WATER RESOURCE ENGINEERING (3-1-0)

Module –I

Hydrologic cycle, availability of water on earth, importance of hydrology and its applications in engineering, Precipitation: Forms & types, measurement of rainfall, optimum number of rain guage stations, consistency of rainfall data, presentation of precipitation data, mean aerial rainfall, depth – area duration curve, design storm, losses from precipitation, evaporation, and infiltration.

THE HYDROLOGICAL CYCLE

As a starting point for the study of hydrology it is useful to consider the **hydrological cycle**. This is a conceptual model of how water moves around between the earth and atmosphere in different states as a gas, liquid or solid. As with any conceptual model it contains many gross simplifications; these are discussed in this section. There are different scales that the hydrological cycle can be viewed at, but it is helpful to start at the large global scale and then move to the smaller hydrological unit of a river basin or catchment.

The global hydrological cycle

Table 1.2 sets out an estimate for the amount of water held on the earth at a single time. These figures are extremely hard to estimate accurately. Estimates cited in Gleick (1993) show a range in total from 1.36 to 1.45 thousand million (or US billion) cubic kilometres of water. The vast majority of this is contained in the oceans and seas. If you were to count groundwater less than 1 km in depth as ‘available’ and discount snow and ice, then the total percentage of water available for human consumption is around 0.27 per cent. Although this sounds very little it works out at about 146 million litres of water per person per day (assuming a world population of 7 billion); hence the ease with which Stumm (1986) was able to state that there is enough to satisfy all human needs.

Table 1.2 Estimated volumes of water held at the earth's surface

	Volume ($\times 10^3$ km^3)	*Percentage of total*
Oceans and seas	1,338,000	96.54
Ice caps and glaciers	24,064	1.74
Groundwater	23,400	1.69
Permafrost	300	0.022
Lakes	176	0.013
Soil	16.5	0.001
Atmosphere	12.9	0.0009
Marsh/wetlands	11.5	0.0008
Rivers	2.12	0.00015
Biota	1.12	0.00008
Total	1,385,984	100.00

Source: Data from Shiklomanov and Sokolov (1983)

Figure 1.6 shows the movement of water around the earth–atmosphere system and is a representation of the global hydrological cycle. The cycle consists of **evaporation** of liquid water into water vapour that is moved around the atmosphere. At some stage the water vapour condenses into a liquid (or solid) again and falls to the surface as **precipitation**. The oceans evaporate more water than they receive as precipitation, while the opposite is true over the continents. The difference between precipitation and evaporation in the terrestrial zone is **runoff**, water moving over or under the surface towards the oceans, which completes the hydrological cycle. As can be seen in Figure 1.6 the vast majority of evaporation and precipitation occurs over the oceans. Ironically this means that the terrestrial zone, which is of greatest concern to hydrologists, is actually rather insignificant in global terms.

The three parts shown in Figure 1.6 (evaporation, precipitation and runoff) are the fundamental processes of concern in hydrology. The figures given in the diagram are global totals but they vary enormously around the globe. This is illustrated in

Figure 1.7 which shows how total precipitation is partitioned towards different hydrological processes in differing amounts depending on climate. In temperate climates (i.e. non tropical or polar) around one third of precipitation becomes evaporation, one third surface runoff and the final third as groundwater recharge. In arid and semi-arid regions the proportion of evaporation is much greater, at the expense of groundwater recharge.

With the advent of satellite monitoring of the earth's surface in the past thirty years it is now possible to gather information on the global distribution of these three processes and hence view how the hydrological cycle varies around the world. In Plates 1 and 2 there are two images of global **rainfall** distribution during 1995, one for January and another for July.

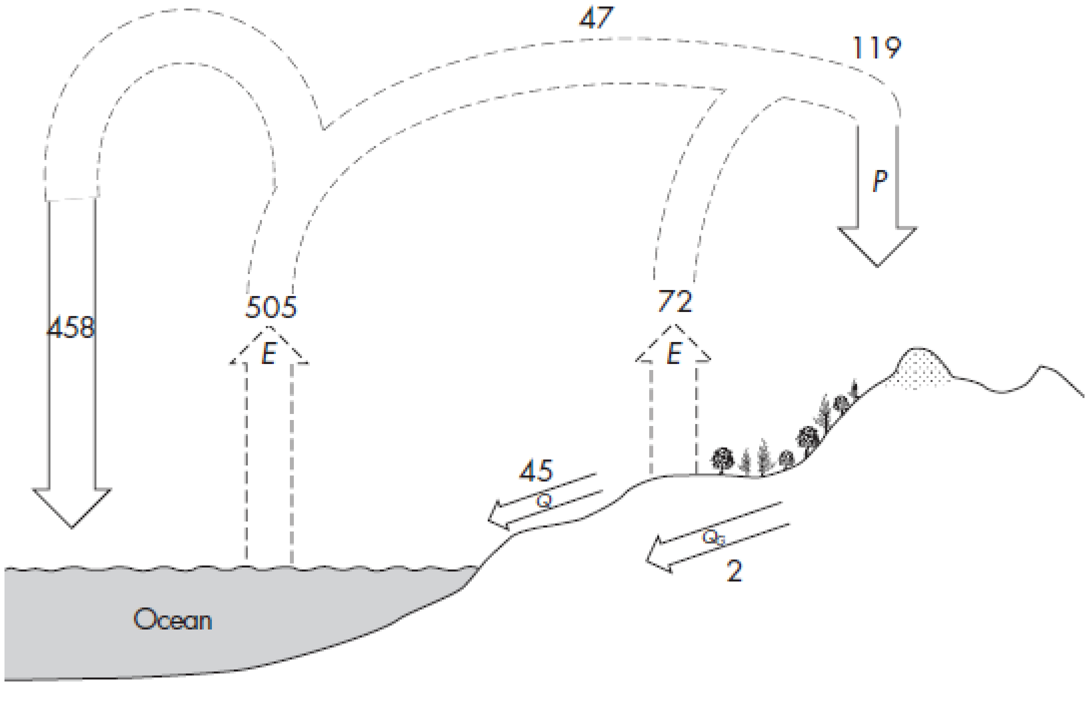

Figure 1.6 The global hydrological cycle. The numbers represent estimates on the total amount of water (thousands of km^3) in each process per annum. E = evaporation; P = precipitation; Q_G = subsurface runoff; Q = surface runoff.

Source: Redrawn from Shiklomanov (1993)

The figure given above of 146 million litres of fresh water per person per year is extremely misleading, as the distribution of available water around the globe varies enormously. The concept of available water considers not only the distribution of rainfall but also population. Table 1.3 gives some indication of those countries that could be considered water rich and water poor in terms of available water. Even this is misleading as a country such as Australia is so large that the high rainfall received in the tropical north-west compensates for the extreme lack of rainfall elsewhere; hence it is considered water rich. The use of rainfall alone is also misleading as it does not consider the importation of water passing across borders, through rivers and groundwater movement.

Table 1.3 gives the amount of available water for various countries, but this takes no account for the amount of water abstracted for actual usage. Figure 1.8 shows the water abstraction per capita for all of the OECD countries. This shows that the USA, Canada and Australia are very high water users, reflecting a very large amount of water used for agricultural and industrial production. The largest water user is the USA with 1,730 m3 per capita per annum, which is still only 1 per cent of the 146 million litres per capita per annum derived from the Stumm quote. Australia as a high water user has run into enormous difficulties in the years 2005–2007 with severe drought, limiting water availability for domestic and agricultural users. In a situation like this the way that water is allocated (see Chapter 8) literally becomes a matter of life and death, and many economic livelihoods depend on equitable allocation of a scarce water resource.

To try and overcome some of the difficulties in interpreting the data in Figure 1.6 and Table 1.2 hydrologists often work at a scale of more relevance to the physical processes occurring. This is frequently the water basin or catchment scale (Figures 1.4 and 1.5).

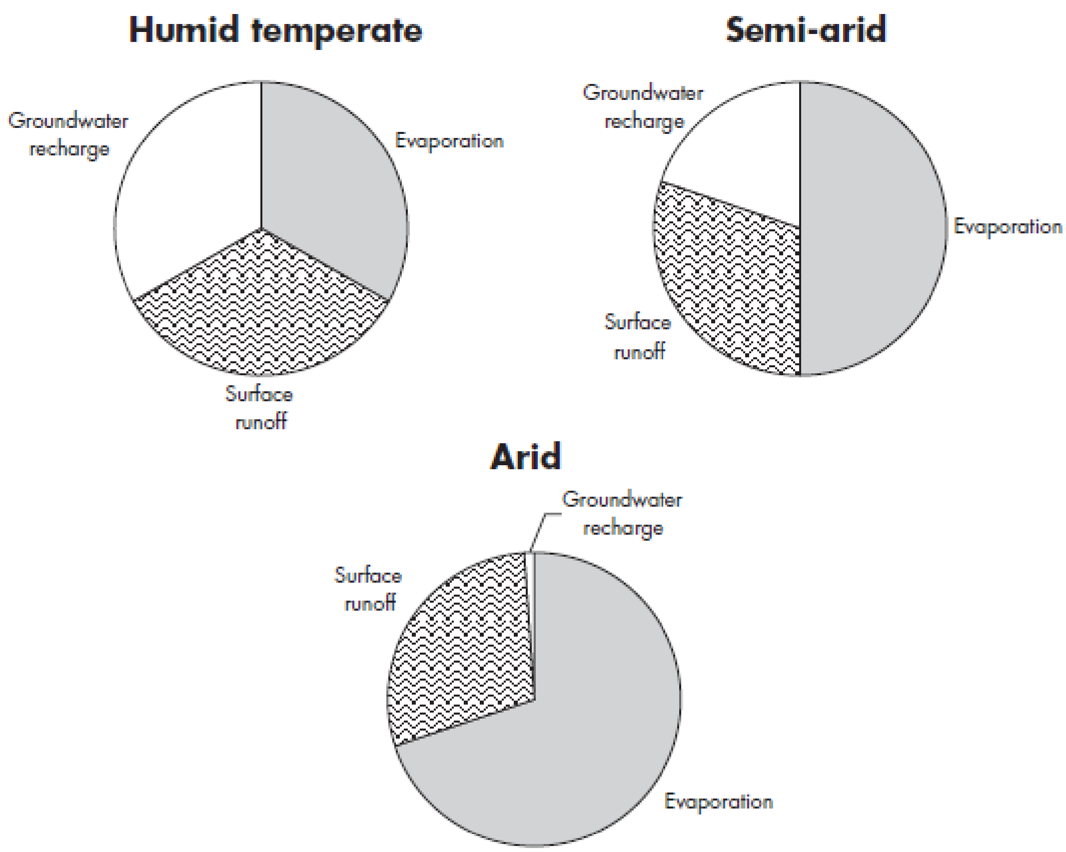

Figure 1.7 Proportion of total precipitation that returns to evaporation, surface runoff or groundwater recharge in three different climate zones.

Source: UNESCO (2006)

The catchment or river basin

In studying hydrology the most common spatial unit of consideration is the **catchment** or **river basin**. This can be defined as the area of land from which water flows towards a river and then in that river to the sea. The terminology suggests that the area is analogous to a basin where all water moves towards a central point (i.e. the plug hole, or in this case, the river mouth). The common denominator of any point in a catchment is that wherever rain falls, it will end up in the same place: where the river meets the sea (unless lost through evaporation). A catchment may range in size from a matter of hectares to millions of square kilometres.

A river basin can be defined in terms of its topography through the assumption that all water falling on the surface flows downhill. In this way a catchment boundary can be drawn (as in Figures 1.4 and 1.5) which defines the actual catchment area for a river basin. The assumption that all water flows downhill to the river is not always correct, especially where the underlying geology of a catchment is complicated. It is possible for water to flow as groundwater into another catchment area, creating a problem for the definition of 'catchment area'.

These problems aside, the catchment does provide an important spatial unit for hydrologists to consider how water is moving about and is distributed at a certain time.

The catchment hydrological cycle

At a smaller scale it is possible to view the catchment hydrological cycle as a more in-depth conceptual model of the hydrological processes operating.

Figure 1.9 shows an adaptation of the global hydrological cycle to show the processes operating within a catchment. In Figure 1.9 there are still essentially three processes operating (evaporation, precipitation and runoff), but it is possible to subdivide each into different sub-processes. Evaporation is a mixture of open water evaporation (i.e. from rivers and lakes); evaporation from the soil; evaporation from plant surfaces; **interception**; and **transpiration** from plants. Precipitation can be in the form of **snowfall**, hail, rainfall or some mixture of the three (sleet).

Interception of precipitation by plants makes the water available for evaporation again before it even reaches the soil surface. The broad term 'runoff' incorporates the movement of liquid water above and below the surface of the earth. The movement of water below the surface necessitates an understanding of infiltration into the soil and how the water moves in the unsaturated zone (**throughflow**) and in the saturated zone (**groundwater flow**). All of these processes and sub-processes are dealt with in detail in later chapters; what is important to realise at this stage is that it is part of one continuous cycle that moves water around the globe and that they may all be operating at different times within a river basin.

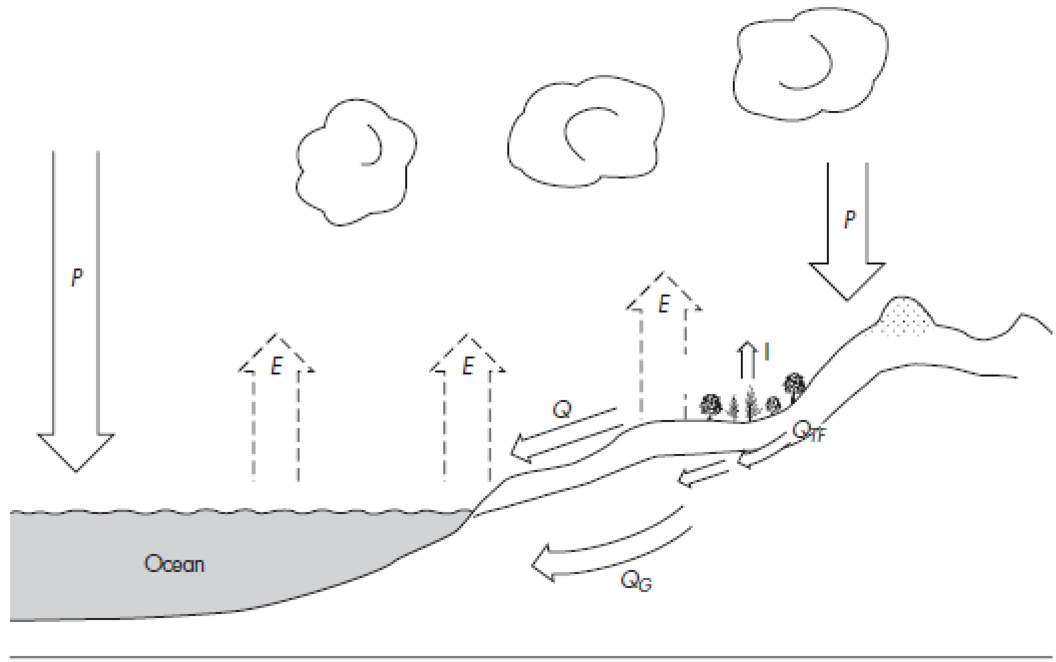

Figure 1.9 Processes in the hydrological cycle operating at the basin or catchment scale. Q = runoff; the subscript G stands for groundwater flow; TF for throughflow; I = interception; E = evaporation; P = precipitation.

THE WATER BALANCE EQUATION

In the previous section it was stated that the hydrological cycle is a conceptual model representing our understanding of which processes are operating within an overall earth–atmosphere system. It is also possible to represent this in the form of an equation, which is normally termed the **water balance equation**. The water balance equation is a mathematical description of the hydrological processes operating within a given timeframe and incorporates principles of mass and energy continuity. In this way the hydrological cycle is defined as a closed system whereby there is no mass or energy created or lost within it. The mass of concern in this case is water.

There are numerous ways of representing the water balance equation but equation 1.1 shows it in its most fundamental form.

$P \pm E \pm \_S \pm Q = 0$ (1.1)

where *P* is precipitation; *E* is evaporation; _*S* is the change in **storage** and *Q* is runoff. Runoff is normally given the notation of *Q* to distinguish it from rainfall which is often given the symbol *R* and frequently forms the major component of precipitation.

