distance of, for instance, 10 km this means 0.5 m. The head in the sluice at the time of maximum flow around LW is, say, 0.3 m. In conclusion the minimum distance d must be 1.8 - 2.0 m for dry soil crops and 1.0 to 1.2 m for paddy.

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