The ± terminology in equation 1.1 represents the fact that each term can be either positive or negative depending on which way you view it – for example, precipitation is a gain (positive) to the earth but a loss (negative) to the atmosphere.

As most hydrology is concerned with water on or about the earth's surface it is customary to consider the terms as positive when they represent a gain to the earth.

Two of the more common ways of expressing the water balance are shown in equations 1.2 and 1.3

P – *Q* – *E* – _*S* = 0 (1.2)

Q = *P* – *E* – _*S* (1.3)

In equations 1.2 and 1.3 the change in storage term can be either positive or negative, as water can be released from storage (negative) or absorbed into storage (positive).

The terms in the water balance equation can be recognised as a series of fluxes and stores. A **flux** is a rate of flow of some quantity (Goudie *et al*., 1994): in the case of hydrology the quantity is water. The water balance equation assesses the relative flux of water to and from the surface with a storage term also incorporated. A large part of hydrology is involved in measuring or estimating the amount of water involved in this flux transfer and storage of water.

Precipitation in the water balance equation represents the main input of water to a surface (e.g. a catchment). As explained on p. 10, precipitation is a flux of both rainfall and snowfall. Evaporation as a flux includes that from open water bodies (lakes, ponds, rivers), the soil surface and vegetation (including both interception and transpiration from plants). The storage term includes soil moisture, deep groundwater, water in lakes, glaciers, seasonal snow cover. The runoff flux is also explained on p. 10. In essence it is the movement of liquid water above and below the surface of the earth.

The water balance equation is probably the closest that hydrology comes to having a fundamental theory underlying it as a science, and hence almost all hydrological study is based around it. Field catchment studies are frequently trying to measure the different components of the equation in order to assess others. Nearly all hydrological **models** attempt to solve the equation for a given time span – for example, by knowing the amount of rainfall for a given area and estimating the amount of evaporation and change in storage it is possible to calculate the amount of runoff that might be expected.

Despite its position as a fundamental hydrological theory there is still considerable uncertainty about the application of the water balance equation.

It is not an uncertainty about the equation itself but rather about how it may be applied. The problem is that all of the processes occur at a spatial and temporal scale (i.e. they operate over a period of time and within a certain area) that may not coincide with the scale at which we make our measurement or estimation.

PRECIPITATION

PRECIPITATION AS A PROCESS

Precipitation is the release of water from the atmosphere to reach the surface of the earth. The term 'precipitation' covers all forms of water being released by the atmosphere, including snow, hail, sleet and rainfall. It is the major input of water to a river catchment area and as such needs careful assessment in any hydrological study. Although rainfall is relatively straightforward to measure (other forms of precipitation are more difficult) it is notoriously difficult to measure *accurately* and, to compound the problem, is also extremely variable within a catchment area.

Precipitation formation

The ability of air to hold water vapour is temperature dependent: the cooler the air the less water vapour is retained. If a body of warm, moist air is cooled then it will become saturated with water

vapour and eventually the water vapour will condense into liquid or solid water (i.e. water or ice droplets). The water will not condense spontaneously however; there need to be minute particles present in the atmosphere, called **condensation nuclei**, upon which the water or ice droplets form.

The water or ice droplets that form on condensation nuclei are normally too small to fall to the surface as precipitation; they need to grow in order to have enough mass to overcome uplifting forces within a cloud. So there are three conditions that need to be met prior to precipitation forming:

- Cooling of the atmosphere
- Condensation onto nuclei
- Growth of the water/ice droplets.

Atmospheric cooling

Cooling of the atmosphere may take place through several different mechanisms occurring independently or simultaneously. The most common form of cooling is from the uplift of air through the atmosphere.

As air rises the pressure decreases; Boyle's Law states that this will lead to a corresponding cooling in temperature. The cooler temperature leads to less water vapour being retained by the air and conditions becoming favourable for **condensation**.

The actual uplift of air may be caused by heating from the earth's surface (leading to **convective precipitation**), an air mass being forced to rise over an obstruction such as a mountain range (this leads to **orographic precipitation**), or from a low pressure weather system where the air is constantly being forced upwards (this leads to **cyclonic precipitation**).

Other mechanisms whereby the atmosphere cools include a warm air mass meeting a cooler air mass, and the warm air meeting a cooler object such as the sea or land.

Condensation nuclei

Condensation nuclei are minute particles floating in the atmosphere which provide a surface for the water vapour to condense into liquid water upon.

They are commonly less than a micron (i.e. one millionth of a metre) in diameter. There are many different substances that make condensation nuclei, including small dust particles, sea salts and smoke particles.

Research into generating artificial rainfall has concentrated on the provision of condensation nuclei into clouds, a technique called **cloud seeding**.

During the 1950s and 1960s much effort was expended in using silver iodide particles, dropped from planes, to act as condensation nuclei. However, more recent work has suggested that other salts such as potassium chloride are better nuclei. There is much controversy over the value of cloud seeding.

Some studies support its effectiveness (e.g. Gagin and Neumann, 1981; Ben-Zvi, 1988); other authors query the results (e.g. Rangno and Hobbs, 1995), while others suggest that it only works in certain atmospheric conditions and with certain cloud types (e.g. Changnon *et al*., 1995). More recent work in

South Africa has concentrated on using hygroscopic flares to release chloride salts into the base of convective storms, with some success (Mather *et al*., 1997). Interestingly, this approach was first noticed through the discovery of extra heavy rainfall occurring over a paper mill in South Africa that was emitting potassium chloride from its chimney stack (Mather, 1991).

Water droplet growth

Water or ice droplets formed around condensation nuclei are normally too small to fall directly to the ground; that is, the forces from the upward draught within a cloud are greater than the gravitational forces pulling the microscopic droplet downwards.

In order to overcome the upward draughts it is necessary for the droplets to grow from an initial size of 1 micron to around 3,000 microns (3 mm).

The vapour pressure difference between a droplet and the surrounding air will cause it to grow through condensation, albeit rather slowly. When the water droplet is ice the vapour pressure difference with the surrounding air becomes greater and the water vapour sublimates onto the ice droplet. This will create a precipitation droplet faster than condensation onto a water droplet, but is still a slow process. The main mechanism by which raindrops grow within a cloud is through *collision and coalescence*. Two raindrops collide and join together (coalesce) to form a larger droplet that may then collide with many more before falling towards the surface as rainfall or another form of precipitation.

Another mechanism leading to increased water droplet size is the so-called **Bergeron process**. The pressure exerted within the parcel of air, by having the water vapour present within it, is called the **vapour pressure**. The more water vapour present the greater the vapour pressure. Because there is a maximum amount of water vapour that can be held by the parcel of air there is also a maximum vapour pressure, the so-called **saturation vapour pressure**.

The saturation vapour pressure is greater over a water droplet than an ice droplet because it is easier for water molecules to escape from the surface of a liquid than a solid. This creates a water vapour gradient between water droplets and ice crystals so that water vapour moves from the water droplets to the ice crystals, thereby increasing the size of the ice crystals. Because clouds are usually a mixture of water vapour, water droplets and ice crystals, the

Bergeron process may be a significant factor in making water droplets large enough to become rain drops (or ice/snow crystals) that overcome gravity and fall out of the clouds.

The mechanisms of droplet formation within a cloud are not completely understood. The relative proportion of condensation-formed, collision formed, and Bergeron-process-formed droplets depends very much on the individual cloud circumstances and can vary considerably. As a droplet is moved around a cloud it may freeze and thaw several times, leading to different types of precipitation (see Table 2.1).

Dewfall

The same process of condensation occurs in **dewfall**, only in this case the water vapour condenses into liquid water after coming into contact with a cold surface. In humid-temperate countries dew is a common occurrence in autumn when the air at night is still warm but vegetation and other surfaces have cooled to the point where water vapour coming into contact with them condenses onto the leaves and forms dew. Dew is not normally a major part of the hydrological cycle but is another form of precipitation.

PRECIPITATION DISTRIBUTION

The amount of precipitation falling over a location varies both spatially and temporally (with time).

The different influences on the precipitation can be divided into static and dynamic influences. Static influences are those such as altitude, aspect and slope; they do not vary between storm events.

Dynamic influences are those that do change and are by and large caused by variations in the weather. At the global scale the influences on precipitation distribution are mainly dynamic being caused by differing weather patterns, but there are static factors such as topography that can also cause major variations through a **rain shadow effect** (see case study on pp. 18–19). At the continental scale large differences in rainfall can be attributed to a mixture of static and dynamic factors. In Figure 2.1 the rainfall distribution across the USA shows marked variations. Although mountainous areas have a higher rainfall, and also act as a block to rainfall reaching the drier centre of the country, they do not provide the only explanation for the variations evident in Figure 2.1. The higher rainfall in the north-west states (Oregon and Washington) is linked to wetter cyclonic weather systems from the northern Pacific that do not reach down to southern California. Higher rainfall in Florida and other southern states is linked to the warm waters of the Caribbean sea. These are examples of dynamic influences as they vary between rainfall events.

At smaller scales the static factors are often more dominant, although it is not uncommon for quite large variations in rainfall across a small area caused by individual storm clouds to exist. As an example: on 3 July 2000 an intense rainfall event caused flooding in the village of Epping Green, Essex, UK.

Approximately 10 mm of rain fell within one hour, although there was no recorded rainfall in the village of Theydon Bois approximately 10 km to the south.

This large spatial difference in rainfall was caused by the scale of the weather system causing the storm – in this case a convective thunderstorm. Often these types of variation lessen in importance over a longer timescale so that the annual rainfall in

Epping Green and Theydon Bois is very similar, whereas the daily rainfall may differ considerably.

For the hydrologist, who is interested in rainfall at the small scale, the only way to try and characterize these dynamic variations is through having as many **rain gauges** as possible within a study area.

Static influences on precipitation distribution

It is easier for the hydrologist to account for static variables such as those discussed below.

Altitude

It has already been explained that temperature is a critical factor in controlling the amount of water vapour that can be held by air. The cooler the air is, the less water vapour can be held. As temperature decreases with altitude it is reasonable to assume that as an air parcel gains altitude it is more likely to release the water vapour and cause higher rainfall.

This is exactly what does happen and there is a strong correlation between altitude and rainfall: so-called *orographic precipitation*.

Aspect

The influence of aspect is less important than altitude but it may still play an important part in the distribution of precipitation throughout a catchment.

In the humid mid-latitudes (35° to 65° north or south of the equator) the predominant source of rainfall is through cyclonic weather systems arriving from the west. Slopes within a catchment those face eastwards will naturally be more sheltered from the rain than those facing westwards. The same principle applies everywhere: slopes with aspects facing away from the predominant weather patterns will receive less rainfall than their opposites.

Slope

The influence of slope is only relevant at a very small scale. Unfortunately the measurement of rainfall occurs at a very small scale (i.e. a rain gauge). The difference between a level rain gauge on a hillslope, compared to one parallel to the slope, may be significant.

It is possible to calculate this difference if it is assumed that rain falls vertically – but of course rain does not always fall vertically. Consequently the effect of slope on rainfall measurements is normally ignored.

Rain shadow effect

Where there is a large and high land mass it is common to find the rainfall considerably higher on one side than the other. This is through a combination of altitude, slope, aspect and dynamic weather direction influences and can occur at many different scales (see Case Study below).

Forest rainfall partitioning

Once rain falls onto a vegetation canopy it effectively partitions the water into separate modes of movement: **throughfall**, **stemflow** and **interception loss**.

MEASUREMENT

For hydrological analysis it is important to know how much precipitation has fallen and when this occurred. The usual expression of precipitation is as a vertical depth of liquid water. Rainfall is measured by millimetres or inches depth, rather than by volume such as litres or cubic metres. The measurement is the depth of water that would accumulate on the surface if all the rain remained where it had fallen (Shaw, 1994). Snowfall may also be expressed as a depth, although for hydrological purposes it is most usefully described in water equivalent depth (i.e. the depth of water that would be present if the snow melted). This is a recognition that snow takes up a greater volume (as much as 90 per cent more) for the same amount of liquid water.

There is a strong argument that can be made to say that there is no such thing as precipitation measurement at the catchment scale as it varies so tremendously over a small area. The logical endpoint to this argument is that all measurement techniques are in fact precipitation estimation techniques. For the sake of clarity in this text precipitation measurement techniques refer to the methods used to quantify the volume of water present, as opposed to estimation techniques where another variable is used as a surrogate for the water volume.

Rainfall measurement

The instrument for measuring rainfall is called a *rain gauge*. A rain gauge measures the volume of water that falls onto a horizontal surface delineated by the rain gauge rim (see Figure 2.5). The volume is converted into a rainfall depth through division by the rain gauge surface area. The design of a rain gauge is not as simple as it may seem at first glance as there are many errors and inaccuracies that need to be minimised or eliminated.

There is a considerable scientific literature studying the accuracy and errors involved in measuring rainfall. It needs to be borne in mind that a rain gauge represents a very small point measurement (or sample) from a much larger area that is covered by the rainfall. Any errors in measurement will be amplified hugely because the rain gauge collection area represents such a small sample size. Because of this amplification it is extremely important that the design of a rain gauge negates any errors and inaccuracies.

The four main sources of error in measuring rainfall that need consideration in designing a method for the accurate measurement of rainfall are:

1 Losses due to evaporation

2 Losses due to wetting of the gauge

3 Over-measurement due to splash from the surrounding area

4 Under-measurement due to turbulence around the gauge.

Evaporation losses

A rain gauge can be any collector of rainfall with a known collection area; however, it is important that any rainfall that does collect is not lost again through evaporation. In order to eliminate, or at least lessen this loss, rain gauges are funnel shaped.

In this way the rainfall is collected over a reasonably large area and then any water collected is passed through a narrow aperture to a collection tank underneath. Because the collection tank has a narrow top (i.e. the funnel mouth) there is very little interchange of air with the atmosphere above the gauge. As will be explained in Chapter 3, one of the necessary requirements for evaporation is the turbulent mixing of saturated air with drier air above. By restricting this turbulent transfer there is little evaporation that can take place. In addition to this, the water awaiting measurement is kept out of direct sunlight so that it will not be warmed; hence there is a low evaporation loss.

Wetting loss

As the water trickles down the funnel it is inevitable that some water will stay on the surface of the funnel and can be lost to evaporation or not measured in the collection tank. This is often referred to as a *wetting loss*. These losses will not be large but may be significant, particularly if the rain is falling as a series of small events on a warm day. In order to lessen this loss it is necessary to have steep sides on the funnel and to have a non-stick surface.

The standard UK Meteorological Office rain gauge is made of copper to create a non-stick surface, although many modern rain gauges are made of non-adhesive plastics.

Rain splash

The perfect rain gauge should measure the amount of rainfall that would have fallen on a surface if the gauge was not there. This suggests that the ideal situation for a rain gauge is flush with the surface.

A difficulty arises, however, as a surface-level gauge is likely to over-measure the catch due to rain landing adjacent to the gauge and splashing into it. If there was an equal amount of splash going out of the gauge then the problem might not be so severe, but the sloping sides of the funnel (to reduce evaporative losses) mean that there will be very little splash-out. In extreme situations it is even possible that the rain gauge could be flooded by water flowing over the surface or covered by snow. To overcome the splash, flooding and snow coverage problem the rain gauge can be raised up above the ground (Figure 2.5) or placed in the middle of a non-splash grid (see Figure 2.6).

Turbulence around a raised gauge

If a rain gauge is raised up above the ground (to reduce splash) another problem is created due to air turbulence around the gauge. The rain gauge presents an obstacle to the wind and the consequent aerodynamic interference leads to a reduced catch (see Figure 2.7). The amount of loss is dependent on both the wind speed and the raindrop diameter (Nesˇpor and Sevruk, 1999). At wind speeds of 20 km/hr (Beaufort scale 2) the loss could be up to 20 per cent, and in severe winds of 90 km/hr (Beaufort scale 8) up to 40 per cent (Bruce and Clark, 1980; Rodda and Smith, 1986). The higher a gauge is from the surface the greater the loss of accuracy. This creates a major problem for gauges in areas that receive large snowfalls as they need to be raised to avoid surface coverage.

One method of addressing these turbulence difficulties is through the fitting of a shield to the rain gauge (see Figure 2.8). A rain gauge shield can take many forms but is often a series of batons surrounding the gauge at its top height. The shield acts as a calming measure for wind around the gauge and has been shown to greatly improve rain gauge accuracy.

The optimum rain gauge design

There is no perfect rain gauge. The design of the best gauge for a site will be influenced by the individual conditions at the site (e.g. prevalence of snowfall, exposure, etc.). A rain gauge with a nonsplash surround, such as in Figure 2.6, can give very accurate measurement but it is prone to coverage by heavy snowfall so cannot always be used. The non-splash surround allows adjacent rainfall to pass through (negating splash) but acts as an extended soil surface for the wind, thereby eliminating the turbulence problem from raised gauges. This may be the closest that it is possible to get to measuring the amount of rainfall that would have fallen on a surface if the rain gauge were not there.

The standard UK Meteorological Office rain gauge has been adopted around the world (although not everywhere) as a compromise between the factors influencing rain gauge accuracy. It is a brass rimmed rain gauge of 5 inches (127 mm) diameter standing 1 foot (305 mm) above the ground. The lack of height above ground level is a reflection of the low incidence of snowfall in the UK; in countries such as Russia and Canada, where winter snowfall is the norm, gauges may be raised as high as 2 m above the surface. There is general recognition that the UK standard rain gauge is not the best design for hydrology, but it does represent a reasonable compromise. There is a strong argument to be made against changing its design. Any change in the measurement instrument would make an analysis of past rainfall patterns difficult due to the differing accuracy.

Forest rainfall measurement

The most common method of assessing the amount of canopy interception loss is to measure the precipitation above and below a canopy and assume that the difference is from interception. Stated in this way it sounds a relatively simple task but in reality it is fraught with difficulty and error.

Durocher (1990) provides a good example of the instrumentation necessary to measure canopy interception, in this case for a deciduous woodland plot.

MOVING FROM POINT MEASUREMENT TO SPATIALLY DISTRIBUTED ESTIMATION

The measurement techniques described here have all concentrated on measuring rainfall at a precise location (or at least over an extremely small area). In reality the hydrologist needs to know how much precipitation has fallen over a far larger area, usually a catchment. To move from point measurements to a spatially distributed estimation it is necessary to employ some form of spatial averaging. The spatial averaging must attempt to account for an uneven spread of rain gauges in the catchment and the various factors that we know influence rainfall distribution (e.g. altitude, aspect and slope).

A simple arithmetic mean will only work where a catchment is sampled by uniformly spaced rain gauges and where there is no diversity in topography.

If these conditions were ever truly met then it is unlikely that there would be more than one rain gauge sampling the area. Hence it is very rare to use a simple averaging technique.

There are different statistical techniques that address the spatial distribution issues, and with the growth in use of **Geographic Information Systems (GIS)** it is often a relatively trivial matter to do the calculation. As with any computational task it is important to have a good knowledge of how the technique works so that any shortcomings are fully understood. Three techniques are described here:

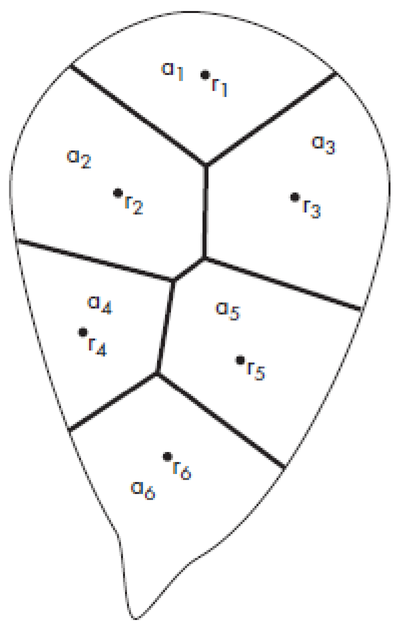

Figure 2.12 Thiessen's polygons for a series of rain gauges (r_i) within an imaginary catchment. The area of each polygon is denoted as a_i. Locations of rain gauges are indicated by bullet points.

Thiessen's polygons, the **hypsometric method** and the **isohyetal method**. These methods are explored further in a Case Study on p. 31.

Thiessen's polygons

Thiessen was an American engineer working around the start of the twentieth century who devised a simple method of overcoming an uneven distribution of rain gauges within a catchment (very much the norm). Essentially Thiessen's polygons attach a representative area to each rain gauge.

The size of the representative area (a polygon) is based on how close each gauge is to the others surrounding it.

Each polygon is drawn on a map; the boundaries of the polygons are equidistant from each gauge and drawn at a right angle (orthogonal) to an imaginary line between two gauges (see Figure 2.12). Once the polygons have been drawn the area of each polygon surrounding a rain gauge is found. The spatially averaged rainfall (*R*) is calculated using formula 2.1:

$$R = \sum_{i=1}^{n} \frac{r_i a_i}{A} \qquad (2.1)$$

where *ri* is the rainfall at gauge *i*, *ai* is the area of the polygon surrounding rain gauge *i*, and *A* is the total catchment area.

The **areal rainfall** value using Thiessen's polygons is a weighted mean, with the weighting being based upon the size of each representative area (polygon). This technique is only truly valid where the topography is uniform within each polygon so that it can be safely assumed that the rainfall distribution is uniform within the polygon. This would suggest that it can only work where the rain gauges are located initially with this technique in mind (i.e. *a priori*).

Hypsometric method

Since it is well known that rainfall is positively influenced by altitude (i.e. the higher the altitude the greater the rainfall) it is reasonable to assume that knowledge of the catchment elevation can be brought to bear on the spatially distributed rainfall estimation problem. The simplest indicator of the catchment elevation is the hypsometric (or hypsographic) curve. This is a graph showing the proportion of a catchment above or below a certain elevation. The values for the curve can be derived from maps using a planimeter or using a digital elevation model (DEM) in a GIS.

The hypsometric method of calculating spatially distributed rainfall then calculates a weighted average based on the proportion of the catchment between two elevations and the measured rainfall between those elevations (equation 2.2).

$$R = \sum_{j=1}^{m} r_j p_j \qquad (2.2)$$

where *rj* is the average rainfall between two contour intervals and *pj* is the proportion of the total catchment area between those contours (derived from the hypsometric curve). The *rj* value may be an average of several rain gauges where there is more than one at a certain contour interval. This is illustrated in Figure 2.13 where the shaded area (*a3*) has two gauges within it. In this case the *rj* value will be an average of *r4* and *r5*.

Intuitively this is producing representative areas for one or more gauges based on contours and spacing, rather than just on the latter as for Thiessen's polygons. There is an inherent assumption that elevation is the only topographic parameter affecting rainfall distribution (i.e. slope and aspect are ignored). It also assumes that the relationship between altitude and rainfall is linear, which is not always the case and warrants exploration before using this technique.

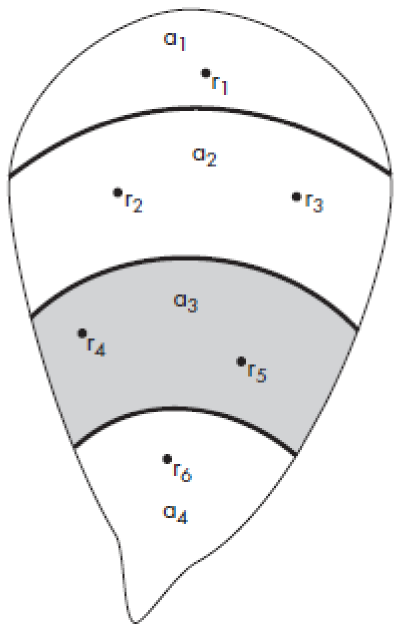

Figure 2.13 Calculation of areal rainfall using the hypsometric method. The shaded region is between two contours. In this case the rainfall is an average between the two gauges within the shaded area. Locations of rain gauges are indicated by bullet points.

Isohyetal and other smoothed surface techniques

Where there is a large number of gauges within a catchment the most obvious weighting to apply on a mean is based on measured rainfall distribution rather than on surrogate measures as described above. In this case a map of the catchment rainfall distribution can be drawn by interpolating between the rainfall values, creating a smoothed rainfall surface. The traditional isohyetal method involved drawing isohyets (lines of equal rainfall) on the map and calculating the area between each isohyet. The spatial average could then be calculated by equation 2.3

$$R = \sum_{i=1}^{n} \frac{r_i a_i}{A} \qquad (2.3)$$

where *ai* is the area between each isohyet and *ri* is the average rainfall between the isohyets. This technique is analogous to Figure 2.13, except in this case the contours will be of rainfall rather than elevation.

With the advent of GIS the interpolating and drawing of isohyets can be done relatively easily, although there are several different ways of carrying out the interpolation. The interpolation subdivides the catchment into small grid cells and then assigns a rainfall value for each grid cell (this is the smoothed rainfall surface). The simplest method of interpolation is to use a nearest neighbor analysis, where the assigned rainfall value for a grid square is proportional to the nearest rain gauges. A more complicated technique is to use **kriging**, where the interpolated value for each cell is derived with knowledge on how closely related the nearby gauges are to each other in terms of their co-variance. A fuller explanation of these techniques is provided by Bailey and Gatrell (1995).

An additional piece of information that can be gained from interpolated rainfall surfaces is the likely rainfall at a particular point within the catchment.

This may be more useful information than total rainfall over an area, particularly when needed for numerical simulation of hydrological processes.

The difficulty in moving from the point measurement to a spatially distributed average is a prime example of the problem of scale that besets hydrology. The scale of measurement (i.e. the rain gauge surface area) is far smaller than the catchment area that is frequently our concern. Is it feasible to simply scale up our measurement from point sources to the overall catchment? Or should there be some form of scaling factor to acknowledge the large discrepancy? There is no easy answer to these questions and they are the type of problem that research in hydrology will be investigating in the twenty-first century.

RAINFALL INTENSITY AND STORM DURATION

Water depth is not the only rainfall measure of interest in hydrology; also of importance is the **rainfall intensity** and **storm duration**. These are simple to obtain from an analysis of rainfall records using frequency analysis. The rainfall needs to be recorded at a short time interval (i.e. an hour or less) to provide meaningful data.

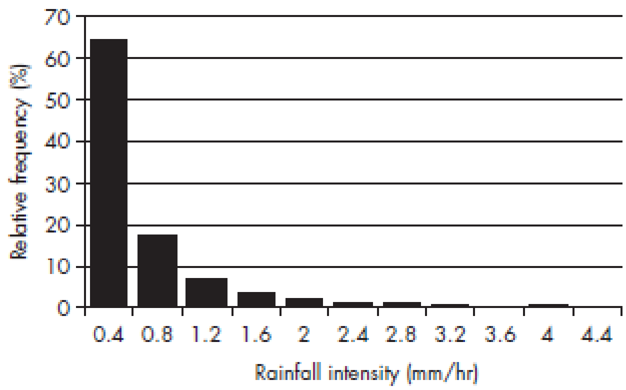

Figure 2.15 Rainfall intensity curve for Bradwell-on-Sea, Essex, UK. Data are hourly recorded rainfall from April 1995 to April 1997.

Figure 2.15 shows the rainfall intensity for a rain gauge at Bradwell-on-Sea, Essex, UK. It is evident from the diagram that the majority of rain falls at very low intensity: 0.4 mm per hour is considered as light rain. This may be misleading as the rain gauge recorded rainfall every hour and the small amount of rain may have fallen during a shorter period than an hour i.e. a higher intensity but lasting for less than an hour. During the period of measurement there were recorded rainfall intensities greater than 4.4 mm/hr (maximum 6.8 mm/hr) but they were so few as to not show up on the histogram scale used in Figure 2.15. This may be a reflection of only two years of records being analyzed, which introduces an extremely important concept in hydrology: the **frequency–magnitude** relationship.

With rainfall (and runoff – see Chapters 5 and 6) the larger the rainfall event the less frequent we would expect it to be. This is not a linear relationship; as illustrated in Figure 2.15 the curve declines in a non-linear fashion. If we think of the relative frequency as a probability then we can say that the chances of having a low rainfall event are very high: a low magnitude–high frequency event. Conversely the chances of having rainfall intensity greater than 5 mm/hr are very low (but not impossible): a high magnitude–low frequency event.

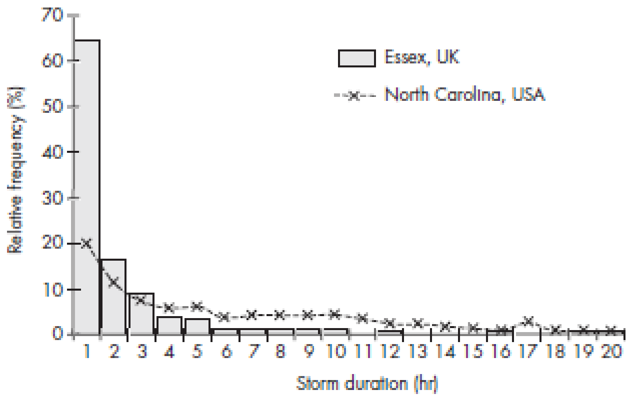

Figure 2.16 Storm duration curves. The bars are for the same data set as Figure 2.12 and the broken line for Ahoskie, North Carolina.

Source: Ahoskie data are redrawn from Wanielista (1990)

In Figure 2.16 the storm duration records for two different sites are compared. The Bradwell-on-Sea site has the majority of its rain events lasting one hour or less. In contrast the Ahoskie site has only 20 per cent of its storms lasting one hour or less but many more than Bradwell-on-Sea that last four hours or more. When the UK site rainfall and intensity curves are looked at together (i.e. Figures 2.15 and 2.16) it can be stated that Bradwell-on-Sea experiences a predominance of low intensity, short duration rainfall events and very few long duration, high intensity storms. This type of information is extremely useful to a hydrologist investigating the likely runoff response that might be expected for the rainfall regime.

Case study

RAINFALL DISTRIBUTION IN A SMALL STUDY CATCHMENT

It is well known that large variations in rainfall occur over quite a small spatial scale. Despite this, there are not many studies that have looked at this problem in detail. One study that has investigated spatial variability in rainfall was carried out in the Plynlimon research catchments in mid-Wales (Clarke *et al*., 1973). In setting up a hydrological monitoring network in the Wye and Severn catchments thirty-eight rain gauges were installed to try and characterise the rainfall variation. The rainfall network had eighteen rain gauges in the Severn catchment (total area 8.7 km2) and twenty gauges in the Wye (10.55 km2).

The monthly data for a period between April 1971 and March 1973 were analyzed to calculate areal average rainfall using contrasting methods.

The results from this can be seen in Figure 2.14. The most startling feature of Figure 2.14 is the lack of difference in calculated values and that they follow no regular pattern. At times the arithmetic mean is greater than the others while in other months it is less. When the total rainfall for the two-year period is looked at, the Thiessen's calculation is 0.3 per cent less than the arithmetic mean, while the isohyetal method is 0.4 per cent less.

When the data were analysed to see how many rain gauges would be required to characterise the rainfall distribution fully it was found that the number varied with the time period of rainfall and the season being measured. When monthly data were looked at there was more variability in winter rainfall than summer. For both winter and summer it showed that anything less than five rain gauges (for the Wye) increased the variance markedly.

A more detailed statistical analysis of hourly mean rainfall showed a far greater number of gauges were required. Four gauges would give accuracy in areal estimate of around 50 per cent, while a 90 per cent accuracy would require 100 gauges (Clarke *et al*., 1973: 62).

The conclusions that can be drawn from the study of Clarke *et al*. (1973) are of great concern to hydrology. It would appear that even for a small catchment a large number of rain gauges are required to try and estimate rainfall values properly. This confirms the statement made at the start of this chapter: although rainfall is relatively straightforward to measure it is notoriously difficult to measure accurately and, to compound the problem, is also extremely variable within a catchment area.

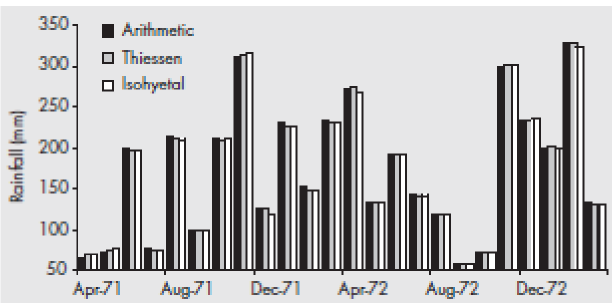

Figure 2.14 Areal mean rainfall (monthly) for the Wye catchment, calculated using three different methods.

Source: Data from Clarke *et al*. (1973)

EVAPORATION

Evaporation is the transferal of liquid water into a gaseous state and its diffusion into the atmosphere.

In order for this to occur there must be liquid water present and available energy from the sun or atmosphere. The importance of evaporation within the hydrological cycle depends very much on the amount of water present and the available energy, two factors determined by a region's climate. During winter months in humid-temperate climates evaporation may be a minor component of the hydrological cycle as there is very little available energy to drive the evaporative process. This alters during summer when there is abundant available energy and evaporation has the potential to become a major part of the water balance. The potential may be limited by the availability of liquid water during the dry months. This can be seen in extremely hot, arid climates where there is often plenty of available energy to drive evaporation but very little water to be evaporated. As a consequence the actual amount of evaporation is small.

It is the presence or lack of water at the surface that provides the major semantic distinction in definitions of the evaporative process. **Open water evaporation** (often denoted as *Eo*) is the evaporation that occurs above a body of water such as a lake, stream or the oceans. Figure 1.6 shows that at the global scale this is the largest source of evaporation, in particular from the oceans.

Potential evaporation

(*PE*) is that which occurs over the land's surface, or would occur if the water supply were unrestricted. This occurs when a soil is wet and what evaporation is able to happen occurs without a lack of water supply. **Actual evaporation** (*Et*) is that which actually occurs (i.e. if there is not much available water it will be less than potential). When conditions are very wet (e.g. during a rainfall event) *Et* will equal *PE*, otherwise it will be less than *PE*.

In hydrology we are most interested in *Eo* and *Et* but normally require *PE* to calculate the *Et* value.

All of these definitions have been concerned with 'evaporation over a surface'. In hydrology the surface is either water (river, lake, ponds, etc.) or the land.

The evaporation above a land surface occurs in two ways – either as actual evaporation from the soil matrix or **transpiration** from plants. The combination of these two is often referred to as **evapotranspiration**, although the term *actual evaporation* is essentially the same (hence the *t* subscript in *Et*). Transpiration from a plant occurs as part of photosynthesis and respiration. The rate of transpiration is controlled by the opening or closing of stomata in the leaf. Transpiration can be ascertained at the individual plant level by instruments measuring the flow of water up the trunk or stem of a plant. Different species of plants transpire at different rates but the fundamental controls are the available water in the soil, the plant's ability to transfer water from the soil to its leaves and the ability of the atmosphere to absorb the transpired water.

Evaporation is sometimes erroneously described as the only loss within the water balance equation.

The water balance equation is a mathematical description of the hydrological cycle and by definition there are no losses and gains within this cycle. What is meant by 'loss' is that evaporation is lost from the earth's surface, where hydrologists are mostly concerned with the water being. To a meteorologist, concerned with the atmosphere, evaporation can be seen as a gain. Evaporation although not a loss, can be viewed as the opposite of precipitation, particularly in the case of dewfall, a form of precipitation. In this case the **dewfall** (or negative evaporation) is a gain to the earth's surface.

MEASUREMENT OF EVAPORATION

In the previous chapter there has been much emphasis on the difficulties of measuring precipitation due to its inherent variability. All these difficulties also apply to the measurement of evaporation, but they pale into insignificance when you consider that now we are dealing with measuring the rate at which a gas (water vapour) moves away from a surface. Concentrations of gases in the atmosphere are difficult to measure, and certainly there is no gauge that we can use to measure total amounts in the same way that we can for precipitation.

In each of the process chapters in this book there is an attempt to distinguish between measurement and estimation techniques. In the case of evaporation this distinction becomes extremely blurred.

In reality almost all the techniques used to find an evaporation rate are estimates, but some are closer to true measurement than others. In this section each technique will include a sub-section on how close to 'true measurement' it is.

Direct micro-meteorological measurement

There are three main methods used to measure evaporation directly: the eddy fluctuation (or correlation), aerodynamic profile, and **Bowen ratio** methods. These are all micro-meteorological measurement techniques and details on them can be found elsewhere (e.g. Oke, 1987). An important point to remember about them all is that they are attempting to measure how much water is being evaporated above a surface, a very difficult task.

The eddy fluctuation method measures the water vapour above a surface in conjunction with a vertical wind speed and temperature profiles. These have to be measured at extremely short timescales (e.g. microseconds) to account for eddies in vertical wind motion. Consequently, extremely detailed micrometeorological instrumentation is required with all instruments having a rapid response time. In recent years this has become possible with hot wire **anemometers** and extremely fine thermistor heads for thermometers. One difficulty is that you are necessarily measuring over a very small surface area and it may be difficult to scale up to something of interest to catchment-scale hydrology.

The aerodynamic profile (or turbulent transfer) method is based on a detailed knowledge of the energy balance over a surface. The fundamental idea is that by calculating the amount of energy available for evaporation the actual evaporation rate can be determined. The measurements required are changes in temperature and humidity giving vertical humidity gradients. To use this method it must be assumed that the atmosphere is neutral and stable, two conditions that are not always applicable.

The Bowen ratio method is similar to the aerodynamic profile method but does not assume as much about the atmospheric conditions. The Bowen ratio is the ratio of sensible heat to latent heat and requires detailed measurement of net radiation, soil heat flux, temperature and humidity gradient above a surface. These measurements need to be averaged over a 30-minute period to allow the inherent assumptions to apply.

All of these micro-meteorological approaches to measuring evaporation use sophisticated instruments that are difficult to leave in the open for long periods of time. In addition to this they are restricted in their spatial scope (i.e. they only measure over a small area). With these difficulties it is not surprising that they tend to be used at the very small scale, mostly to calibrate estimation techniques (see pp. 46–52). They are accurate in the assessment of an evaporation rate, hence their use as a standard for the calibration of estimation techniques. The real problem for hydrology is that it is not a robust method that can be relied on for long periods of time.

Indirect measurement (water balance techniques)

Evaporation pans

The most common method for the measurement of evaporation is using an **evaporation pan** (see Figure 3.3). This is a large pan of water with a water depth measuring instrument or weighing device underneath that allows you to record how much water is lost through evaporation over a time period.

This technique is actually a manipulation of the water balance equation, hence the terminology used here of a water balance technique. An evaporation pan is constructed from impervious material and the water level is maintained below the top so that no seepage or leakage occurs. This eliminates runoff (*Q* term) from the water balance. Therefore it can be assumed that any change in storage is related to either evaporative loss or precipitation gain. This means that the water balance equation can be rearranged as shown in equation 3.2.

$$E = \Delta S - P \qquad (3.2)$$

If there is a precipitation gauge immediately adjacent to the evaporation pan then the *P* term can be accounted for, leaving only the change in storage(ΔS) to be measured as either a weight loss or a drop in water depth. At a standard meteorological station the evaporation is measured daily as the change in water depth. For a finer temporal resolution (e.g. hourly) there are load cell instruments available which measure and record the weight at regular intervals.

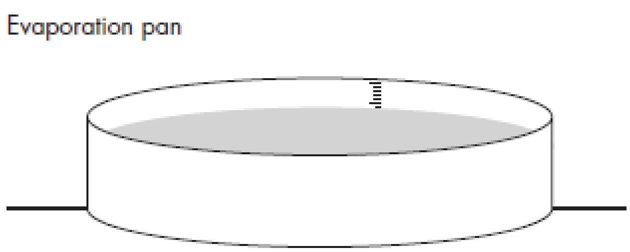

Figure 3.3 An evaporation pan. This sits above the surface (to lessen rain splash) and has either an instrument to record water depth or a continuous weighing device, to measure changes in volume.

An evaporation pan is filled with water, hence you are measuring *Eo*, the open water evaporation.

Although this is useful, there are severe problems with using this value as an indicator of actual evaporation (*Et*) in a catchment. The first problem is that *Eo* will normally be considerably higher than *Et* because the majority of evaporation in a catchment will be occurring over a land surface where the available water is contained within soil and may be limited. This will lead to a large overestimation of the actual evaporation. This factor is well known and consequently evaporation pans are rarely used in catchment water balance studies, although they are useful for estimating water losses from lakes and reservoirs.

There are also problems with evaporation pans that make them problematic even for open water evaporation estimates. A standard evaporation pan, called a Class A evaporation pan, is 1,207 mm in diameter and 254 mm deep. The size of the pan makes them prone to the ‘edge effect’. As warm air blows across a body of water it absorbs any water vapour evaporated from the surface. Numerous studies have shown that the evaporation rate is far higher near the edge of the water than towards the centre where the air is able to absorb less water vapour (this also applies to land surfaces). The small size of an evaporation pan means that the whole pan is effectively an ‘edge’ and will have a higher evaporation rate than a much larger body of water. A second, smaller, problem with evaporation pans is that the sides, and the water inside, will absorb radiation and warm up quicker than in a much larger lake, providing an extra energy source and greater evaporation rate.

To overcome the edge effect, empirical (i.e. derived from measurement) coefficients can be used which link the evaporation pan estimates to larger water body estimates. Doorenbos and Pruitt (1975) give estimates for these coefficients that require extra information on upwind fetch distance, wind run and relative humidity at the pan (Goudie *et al.*, 1994). Grismer *et al*. (2002) provide empirical relationships linking pan evaporation measurements to potential evapotranspiration, i.e. from a vegetated surface not open water evaporation.

Lysimeters

A **lysimeter** takes the same approach to measurement as the evaporation pan, the fundamental difference being that a lysimeter is filled with soil and vegetation as opposed to water (see Figure 3.4).

This difference is important, as *Et* rather than *Eo* is being indirectly measured. A lysimeter can also be made to blend in with the surrounding land cover, lessening the edge effect described for an evaporation pan.

There are many versions of lysimeters in use, but all use some variation of the water balance equation to estimate what the evaporation loss has been. One major difference from an evaporation pan is that a

lysimeter allows percolation through the bottom, although the amount is measured. Percolation is necessary so that the lysimeter mimics as closely as possible the soil surrounding it; without any it would fill up with water. In the same manner as an evaporation pan it is necessary to measure the precipitation input immediately adjacent to the lysimeter. Assuming that the only runoff (*Q*) is through percolation, the water balance equation for a lysimeter i shown in equation 3.3.

$$E = \Delta S - P - Q \qquad (3.3)$$

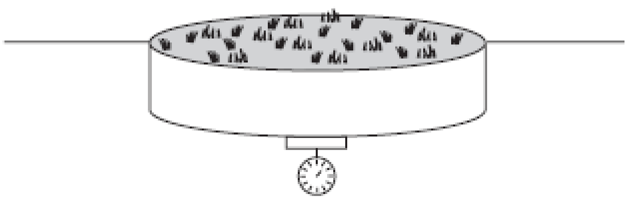

Figure 3.4 A weighing lysimeter sitting flush with the surface. The cylinder is filled with soil and vegetation similar to the surroundings.

A lysimeter faces similar problems to a rain gauge in that it is attempting to measure the evaporation that would be lost from a surface if the lysimeter were not there. The difference from a rain gauge is that what is contained in the lysimeter should closely match the surrounding plants and soil.

Although it is never possible to recreate the soil and plants within a lysimeter perfectly, a close approximation can be made and this represents the best efforts possible to measure evaporation. Although

lysimeters potentially suffer from the same edge effect as evaporation pans, the ability to match the surrounding vegetation means there is much less of an edge effect.

A *weighing lysimeter* has a weighing device underneath that allows any change in storage to be monitored. This can be an extremely sophisticated device (e.g. Campbell and Murray, 1990; Yang *et al*.,

2000), where percolation is measured continuously using the same mechanism for a tipping-bucket rain gauge, weight changes are recorded continuously using a hydraulic pressure gauge, and precipitation is measured simultaneously. A variation on this is to have a series of small weighing lysimeters (such as small buckets) that can be removed and weighed individually every day to provide a record of weight loss. At the same time as weighing, the amount of percolation needs to be recorded. This is a very cheap way of estimating evaporation loss for a study using low technology.

Without any instrument to weigh the lysimeter (this is sometimes referred to as a *percolation gauge*) it must be assumed that the change in soil moisture over a period is zero and therefore evaporation equals rainfall minus runoff. This may be a reasonable assumption over a long time period such as a year where the soil storage will be approximately the same between two winters. An example of this type of lysimeter was the work of Law who investigated the effect that trees had on the water balance at Stocks Reservoir in Lancashire, UK (Law, 1956; see Case Study on p. 42).

A well-planned and executed lysimeter study probably provides the best information on evaporation that a hydrologist could find. However, it must be remembered that it is not evaporation that is being measured in a lysimeter – it is almost everything else in the water balance equation, with an assumption being made that whatever is left must be caused by evaporation. One result of this is that any errors in measurement of precipitation and/or percolation will transfer and possibly magnify into errors of evaporation measurement.

STORAGE

The water balance equation, explained in Chapter 1, contains a storage term (S). Within the hydrological cycle there are several areas where water can be considered to be stored, most notably soil moisture, groundwater, snow and ice and, to a lesser extent, lakes and reservoirs. It is tempting to see stored water as static, but in reality there is considerable movement involved. The use of a storage term is explained in Figure 4.1 where it can be seen that there is an inflow, an outflow and a movement of water between the two. The inflow and outflow do not have to be equal over a time period; if not, then there has been a *change in storage* (ΔS). The critical point is that at all times there is some water stored, even if it is not the same water throughout a measurement period.

This definition of stored water is not perfect as it could include rivers as stored water in addition to groundwater, etc. The distinction is often made on the basis of flow rates (i.e. how quickly the water moves while in storage). There is no critical limit to say when a deep, slow river becomes a lake, and likewise there is no definition of how slow the flow has to be before becoming stored water. It relies on an intuitive judgement that slow flow rates constitute stored water.

The importance of stored water is highlighted by the fact that it is by far the largest amount of freshwater in or around planet earth (see Table 1.2, p. 6). The majority of this is either in snow and ice (particularly the polar ice caps) or groundwater.

For many parts of the world groundwater is a major source of drinking water, so knowledge of amounts and replenishment rates is important for water resource management. By definition, stored water is slow moving so it is particularly prone to contamination by pollutants. The three 'Ds' of water pollution control (dilution, dispersion and degradation; see Chapter 7) all occur at slow rates in stored water, making pollution management a particular problem.

When this is combined with the use of these waters for potable supply, an understanding of the hydrological processes occurring in stored water is very important.

In this chapter two major stores of water are described: water beneath the earth's surface in the unsaturated and saturated zones; and snow and ice.

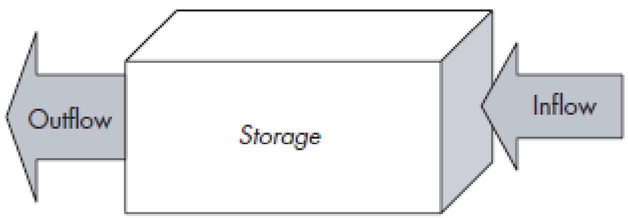

Figure 4.1 Illustration of the storage term used in the water balance equation.

Infiltration rate

The rate at which water infiltrates the soil is not constant. Generally, water initially infiltrates at a faster rate and slows down with time (see Figure 4.3). When the infiltration rate slows down to a steady level (where the curve flattens off in Figure 4.3) the **infiltration capacity** has been reached.

This is the rate of infiltration when the soil is fully saturated. The terminology of infiltration capacity is misleading as it suggests a capacity value rather than a rate. In fact infiltration capacity is the infiltration rate when the water is filled to capacity with water.

Infiltration capacity is sometimes referred to as the saturated hydraulic conductivity. This is not absolutely true as the measurement is dependent on the amount of water that may be ponded on the surface creating a high hydraulic head. Saturated hydraulic conductivity should be independent of this ponded head of water. There are conditions when infiltration capacity equals saturated hydraulic conductivity, but this is not always the case.

The curve shown in Figure 4.3 is sometimes called the Philip curve, after Philip (1957) who built upon the pioneering work of Horton (1933) and provided sound theory for the infiltration of water.

The main force driving infiltration is gravity, but it may not be the only force. When soil is very dry it exerts a pulling force (soil suction; see p. 6) that will suck the infiltrating water towards the drier area. With both of these forces the infiltrating water moves down through the soil profile in a wetting front. The wetting front is three-dimensional, as the water moves outwards as well as vertically down.

The shape of the curve in Figure 4.3 is related to the speed at which the wetting front is moving. It slows down the further it gets away from the surface as it takes longer for the water at the surface to feed the front (and as the front increases in size).

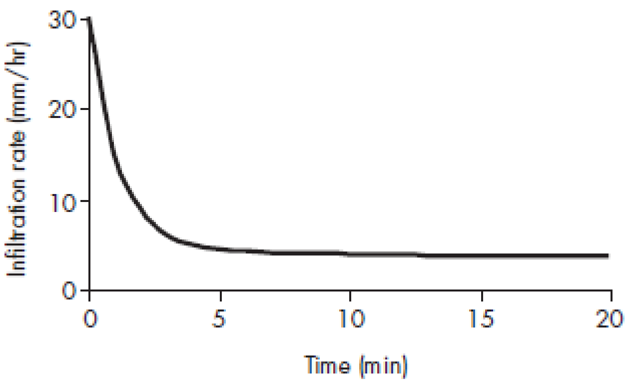

Figure 4.3 Typical infiltration curve.

Measurement of infiltration rate

Infiltration rate is measured by recording the rate at which water enters the soil. There are numerous methods available to do this, the simplest being a ring **infiltrometer** (see Figure 4.12). A solid ring is pushed into the ground and a pond of water sits on the soil (within the ring). This pond of water is kept at a steady level by a reservoir held above the ring. Recordings of the level of water in the reservoir (with time) give a record of the infiltration rate. To turn the infiltration volume into an infiltration depth the volume of water needs to be divided by the cross-sectional area of the ring.

A simple ring infiltrometer provides a measure of the ponded infiltration rate, but there are several associated problems. The first is that by using a single ring a large amount of water may escape around the sides of the ring, giving higher readings than would be obtained from a completely saturated surface. To overcome this a double ring infiltrometer is sometimes used.

With this, a second wider ring is placed around the first and filled with water so that the area surrounding the measured ring is continually wet. The second problem is that ponded infiltration is a relatively rare event across a catchment. It is more common for rainfall to infiltrate directly without causing a pond to form on the surface. To overcome this a rainfall simulator may be used to provide the infiltrating water.

Module – II

Runoff: Computation, factors affecting runoff, Design flood: Rational formula, Empirical formulae, Stream flow: Discharge measuring structures, approximate average slope method, area-velocity method, stage-discharge relationship.

Hydrograph; Concept, its components, Unit hydrograph: use and its limitations, derivation of UH from simple and complex storms, S-hydrograph, derivation of UH from S-hydrograph. Synthetic unit hydrograph: Snyder's approach, introduction to instantaneous unit hydrograph (IUH).

RUNOFF

The amount of water within a river or stream is of great interest to hydrologists. It represents the end product of all the other processes in the hydrological cycle and is where the largest amount of effort has gone into analysis of historical records. The methods of analysis are covered in Chapter 6; this chapter deals with the mechanisms that lead to water entering the stream: the runoff mechanisms. *Runoff* is a loose term that covers the movement of water to a channelised stream, after it has reached the ground as precipitation. The movement can occur either on or below the surface and at differing velocities. Once the water reaches a stream it moves towards the oceans in a channelised form, the process referred to as **streamflow** or **riverflow**.

Streamflow is expressed as **discharge**: the volume of water over a defined time period. The SI units for discharge are m3/s (*cumecs*). A continuous record of streamflow is called a **hydrograph** (see Figure 5.1).

Although we think of this as continuous measurement it is normally either an averaged flow over a time period or a series of samples (e.g. hourly records).

In Figure 5.1 there are a series of peaks between periods of steady, much lower flows. The hydrograph peaks are referred to as **peakflow**, **stormflow** or even **quickflow**. They are the water in the stream during and immediately after a significant rainfall event. The steady periods between peaks are referred to as **baseflow** or sometimes slowflow (NB this is different from **low flow**; see Chapter 6). The shape of a hydrograph, and in particular the shape of the stormflow peak, is influenced by the storm characteristics (e.g. rainfall intensity and duration) and many physical characteristics of the upstream catchment. In terms of catchment characteristics the largest influence is exerted by catchment size, but other factors include slope angles, shape of catchment, soil type, vegetation type and percentage cover, degree of urbanisation and the antecedent soil moisture.

Figure 5.2 shows the shape of a storm hydrograph in detail. There are several important hydrological terms that can be seen in this diagram. The **rising limb** of the hydrograph is the initial steep part leading up to the highest or peakflow value. The water contributing to this part of the hydrograph is from *channel precipitation* (i.e. rain that falls directly onto the channel) and rapid runoff mechanisms.

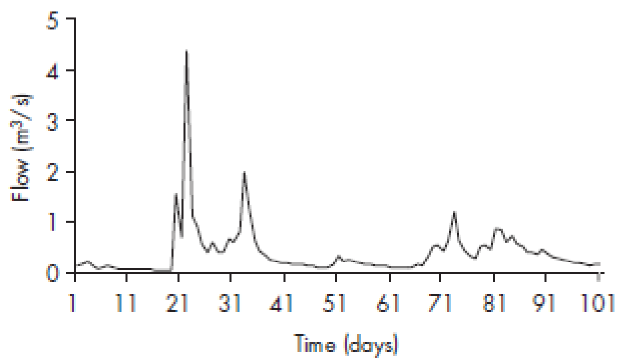

Figure 5.1 A typical hydrograph, taken from the river Wye, Wales for a 100-day period during the autumn of 1995. The values plotted against time are mean daily flow in cumecs.

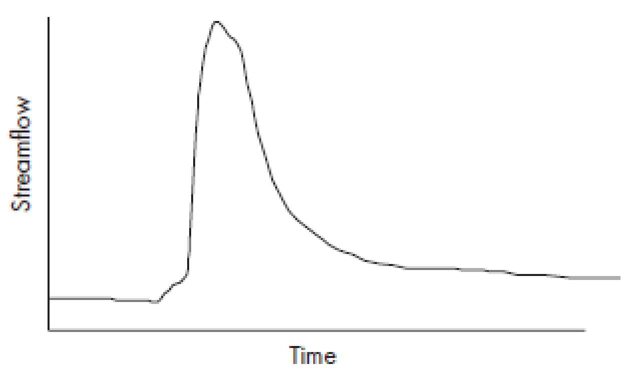

Figure 5.2 Demonstration storm hydrograph.

Some texts claim that channel precipitation shows up as a preliminary blip before the main rising limb.

In reality this is very rarely observed, a factor of the complicated nature of storm runoff processes. The **recession limb** of the hydrograph is after the peak and is characterised by a long, slow decrease in streamflow until the baseflow is reached again. The recession limb is attenuated by two factors: storm water arriving at the mouth of a catchment from the furthest parts, and the arrival of water that has moved as underground flow at a slower rate than the streamflow.

Exactly how water moves from precipitation reaching the ground surface to channelised streamflow is one of the most intriguing hydrological questions, and one that cannot be answered easily.

Much research effort in the past hundred years has gone into understanding runoff mechanisms; considerable advances have been made, but there are still many unanswered questions. The following section describes how it is believed runoff occurs, but there are many different scales at which these mechanisms are evident and they do not occur everywhere.

RUNOFF MECHANISMS

Figure 5.3 is an attempt to represent the different runoff processes that can be observed at the hillslope scale. **Overland flow** (*Qo*) is the water which runs across the surface of the land before reaching the stream. In the subsurface, throughflow (*Qt*) (some authors refer to this as **lateral flow**) occurs in the shallow subsurface, predominantly, although not always, in the unsaturated zone. Groundwater flow (*QG*) is in the deeper saturated zone. All of these are runoff mechanisms that contribute to streamflow.

The relative importance of each is dependent on the catchment under study and the rainfall characteristics during a storm.

Overland flow

Some of the earliest research work on how overland flow occurs was undertaken by Robert Horton

(1875–1945). In a classic paper from 1933, Horton hypothesised that overland flow occurred when the rainfall rate was higher than the infiltration rate of a soil. Horton went on to suggest that under these circumstances the excess rainfall collected on the surface before travelling towards the stream as a thin sheet of water moving across the surface. Under this hypothesis it is the infiltration rate of a soil that acts as a controlling barrier or partitioning device.

Where the infiltration capacity of a soil is low, overland flow occurs readily. This type of overland flow is referred to as **infiltration excess overland flow** or **Hortonian overland flow** although as Beven (2004) points out, Horton himself referred to it as 'rainfall excess'.

Horton's ideas were extremely important in hydrology as they represented the first serious attempt to understand the processes of storm runoff that lead to a storm hydrograph.

Unfortunately, when people started to measure infiltration capacities of soils they invariably found that they were far higher than most normal rainfall rates. This is illustrated in Table 5.1 where some typical infiltration capacities and rainfall rates are shown. Other measurements confirm high infiltration capacities for soils, e.g. Selby (1970) reports infiltration capacities of between 60 and 600 mm/hour on short grazed pasture on yellow-brown pumice soils in the central North Island of New Zealand. The values were higher for ungrazed grass and under trees and are generally higher than the measured rainfall intensities (Selby, 1970).

In addition to the infiltration capacity information, it is extremely rare to find a thin sheet of water moving over the surface during a storm event.

It was observations such as those by Hursh (1944) and others that led to a general revision of Horton's hypothesis. Betson (1964) proposed the idea that within a catchment there are only limited areas that contribute overland flow to a storm hydrograph.

This is referred to as the **partial areas concept**. Betson did not challenge the role of infiltration excess overland flow as the primary source of stormflow, but did challenge the idea of overland flow occurring as a thin sheet of water throughout a catchment.

Hewlett and Hibbert (1967) were the first to suggest that there might be another mechanism of overland flow occurring. This was particularly basedon the observations from the eastern USA: that during a storm it was common to find all the rainfall infiltrating a soil. Hewlett and Hibbert (1967) hypothesised that during a rainfall event all the water infiltrated the surface. This hypothesis was confirmed by a comprehensive field study by Dunne and Black (1970).

Through a mixture of infiltration and throughflow the water table would rise until in some places it reached the surface. At this stage overland flow occurs as a mixture of return flow (i.e. water that has been beneath the ground but returns to the surface) and rainfall falling on saturated areas. This type of overland flow is referred to as **saturated overland flow**. Hewlett and Hibbert (1967) suggested that the water table was closest to the surface, and therefore likely to rise to the surface quickest, adjacent to stream channels and at the base of slopes. Their ideas on stormflow were that the areas contributing water to the hydrograph peaks were the saturated zones, and that these vary from storm to storm. In effect the saturated areas immediately adjacent to the stream act as extended channel networks. This is referred to as the **variable source areas concept**. This goes a step beyond the ideas of Betson (1964) as the catchment has a partial areas response but the response area is dynamic; i.e. variable in space and time.

So who was right: Horton, or Hewlett and Hibbert? The answer is that both were. Table 5.2 provides a summary of the ideas for storm runoff generation described here. It is now accepted that saturated overland flow (Hewlett and Hibbert) is the dominant overland flow mechanism in humid, midlatitude areas. It is also accepted that the variable source areas concept is the most valid description of stormflow processes. However, where the infiltration capacity of a soil is low or the rainfall rates are high, Hortonian overland flow does occur. In Table 5.1 it can be seen that there are times when rainfall intensities will exceed infiltration rates under natural circumstances. In arid and semi-arid zones it is not uncommon to find extremely high rainfall rates (fed by convective storms) that can lead to infiltration excess overland flow and rapid flood events; this is called **flash flooding**.

Examples of low infiltration rates can be found with compacted soils (e.g. from vehicle movements in an agricultural field), on roads and paved areas, on heavily crusted soils and what are referred to as **hydrophobic soils**.

Basher and Ross (2001) reported infiltration capacities of 400 mm/hour in market gardens in the North Island of New Zealand and that these rates increased during the growing season to as high as 900 mm/hour. However, Basher and Ross (2001) also showed a decline in infiltration capacity to as low as 0.5 mm/hour in wheel tracks at the same site.

Hydrophobic soils have a peculiar ability to swell rapidly on contact with water, which can create an impermeable barrier at the soil surface to infiltrating water, leading to Hortonian overland flow. The cause of hydrophobicity in soils has been linked into several factors including the presence of microrhizal fungi and swelling clays such as allophane (Doerr *et al*., 2007). Hydrophobicity is a temporary soil property; continued contact with water will increase the infiltration rate. For example Clothier *et al*. (2000) showed how a yellow brown earth/loam changed from an initial infiltration capacity of 2 mm/hour to 14 mm/hour as the soil hydrophobicity breaks down.

In Hewlett and Hibbert’s (1967) original hypothesis it was suggested that contributing saturated areas would be immediately adjacent to stream channels. Subsequent work by the likes of Dunne and Black (1970), Anderson and Burt (1978) and others has identified other areas in a catchment prone to inducing saturated overland flow. These include hillslope hollows, slope concavities (in section) and where there is a thinning of the soil overlying an impermeable base. In these situations any throughflow is likely to return to the surface as the volume of soil receiving it is not large enough for the amount of water entering it. This can be commonly observed in the field where wet and boggy areas can be found at the base of slopes and at valley heads (hillslope hollows).

Subsurface flow

Under the variable source areas concept there are places within a catchment that contribute overland flow to the storm hydrograph. When we total up the amount of water found in a storm hydrograph it is difficult to believe that it has all come from overland flow, especially when this is confined to a relatively small part of the catchment (i.e. variable source areas concept). The manner in which the recession limb of a hydrograph attenuates the stormflow suggests that it may be derived from a slower movement of water: subsurface flow. In addition to this, tracer studies looking at where the water has been before entering the stream as stormflow have found that a large amount of the storm hydrograph consists of ‘old water’ (e.g. Martinec *et al*., 1974; Fritz *et al*., 1976). This old water has been sitting in the soil, or as fully saturated groundwater, for a considerable length of time and yet enters the stream during a storm event. There have been several theories put forward to try and explain these findings, almost all involving throughflow and groundwater.

Throughflow is a general term used to describe the movement of water through the unsaturated zone; normally this is the soil matrix. Once water infiltrates the soil surface it continues to move, either through the soil matrix or along preferential flow paths (referred to as lateral or preferential flow). The rate of soil water movement through a saturated soil matrix is described by Darcy’s law (see Chapter 4) and the Richards approximation of Darcy’s law when below saturation. Under normal, vertical, infiltration conditions the hydraulic gradient has a value of –1 and the saturated hydraulic conductivity is the infiltration capacity. Once the soil is saturated the movement of water is not only vertical. With a sloping water table on a hillsope, water moves down slope. However, the movement of water through a saturated soil matrix is not rapid, e.g. Kelliher and

Scotter (1992) report a *Ksat* value of 13 mm/hour for a fine sandy loam. In order for throughflow to contribute to storm runoff there must be another mechanism (other than matrix flow) operating.

One of the first theories put forward concerning the contribution of throughflow to a storm hydrograph was by Horton and Hawkins (1965) (this Horton was a different person from the proposer of Hortonian overland flow). They proposed the mechanism of *translatory* or *piston flow* to explain the rapid movement of water from the subsurface to the stream. They suggested that as water enters the top of a soil column it displaces the water at the bottom of the column (i.e. old water), and the displaced water enters the stream. The analogy is drawn to a piston where pressure at the top of the piston chamber leads to a release of pressure at the bottom.

The release of water to the stream can be modeled as a pressure wave rather than tracking individual particles of water. Piston flow has been observed in laboratory experiments with soil columns (e.g. Germann and Beven, 1981).

At first glance the simple piston analogy seems unlike a real-life situation since a hillslope is not bounded by impermeable sides in the same way as a piston chamber. However the theory is not as farfetched as it may seem, as the addition of rainfall infiltrating across a complete hillslope is analogous to pressure being applied from above and in this case the boundaries are upslope (i.e. gravity) and the bedrock below. Brammer and McDonnell (1996) suggest that this may be a mechanism for the rapid movement of water along the bedrock and soil interface on the steep catchment of Maimai in New Zealand. In this case it is the hydraulic gradient created by an addition of water to the bottom of the soil column, already close to saturated, that forces water along the base where hydraulic conductivities are higher.

Ward (1984) draws the analogy of a thatched roof to describe the contribution of subsurface flow to a stream (based on the ideas of Zaslavsky and Sinai, 1981). When straw is placed on a sloping roof it is very efficient at moving water to the bottom of the roof (the guttering being analogous to a stream) without visible overland flow. This is due to the preferential flow direction along, rather than between, sloping straws. Measurements of hillslope soil properties do show a higher hydraulic conductivity in the downslope rather than vertical direction. This would account for a movement of water downslope as through flow, but it is still bound up in the soil matrix and reasonably slow.

There is considerable debate on the role of **macropores** in the rapid movement of water through the soil matrix. Macropores are larger pores within a soil matrix, typically with a diameter greater than 3 mm. They may be caused by soils cracking, worms burrowing or other biotic activities. The main interest in them from a hydrologic point of view is that they provide a rapid conduit for the movement of water through a soil. The main area of contention concerning macropores is whether they form continuous networks allowing rapid movement of water down a slope or not. There have been studies suggesting macropores as a major mechanism contributing water to stormflow (e.g. Mosley, 1979, 1982; Wilson *et al*., 1990), but it is difficult to detect whether these are from small areas on a hillslope or continuous throughout. Jones (1981) and Tanaka

(1992) summarise the role of pipe networks (a form of continuous macropores) in hillslope hydrology.

Where found, pipe networks have considerable effect on the subsurface hydrology but they are not a common occurrence in the field situation.

The role of macropores in runoff generation is unclear. Although they are capable of allowing rapid movement of water towards a stream channel there is little evidence of networks of macropores moving large quantities of water in a continuous fashion.

Where macropores are known to have a significant role is in the rapid movement of water to the saturated layer (e.g. Heppell *et al*., 1999) which may in turn lead to piston flow (McGlynn *et al*., 2002).

Groundwater contribution to storm flow

Another possible explanation for the presence of old water in a storm hydrograph is that it comes from the saturated zone (groundwater) rather than from through flow. This is contrary to conventional hydrological wisdom which suggests that groundwater contributes to baseflow but not to the stormflow component of a hydrograph. Although a groundwater contribution to stormflow had been suggested before, it was not until Sklash and Farvolden (1979) provided a theoretical mechanism for this to occur that the idea was seriously considered.

They proposed the capillary fringe hypothesis to explain the groundwater ridge, a rise in the water table immediately adjacent to a stream (as observed by Ragan, 1968). Sklash and Farvolden (1979) suggested that the addition of a small amount of infiltrating rainfall to the zone immediately adjacent to a stream causes the soil water to move from an unsaturated state (i.e. under tension) to a saturated state (i.e. a positive pore pressure expelling water). As explained in Chapter 4, the relationship between soil water content and soil water tension is non-linear. The addition of a small amount of water can cause a rapid change in soil moisture status from unsaturated to saturated. This provides the groundwater ridge which not only provides the early increased impetus for the displacement of the groundwater already in a discharge position, but it also results in an increase in the size of the groundwater discharge area which is essential in producing large groundwater contributions to the stream. (Sklash and Farvolden, 1979: 65)

An important point to stress from the capillary fringe hypothesis is that the groundwater ridge is developing well before any through flow may have been received from the contributing hillslope areas. These ideas confirm the variable source areas concept and provide a mechanism for a significant old water contribution to storm hydrographs.

Field studies such as that by McDonnell (1990) have observed groundwater ridging to a limited extent, although it is not an easy task as often the instrument response time is too slow to detect the rapid change in pore pressure properly.

Summary of storm runoff mechanisms

The mechanisms that lead to a storm hydrograph are extremely complex and still not fully understood.

Although this would appear to be a major failing in a science that is concerned with the movement of water over and beneath the surface, it is also an acknowledgement of the extreme diversity found in nature. In general there is a reasonable understanding of possible storm runoff mechanisms but it is not possible to apply this universally.

In some field situations the role of through flow and piston flow are important, in others not; likewise for groundwater contributions, overland flow and pipe flow. The challenge for modern hydrology is to identify quickly the dominant mechanisms for a particular hillslope or catchment so that the understanding of the hydrological processes in that situation can be used to aid management of the catchment.

The processes of storm runoff generation described here are mostly observable at the hillslope scale. At the catchment scale (and particularly for large river basins) the timing of peak flow (and consequently the shape of the storm hydrograph) is influenced more by the channel drainage network and the precipitation characteristics of a storm than by the mechanisms of runoff. This is a good example of the problem of scale described in Chapter 1. At the small hillslope scale storm runoff generation mechanisms are important, but they become considerably less so at the much larger catchment scale.

Baseflow

In sharp contrast to the storm runoff debate, there is general consensus that the major source of baseflow is groundwater – and to a lesser extent through flow. This is water that has infiltrated the soil surface and moved towards the saturated zone.

Once in the saturated zone it moves downslope, often towards a stream. A stream or lake is often thought to occur where the regional water table intersects the surface, although this may not always be the case. In Chapter 4 the relationship between groundwater and streamflow has been explained (see Figure 4.9). However in general it can be said that baseflow is provided by the slow seepage of water from groundwater into streams. This will not necessarily be visible (e.g. springs) but can occur over a length of streambank and bed and is only detectable through repeated measurement of streamflow down a reach.

Channel flow

Once water reaches the stream it will flow through a channel network to the main river. The controls over the rate of flow of water in a channel are to do with the volume of water present, the gradient of the channel, and the resistance to flow experienced at the channel bed. This relationship is described in uniform flow formulae such as the Chezy and Manning equations (see p. 92). The resistance to flow is governed by the character of the bed surface. Boulders and vegetation will create a large amount of friction, slowing the water down as it passes over the bed.

In many areas of the world the channel network is highly variable in time and space. Small channels may be ephemeral and in arid regions will frequently only flow during flood events. The resistance to flow under these circumstances is complicated by the infiltration that will be occurring at the water front and bed surface. The first flush of water will infiltrate at a much higher rate as it fills the available pore space in the soil/rock at the bed surface. This will remove water from the stream and also slow the water front down as it creates a greater friction surface. Under a continual flow regime the infiltration from the stream to ground will depend on the hydraulic gradient and the infiltration capacity.

MEASURING STREAMFLOW

The techniques and research into the measurement of streamflow are referred to as **hydrometry**.

Streamflow measurement can be subdivided into two important subsections: instantaneous and continuous techniques.

Instantaneous streamflow measurement

Velocity–area method Streamflow or discharge is a volume of water per unit of time. The standard units for measurement of discharge are m3/s (cubic metres per second or *cumecs*). If we rewrite the units of discharge we can think of them as a water velocity (m/s) passing through a cross-sectional area (m2). Therefore:

$$m^3/s = m/s \times m^2 \qquad (5.1)$$

The **velocity–area method** measures the stream velocity, the stream cross-sectional area and multiplies the two together. In practice this is carried out by dividing the stream into small sections and measuring the velocity of flow going through each cross-sectional area and applying equation 5.2.

$$Q = v_1 a_1 + v_2 a_2 + \dots v_i a_i \qquad (5.2)$$

where Q is the streamflow or discharge (m3/s), v is the velocity measured in each trapezoidal cross-sectional area (see Figure 5.6), and a is the area of the trapezoid (usually estimated as the average of two depths divided by the width between).

The number of cross-sectional areas that are used in a discharge measurement depend upon the width and smoothness of stream bed. If the bed is particularly rough it is necessary to use more cross-sectional areas so that the estimates are as close to reality as possible (note the discrepancy between the broken and solid lines in Figure 5.6).

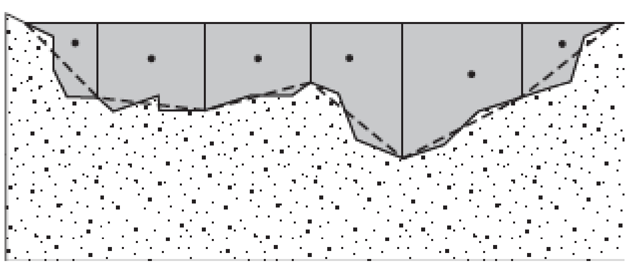

Figure 5.6 The velocity–area method of streamflow measurement. The black circles indicate the position of velocity readings. Dashed lines represent the triangular or trapezoidal cross-sectional area through which the velocity is measured.

The water velocity measurement is usually taken with a flow meter (Figure 5.7). This is a form of propeller inserted into the stream which records the number of propeller turns with time. This reading can be easily converted into a stream velocity using the calibration equation supplied with the flow meter.

In the velocity–area method it is necessary to assume that the velocity measurement is representative of all the velocities throughout the cross-sectional area. It is not normally possible to take multiple measurements so an allowance has to be made for the fact that the water travels faster along the surface than nearer the stream bed. This difference in velocity is due to friction exerted on the water as it passes over the stream bed, slowing it down. As a general rule of thumb the sampling depth should be 60 per cent of the stream depth– that is, in a stream that is 1 m deep the sampling point should be 0.6 m below the surface or 0.4 m above the bed. In a deep river it is good practice to take two measurements (one at 20 per cent and the other at 80 per cent of depth) and average the two.

Where there is no velocity meter available it may be possible to make a very rough estimate of stream velocity using a float in the stream (i.e. the time it takes to cover a measured distance). When using this method allowance must be made for the fact that the float is travelling on the surface of the stream at a faster rate than water closer to the stream bed.

The velocity–area method is an effective technique for measuring streamflow in small rivers, but its reliability is heavily dependent on the sampling strategy. The technique is also less reliable in small, turbid streams with a rough bed (e.g. mountain streams). Under these circumstances other streamflow estimation techniques such as **dilution gauging** may be more applicable (see streamflow estimation section).

Continuous streamflow measurement

The methods of instantaneous streamflow measurement described above only allow a single measurement to be taken at a location. Although this can be repeated at a future date it requires a continuous measurement technique to give the data for a hydrograph.

There are three different techniques that can be used for this method: stage discharge relationships, flumes and weirs, and ultrasonic flow gauging.

Stage vs discharge relationship

River **stage** is another term for the water level or height. Where multiple discharge measurements have been taken (i.e. repeat measurements using velocity–area method) it is possible to draw a relationship between river stage and discharge: the so-called **rating curve**. An example of a rating curve is shown in Figure 5.8. This has the advantage of allowing continuous measurement of river stage (a relatively simple task) that can then be equated to the actual discharge. The stage discharge relationship is derived through a series of velocity–area measurements at a particular site while at the same time recording the stage with a stilling well (see Figure 5.9). As can be seen in Figure 5.8, the rating curve is non-linear, a reflection of the river bank profile. As the river fills up between banks it takes a greater volume of water to cause a change in stage than at low levels.

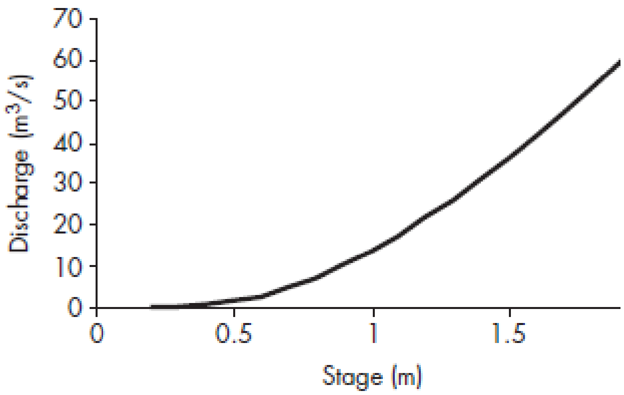

Figure 5.8 A rating curve for the river North Esk in Scotland based on stage (height) and discharge measurements from 1963–1990.

An accurate stage vs discharge relationship is dependent on frequent and accurate measurement of river discharge, and a static river bed profile. If the river bed profile changes (e.g. during a large flood event it may get scoured out or new sediment deposited), the stage vs discharge relationship will change and the historic relationship will no longer be valid. This assumption of a static river bed profile can sometimes be problematic, leading to the installation of a concrete structure (e.g. flume or weir) to maintain stability.

One of the difficulties with the stage vs discharge relationship is that the requirement of frequent measurements of river discharge lead to many measurements taken during periods of low and medium flow but very few during flood events. This is for the double reason that: floods are infrequent and unlikely to be measured under a regular monitoring programme; and the danger of streamflow gauging during a flood event. The lack of data at the extreme end of the stage vs discharge curve may lead to difficulties in interpreting data during peak flows.

The error involved in estimating peak discharge from a measured stage vs discharge relationship will be much higher at the high flow end of curve.

When interpreting data derived from the stage discharge vs relationship it is important that the hydrologist bears in mind that it is stream stage that is being measured and from this stream discharge is inferred (i.e. it is not a direct measurement of stream discharge).

Flumes and weirs

Flumes and weirs utilize the stage–discharge relationship described above but go a step further towards providing a continuous record of river discharge.

If we think of stream discharge as consisting of a river velocity flowing through a cross-sectional area (as in the velocity profile method) then it is possible to isolate both of these terms separately.

This is what flumes and weirs, or *stream gauging structures*, attempt to do. The first part to isolate is the stream velocity.

The way to do this is to slow a stream down (or, in some rare cases, speed a stream up) so that it flows with constant velocity through a known cross-sectional area. The critical point is that in designing a flume or weir the river flows at the same velocity (or at least a known velocity) through the gauging structure irrespective of how high the river level is. Although this seems counter-intuitive (rivers normally flow faster during flood events) it is achievable if there is an area prior to the gauging structure that slows the river down: a stilling pond.

The second part of using a gauging structure is to isolate a cross-sectional area. To achieve this rigid structure is imposed upon the stream so that it always flows through a known cross-sectional area.

In this way a simple measure of stream height through the gauging structure will give the cross-sectional area. Stream height is normally derived through a stilling well, as described in Figure 5.9, except in this case there is a regular cross-sectional area.

Once the velocity and cross-sectional area are kept fixed the rating curve can be derived through a mixture of experiment and hydraulic theory. These relationships are normally power equations dependent on the shape of cross-sectional area used in the flume or weir. There is an international standard for manufacture and maintenance of weirs (ISO 1438) that sets out theoretical ratings curves for different types of structures. The general formula for a V notch weir is shown in equation 5.3.

$$Q = 0.53.\sqrt{2g}.C.\tan\left(\frac{\theta}{2}\right).h^{2.5} \qquad (5.3)$$

where *Q* is discharge (m3/s); *g* is the acceleration due to gravity (9.81 m/s2); C is coefficient of discharge (see Figure 5.10); is the angle of V-notch (°); *h* is the height of water or stage (m). The coefficient of discharge can be estimated from figure 5.10 for a certain angle of V-notch. For a 90° V-notch the coefficient of discharge is 0.578 and the rating equation becomes:

$$Q = 1.366h^{2.5} \qquad (5.4)$$

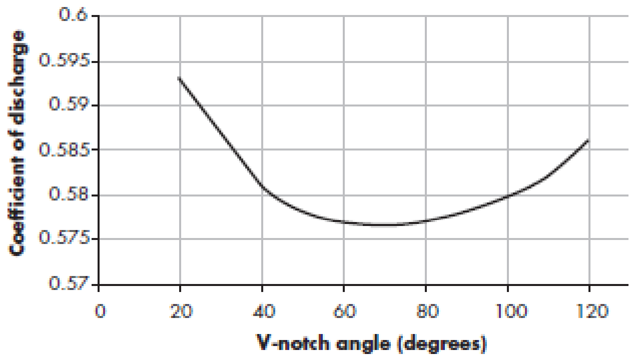

Figure 5.10 Coefficient of discharge for V-notch weirs (ISO 1438).

There is a similar type of equation for rectangular weirs, based on the width of the rectangular exit and another version of the coefficient of discharge relationship.

The shape of cross-sectional area is an important consideration in the design of flumes and weirs. The shape of permanent structure that the river flows through is determined by the flow regime of the river and the requirements for the streamflow data.

A common shape used is based on the V-structure (see Figure 5.11). The reason for this is that when river levels are low, a small change in river flow will correspond to a significant change in stage (measured in the stilling well). This sensitivity to low flows makes data from this type of flume or weir particularly suitable for studying low flow hydrology. It is important that under high flow conditions the river does not overtop the flume or weir structure. The V shape is convenient for this also because as discharge increases the cross-sectional area flowed through increases in a non-linear fashion.

The angle of the V-notch will vary depending on the size of stream being measured and the sensitivity required (90° and 120° V-notch weirs are both commonly used).

One of the difficulties in maintaining gauging structures is that by slowing the river down in a stilling pond any sediment being carried by the water may be deposited (see **Hjulstrom curve** in Chapter 7), which in time will fill the stilling pond and lessen its usefulness. Because of this the stilling pond needs to be dredged regularly, particularly in a high energy environment such as mountain streams. To overcome this difficulty there is a design of trapezoidal flume that speeds the stream up rather than slows it down (see Figure 5.12). The stream is forced to go down a steep section immediately prior to the gauging structure. In this way any sediment is flushed out of the weir, removing the need for regular dredging. This is really only possible for small streams as the power of large rivers at high velocities would place enormous strains on the gauging structure.

The difference between flumes and weirs

Although flumes and weirs perform the same function– measuring stream discharge in a continuous fashion – they are not the same. In a weir the water is forced to drop over a structure (the weir – Figure 5.11) in the fashion of a small waterfall. In a flume (Figure 5.12) the water passes through the structure without having a waterfall at the end.

Ultrasonic flow gauging

Recent technological developments have led to the introduction of a method of measuring stream discharge using the properties of sound wave propagation in water. The method actually measures water velocity, but where the stream bed cross-sectional area is known (and constant) the instrumentation can be left in place and combined with measurements of stage to provide a continuous measurement of river discharge. There are two types of **ultrasonic flow gauges** that work in slightly different ways.

The first method measures the time taken for an ultrasonic wave emitted from a transmitter to reach a receiver on the other side of a river. The faster the water speed, the greater the deflection of the wave path and the longer it will take to cross the river.

Sound travels at approximately 1,500 m/s in water (dependent on water purity and depth) so the instrumentation used in this type of flow gauging needs to be extremely precise and be able to measure in nanoseconds. This type of flow gauging can be installed as a permanent device but needs a width of river greater than 5 m and becomes unreliable with a high level of suspended solids.

The second method utilises the Doppler effect to measure the speed of particles being carried by the stream. At an extremely simple level this is a measurement of the wavelength of ultrasonic waves that bounce off suspended particles – the faster the particle the shorter the wavelength. This type of instrument works in small streams (less than 5 m width) and requires some suspended matter.

FLOODS

The term *flood* is difficult to define except in the most general of terms. In a river a flood is normally considered to be an inundation of land adjacent to a river caused by a period of abnormally large discharge or encroachment by the sea (see cover photograph, Figure 5.13, and Plate 6), but even this definition is fraught with inaccuracy. Flooding may occur from sources other than rivers (e.g. the sea and lakes), and 'abnormal' is difficult to pin down, particularly within a timeframe. Floods come to our attention through the amount of damage that they cause and for this reason they are often rated on a cost basis rather than on hydrological criteria.

Hydrological and monetary assessments of flooding often differ markedly because the economic valuation is highly dependent on location. If the area of land inundated by a flooding river is in an expensive region with large infrastructure then the cost will be considerably higher than, say, for agricultural land. Two examples of large-scale floods during the 1990s illustrate this point. In 1998 floods in China caused an estimated US$20 billion of damage with over 15 million people being displaced and 3,000 lives lost (Smith, 2001). This flood was on a similar scale to one that occurred in the same region during 1954. A much larger flood (in a hydrological sense) in the Mississippi and Missouri rivers during 1993 resulted in a similar economic valuation of loss (US$15–20 billion) but only 48 lives were lost (USCE, 1996). The flood was the highest in the hydrological record and had an average recurrence interval of between 100 and 500 years (USCE, 1996). The difference in lives lost and relative economic loss (for size of flood) is a reflection of the differing response to the flood in two economically contrasting countries.

As described in Chapter 2 for precipitation, flooding is another example where the *frequency–magnitude relationship* is important. Small flood events happen relatively frequently whereas the really large floods occur rarely but cause the most damage. The methods for interpreting river flows that may be used for flood assessment are discussed in Chapter 6. They provide some form of objective flood size assessment, but their value is highly dependent on the amount of data available.

Floods are a frequently occurring event around the world. At the time of preparing of this chapter (June and July 2007) there were eleven large flood events reported in the news media (see Table 5.4).

These floods were caused by varying amounts of rainfall, and occurred in different seasons of the year but all caused significant damage and in many cases loss of lives. There are numerous reasons why a river will flood and they almost always relate back to the processes found within the hydrological cycle. The main cause of river floods is when there is too much rainfall for the river to cope with. Other, more special causes of floods are individual events like dam bursts, **jökulhlaups** (ice-dam bursts) or snow melt (see pp. 72–75).

Influences on flood size

The extent and size of the flood can often be related to other contributing factors that increase the effect of high rainfall. Some of these factors are described here but all relate back to concepts introduced in earlier chapters detailing the processes found within the hydrological cycle. Flooding provides an excellent example of the importance of scale, introduced in Chapter 1. Many of the factors discussed here have an influence at the small scale (e.g. hillslopes or small research catchments of less than 10 km2) but not at the larger overall river catchment scale.

Antecedent soil moisture

The largest influence on the size of a flood, apart from the amount and intensity of rainfall, is the wetness of the soil immediately prior to the rainfall or snow melt occurring. As described on p. 59, the amount of infiltration into a soil and subsequent storm runoff are highly dependent on the degree of saturation in the soil. Almost all major flood events are heavily influenced by the amount of rainfall that has occurred prior to the actual flood causing rainfall.

Deforestation

The effect of trees on runoff has already been described, particularly with respect to water resources. There is also considerable evidence that a large vegetation cover, such as forest, decreases the severity of flooding. There are several reasons for this. The first has already been described, in that trees provide an intercepting layer for rainfall and therefore slow down the rate at which the water reaches the surface. This will lessen the amount of rainfall available for soil moisture and therefore the antecedent soil moisture may be lower under forest than for an adjoining pasture (NB this is not always the case, it is dependent on the time of year). The second factor is that forests often have a high organic matter in the upper soil layers which, as any gardener will tell you, is able to absorb more water.

Again this lessens the amount of overland flow, although it may increase the amount of throughflow. Finally, the infiltration rates under forest soils are often higher, leading again to less saturation excess overland flow.

The removal of forests from a catchment area will increase the propensity for a river to flood and also increase the severity of a flood event. Conversely the planting of forests on a catchment area will decrease the frequency and magnitude of flood events. Fahey and Jackson (1997) show that after conversion of native tussock grassland to exotic pine plantations a catchment in New Zealand showed a decrease in the mean flood peaks of 55–65 per cent. Although data of this type look alarming they are almost always taken from measurements at the small research catchment scale. At the larger scale the influence of deforestation is much harder to detect (see Chapter 8).

Urbanisation

Urban areas have a greater extent of impervious surfaces than in most natural landforms. Consequently the amount of infiltration excess (Hortonian) overland flow is high. In addition to this, urban areas are often designed to have a rapid drainage system, taking the overland flow away from its source. This network of gutters and drains frequently leads directly to a river drainage system, delivering more flood water in a faster time. Where extensive urbanization of a catchment occurs; flood frequency and magnitude increases. Cherkauer (1975) shows a massive increase in flood magnitude for an urban catchment in Wisconsin, USA when compared to a similar rural catchment (see pp. 169–170). Urbanisation is another influence on flooding that is most noticeable at the small scale. This is mostly because the actual percentage area covered by impermeable urban areas in a larger river catchment is still very small in relation to the amount of permeable nonmodified surfaces.

River channel alterations

Geomorphologists traditionally view a natural river channel as being in equilibrium with the river flowing within it. This does not mean that a natural river channel never floods, but rather that the channel has adjusted in shape in response to the normal discharge expected to flow through it. When the river channel is altered in some way it can have a detrimental effect on the flood characteristics for the river. In particular, **channelisation** using rigid structures can increase flood risk. Ironically, channelisation is often carried out to lessen flood risk in a particular area. This is frequently achieved, but in doing so water is passed on downstream at a faster rate than normal, increasing the flood risk further downstream. If there is a natural floodplain further downstream this may not be a problem, but if there is not, downstream riparian zones will be at greater risk.

Land drainage

It is common practice in many regions of the world to increase agricultural production through the drainage of ‘swamp’ areas. During the seventeenth and eighteenth centuries huge areas of the fenlands of East Anglia in England were drained and now are highly productive cereal and horticultural areas. The drainage of these regions provides for rapid removal of any surplus water, i.e. not needed by plants.

Drained land will be drier than might be expected naturally, and therefore less storm runoff might be assumed. This is true in small rainfall events but the rapid removal of water through subsurface and surface drainage leads to flood peaks in the river drainage system where normally the water would have been slower to leave the land surface.

So, although the drainage of land leads to an overall drying out of the affected area it can also lead to increased flooding through rapid drainage. Again this is scale-related, as described further in Chapter 8.

Climate change

In recent years any flooding event has led to a clamour of calls to explain the event in terms of climatic change. This is not easy to do as climate is naturally so variable. What can be said though is that river channels slowly adjust to changes in flow regime which may in turn be influenced by changes in climate. Many studies have suggested that future climate change will involve greater extremes of weather (IPCC, 2007), including more high intensity rainfall events. This is likely to lead to an increase in flooding, particularly while a channel adjusts to the differing flow regime (if it is allowed to).

STREAMFLOW ANALYSIS AND MODELLING

One of the most important tasks in hydrology is to analyze streamflow data. These data are continuous records of discharge, frequently measured in permanent structures such as flumes and weirs (see Chapter 5). Analysis of these data provides us with three important features:

- Description of a flow regime
- Potential for comparison between rivers, and
- Prediction of possible future river flows.

There are well-established techniques available to achieve these, although they are not universally applied in the same manner. This chapter sets out three important methods of analyzing streamflow: hydrograph analysis, flow duration curves and frequency analysis. These three techniques are explained with reference to worked examples, all drawn from the same data set. The use of data from within the same study area is important for comparison between the techniques.

RATIONAL METHOD

Rational method proposes a simple rainfall-runoff model to estimate the peak discharge for a drainage basin, not the entire direct runoff hydrograph. The method assumes that when a steady rainfall rate occurs, then runoff rate increases until the entire watershed is contributing to the outlet. The peak discharge at the outlet is approached when the rainfall duration starts exceeding the time of concentration, which is the time for the most remote point to the outlet for the total drainage system. The rational model is then expressed by

$$Q_p = CiA \qquad (2.8)$$

where Q_p= peak runoff rate; C= runoff coefficient, dimensionless; i=intensity of rainfall; and A= area of the watershed. All watershed losses are incorporated into the runoff coefficient C (0<=C<=1); therefore i in the rational equation is not excess rainfall. Recommended values of C for other surfaces are given in Table (2.2). For composite areas

$$\bar{C} = \frac{\sum C_i A_i}{\sum A_i} \qquad (2.9)$$

where C¯ =average (or composite) runoff coefficient; and Ci= runoff coefficient of Ai. The intensity of rainfall i is obtained from intensity-duration-frequency (IDF) curves (Figure 2.13), where rainfall is assumed to be uniformly distributed over the catchment area1.

In the IDF curves, the return period is the time interval for which an event will occur once on average. A storm sewer is generally designed for a return period of 5, 10, or 25 years. Because a drainage basin can frequently receive storm events higher than that, streets and other open channel systems must be designed to carry flood flow that is in excess of the storm sewer capacity.

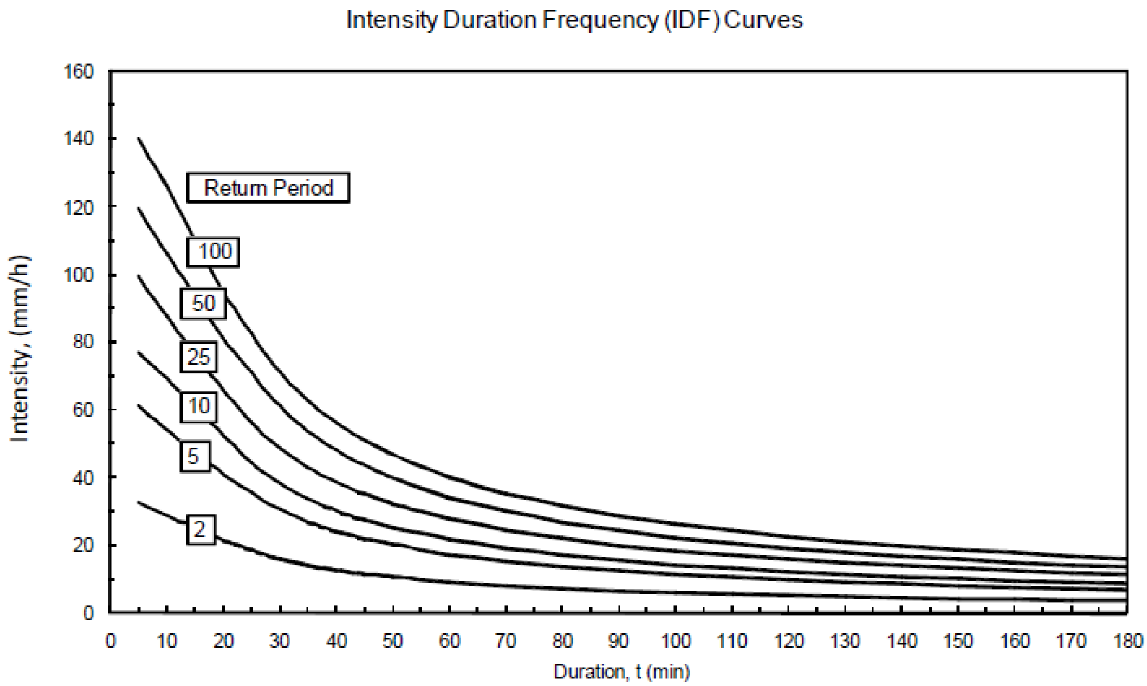

Figure (2.13). Intensity duration frequency curves for Kuwait. The values are the standards used by the Ministry of Public Works in Kuwait (source: plotted from numerical data provided by MPW).

FLOW DURATION CURVES

An understanding of how much water is flowing down a river is fundamental to hydrology. Of particular interest for both flood and low flow hydrology is the question of how representative a certain flow is. This can be addressed by looking at the frequency of daily flows and some statistics that can be derived from the frequency analysis. The culmination of the frequency analysis is a **flow duration curve** which is described below.

Flow duration curves are concerned with the amount of time a certain flow is exceeded. The data most commonly used are daily mean flows: the average flow for each day (note well that this is not the same as a mean daily flow, which is the average of a series of daily flows). To derive a flow duration curve the daily mean flow data are required for a long period of time, in excess of five years. A worked example is provided here, using twenty-six years of data for the upper reaches of the river Wye in mid-Wales, UK (see pp. 108–109).

Flow duration curve: step 1

A table is derived that has the frequency, cumulative frequency (frequency divided by the total number of observations) and percentage cumulative frequency.

The percentage cumulative frequency is assumed to equal the percentage of time that the flow is exceeded. While carrying out the frequency analysis it is important that a small class (or bin) interval is used; too large an interval and information will be lost from the flow duration curve. The method for choosing the best class interval is essentially through trial and error. As a general rule you should aim not to have more than around 10 per cent of your values within a single class interval. If you have more than this you start to lose precision in plotting. As shown in the worked example, it is not essential that the same interval is used throughout.

Flow duration curve: step 2

The actual flow duration curve is created by plotting the percentage cumulative frequency on the x-axis against the mid-point of the class interval on the yaxis.

Where two flow duration curves are presented on the same axes they need to be standardised for direct comparison. To do this the values on the y-axis (mid-point of class interval) are divided by the average flow for the record length. This makes the y-axis a percentage of the average flow (see Figure 6.6).

The presentation of a flow duration curve may be improved by either plotting on a special type of graph paper or transforming the data. The type of graph paper often used has the x-axis transformed in the form of a known distribution such as the Gumbel or Log Pearson. A natural log transformation of the flow values (y-axis) achieves a similar effect, although this is not necessarily standard practice.

Interpreting a flow duration curve

The shape of a flow duration curve can tell a lot about the hydrological regime of a catchment. In Figure 6.6 two flow duration curves of contrasting shape are shown. With the dotted line there is a large difference between the highest and lowest flow values, whereas for the solid line there is far less variation. This tells us that the catchment shown by the solid line never has particularly low flows or particularly high flows. This type of hydrological response is found in limestone or chalk catchments where there is a high baseflow in the summer (groundwater derived) and high infiltration rates during storm events. In contrast the catchment shown with a solid line has far more variation.

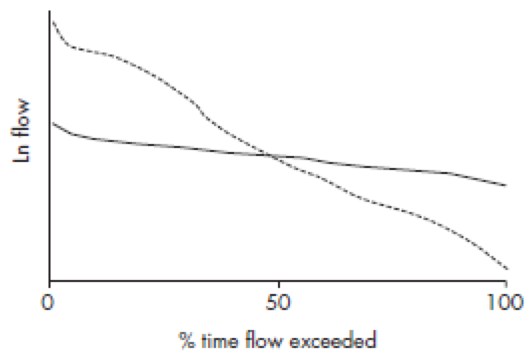

Figure 6.6 Two contrasting flow duration curves. The dotted line has a high variability in flow (similar to a small upland catchment) compared to the solid line (similar to a catchment with a high baseflow).

During dry periods it has a very low flow, but responds to rainfall events with a high flow. This is characteristic of impermeable upland catchments or streamflow in dry land areas.

Statistics derived from a flow duration curve

The interpretation of flow duration curve shape discussed so far is essentially subjective. In order to introduce some objectivity there are statistics derived from the curve; the three most important ones are:

- The flow value that is exceeded 95 per cent of the time (*Q*95)- A useful statistic for low flow analysis.
- The flow value that is exceeded 50 per cent of the time (*Q*50). This is the median flow value.
- The flow value that is exceeded 10 per cent of the time (*Q*10)- A useful statistic for analysis of high flows and flooding.

HYDROGRAPH ANALYSIS

Hydrograph, which is a graph showing changes in water discharge over a period of time for a given point on a channel or conduit, is commonly used in many engineering applications. For example, hydrologists depend on hydrographs to provide peak flow rates so that hydraulic structures can be designed to accommodate the flow safely. Also, the area under the curve of a hydrograph can provide the volume of water, which allows analysis of reservoir sizes, storage tanks, detention ponds, and other facilities that deal with volumes of runoff passing the point of interest during a time period. The shape of the hydrograph changes according to the properties of the basin and meteorological conditions such as rainfall pattern, hydrologic losses, groundwater flow, and surface characteristics including size, roughness, shape, slope and imperviousness. A specific hydrograph can be estimated by following two main steps. Initially, continuous records of water flow versus depth are taken at the channel location. In small channels, these records are estimated by using a device such as a weir. In large channels, placing a flow measuring device in the channel becomes practically difficult, and instead flow rates are measured by using a current meter. Then, a rating curve is plotted, which is a relation between flow rate and stage1.Indirect methods to estimate the water discharge for a given depth is accomplished by employing open channel flow equations such as that of Manning or Chezy. Next, the relation between flow rate and depth is used to transform continuous records of stage versus time into a hydrograph.

Hydrographs can be classified into annual and storm hydrographs. Annual hydrograph shows the long term water balance in a watershed with a relation between discharge and time over the year, while a storm hydrograph shows the effects of a particular rainfall event on the discharge of a channel. The total volume of flow under the annual hydrograph is the basin yield. Annual hydrographs may be perennial, ephemeral, or snow-fed (Figure 2.10). Perennials have continuous flow over the year which is typical of a humid climate, and most of the basin yield comes from the subsurface flow indicating that a large proportion of rainfall is infiltrated into the basin and reaches the channel as groundwater. Ephemerals are typically found in arid climates with long periods when the channel is dry indicating that the groundwater table is considerably below the channel bed. Basin yield from this watershed is the result of runoff from large storms. Snow-fed annual hydrograph has a basin yield occurring mainly in spring and early summer from snowmelt. The large volume of water stored as snow and its steady release develop smoother flow variations over the year than for the perennial or ephemeral.

A hydrograph is a continuous record of stream or river discharge (see Figure 5.1). It is a basic working unit for a hydrologist to understand and interpret.

The shape of a hydrograph is a response from a particular catchment to a series of unique conditions, ranging from the underlying geology and catchment shape to the antecedent wetness and storm duration. The temporal and spatial variations in these underlying conditions make it highly unlikely that two hydrographs will ever be the same.

Although there is great variation in the shape of a hydrograph there are common characteristics of a storm hydrograph that can be recognised. These have been described at the start of Chapter 5 where terms such as *rising limb*, *falling limb*, *recession limb* and *baseflow* are explained.

Hydrograph separation

A typical storm hydrograph is shown in Figure (2.11) with the following various parts:

ABXCYDE: total hydrograph AB: baseflow before the storm (groundwater) BXCYD: storm hydrograph (or direct runoff hydrograph) BXC: rising limb (concentration curve) CYD: falling limb (recession curve) XCY: peak segment DE: baseflow after the storm (groundwater) C: peak discharge

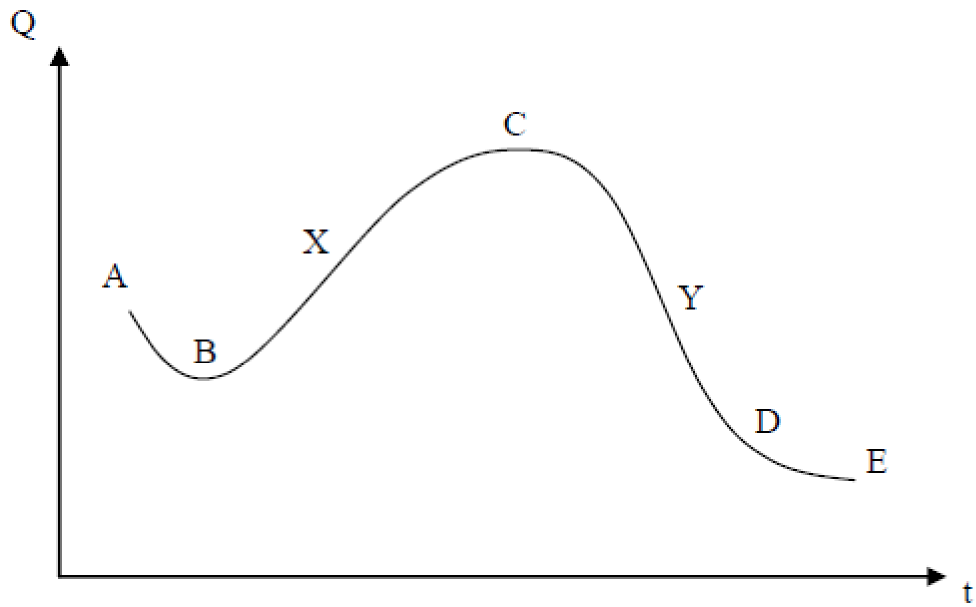

Figure (2.11). Storm hydrograph.

A common practice is to consider the hydrograph divided into two parts of baseflow and direct surface runoff hydrograph. Baseflow indicates the groundwater contribution, and direct runoff indicates the runoff caused by excess rainfall. The simplest method for the separation of baseflow and direct runoff is by drawing a straight line from B to D. The portion of the hydrograph below BD is considered to be baseflow, and that above BD is the direct runoff.

The separation of a hydrograph into baseflow and stormflow is a common task, although never easy.

The idea of **hydrograph separation** is to distinguish between stormflow and baseflow so that the amount of water resulting from a storm can be calculated. Sometimes further assumptions are made concerning where the water in each component has come from (i.e. groundwater and overland flow) but, as explained in the previous chapter, this is controversial.

The simplest form of hydrograph separation is to draw a straight, level line from the point where the hydrograph starts rising until the stream discharge reaches the same level again (dashed line in Figure 6.1). However, this is frequently problematic as the stream may not return to its pre-storm level before another storm arrives. Equally the storm may recharge the baseflow enough so that the level is raised after the storm (as shown in Figure 6.1).

To overcome the problem of a level baseflow separation a point has to be chosen on the receding limb where it is decided that the discharge has returned to baseflow. Exactly where this point will be is not easy to determine. By convention the point is taken where the recession limb fits an exponential curve. This can be detected by plotting the natural log (ln) of discharge (Q) and noting where this line becomes straight. The line drawn between the start and 'end' of a storm may be straight (dotted line, see Figure 6.1) or curved (thin solid line, see Figure 6.1) depending on the preference of hydrologist – Arnold *et al*. (1995) provides a summary of different automated techniques.

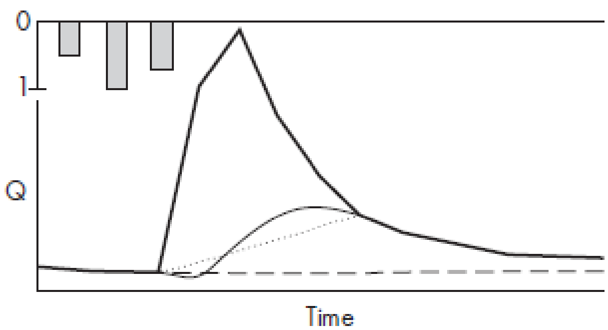

Figure 6.1 Hydrograph separation techniques. See text for explanation.

In very large catchments equation 6.1 can be applied to derive the time where stormflow ends. This is the fixed time method which gives the time from peak flow to the end of stormflow (τ):

$$\tau = D^n \qquad (6.1)$$

where D is the drainage area and n is a recession constant. When D is in square miles and τ in days, the value of n has been found to be approximately 0.2.

The problem with hydrograph separation is that the technique is highly subjective. There is no physical reasoning why the 'end' of a storm should be when the recession limb fits an exponential curve; it is pure convention. Equally the shape of the curve between start and 'end' has no physical reasoning. It does not address the debate covered in Chapter 5: where does the stormflow water come from? Furey and Gupta (2001) have recently provided a hydrograph separation technique that ties into physical characteristics of a catchment and therefore is not as subjective as other techniques, although it still requires considerable interpretation by the user. What hydrograph separation does offer is a means of separating stormflow from baseflow, something that is needed for the use of the unit hydrograph (see pp. 103–106), and may be useful for hydrological interpretation and description.

Time Base of Hydrograph

Time base of the hydrograph is defined as the time duration within which the storm hydrograph occurs. Time base can be taken as equal to the sum of the time duration of rainfall event producing the storm hydrograph and the time of concentration for the drainage area. The time of concentration is defined here as the flow time from the most remote point in the drainage area to the outlet.

The unit hydrograph

A unit hydrograph is the hydrograph of direct water surface runoff that results from a total depth of *one* unit of excess rainfall (e.g., 1 in, 1 mm, etc.) uniformly distributed over the basin and occurring within a specified duration of time. Unit hydrograph method is useful in representing the effects of variable rainfall patterns on a particular basin, since the procedure developed can make use of the linear theory practiced in various branches of engineering. Two characteristics of linear systems are that they are linearly scalable (proportionality) and can be added together (superposition). Scaling the unit hydrograph by the amount of excess rainfall illustrates the concept of proportionality, while adding multiple direct runoff hydrographs illustrates the concept of superposition. The unit hydrograph theory is based on several assumptions. The storm used in deriving the unit hydrograph should be restricted to constant rainfall intensity within the time duration, implying a short duration storm event. The time of concentration for the basin is considered to remain constant for any rainfall intensity and duration. The rainfall intensity should also have uniform spatial distribution over the watershed area, implying that the watershed is not too large. If the watershed area is too large, then it can be divided into subareas of which each has to be analyzed separately. The unit hydrograph reflects the unchanging characteristics of the watershed when channel conditions remain unchanged and the catchment does not have appreciable storage. An example for a violated condition is when the watershed contains many reservoirs or when the water flood overflows into the floodplain producing considerable storage. The total duration of a unit hydrograph, referred as the base time, depends only on rainfall duration, not the excess rainfall intensity. Here, the unit hydrograph notation to be used is UHd, where "*d*" denotes the rainfall duration.

The idea of a **unit hydrograph** was first put forward by Sherman, an American engineer working in the 1920s and 1930s. The idea behind the unit hydrograph is simple enough, although it is a somewhat tedious exercise to derive one for a catchment. The fundamental concept of the unit hydrograph is that the shape of a storm hydrograph is determined by the physical characteristics of the catchment. The majority of those physical characteristics are static in time, therefore if you can find an average hydrograph for a particular storm size then you can use that to predict other storm events. In short: two identical rainfall events that fall on a catchment with exactly the same antecedent conditions should produce identical hydrographs.

With the unit hydrograph a hydrologist is trying to predict a future storm hydrograph that will result from a particular storm. This is particularly useful as it gives more than just the peak runoff volume and includes the temporal variation in discharge.

Sherman (1932) defines a unit hydrograph as 'the hydrograph of surface runoff resulting from **effective rainfall** falling in a unit of time such as 1 hour or 1 day'. The term effective rainfall is taken to be that rainfall that contributes to the storm hydrograph. This is often assumed to be the rainfall that does not infiltrate the soil and moves into the stream as overland flow. This is infiltration excess or Hortonian overland flow. Sherman's ideas fitted well with those of Horton. Sherman assumed that the 'surface runoff is produced uniformly in space and time over the total catchment area'.

Deriving the unit hydrograph: step 1

Take historical rainfall and streamflow records for a catchment and separate out a selection of typical single-peaked storm hydrographs. It is important that they are separate storms as the method assumes that one runoff event does not affect another. For each of these storm events separate the baseflow from the stormflow; that is, hydrograph separation (see p. 102). This will give you a series of storm hydrographs (without the baseflow component) for a corresponding rainfall event.

Deriving the unit hydrograph: step 2

Take a single storm hydrograph and find out the total volume of water that contributed to the storm.

This can be done either by measuring the area under the stormflow hydrograph or as an integral of the curve. If you then divide the total volume in the storm by the catchment area, you have the runoff as a water equivalent depth. If this is assumed to have occurred uniformly over space and time within the catchment then you can assume it is equal to the effective rainfall. This is an important assumption of the method: that the effective rainfall is equal to the water equivalent depth of storm runoff. It is also assumed that the effective rainfall all occurred during the height of the storm (i.e. during the period of highest rainfall intensity). That period of high rainfall intensity becomes the time for the unit hydrograph.

Deriving the unit hydrograph: step 3

The unit hydrograph is the stormflow that results from one unit of effective rainfall. To derive this you need to divide the values of stormflow (i.e. each value on the storm hydrograph) by the amount of effective rainfall (from step 2) to give the unit hydrograph. This is the discharge per millimetre of effective rainfall during the time unit.

Deriving the unit hydrograph: step 4

Repeat steps 2 and 3 for all of the typical hydrographs.

Then create an average unit hydrograph by merging the curves together. This is achieved by averaging the value of stormflow for each unit of time for every derived unit hydrograph. It is also possible to derive different unit hydrographs for different rain durations and intensities, but this is not covered here (see Maidment, 1992, or Shaw, 1994, for more details).

Unit Hydrographs of Different Duration

In many occasions, the rainfall pattern is not simple by which it produces a composite storm hydrograph. In these cases, the unit hydrograph can be obtained indirectly by following the procedure.

For a given unit hydrograph of a specific duration, another one with different duration can be derived for the same watershed by two methods.

Lagging Method

Using a unit hydrograph of duration *a* hours, another one with duration *b* hours can be obtained by lagging UHa by *a* hours as many as *b/a* times and superposing all these hydrographs together. The sum of these hydrographs must be divided by *b/a* to obtain UHb. This method requires *b/a* to be integer; therefore, this method is also called “Integral Multiples Method”.

S-hydrograph Method

For a storm with excess rainfall duration not being an integral multiple of the unit hydrograph, the S-hydrograph method can be used. S-hydrograph is a direct runoff hydrograph for a continuous rainfall duration and constant intensity. As shown in Figure (2.12), the S-hydrograph *Sa* can be obtained by lagging and superposing infinitely many unit hydrographs UH*a* with duration *a* and excess rainfall intensity*1/a*. If *Sa* is lagged for *b* hours (denoted by *Sb*) and subtracted as *Sa* – *Sb*, then a hydrograph is obtained of a rainfall duration *b* and intensity*1/a*, i.e. with total rainfall depth *b/a*. Accordingly, a unit hydrograph of duration *b* can be determined as

$$UH_b = \frac{S_a - S_b}{(b/a)} \qquad (2.7)$$

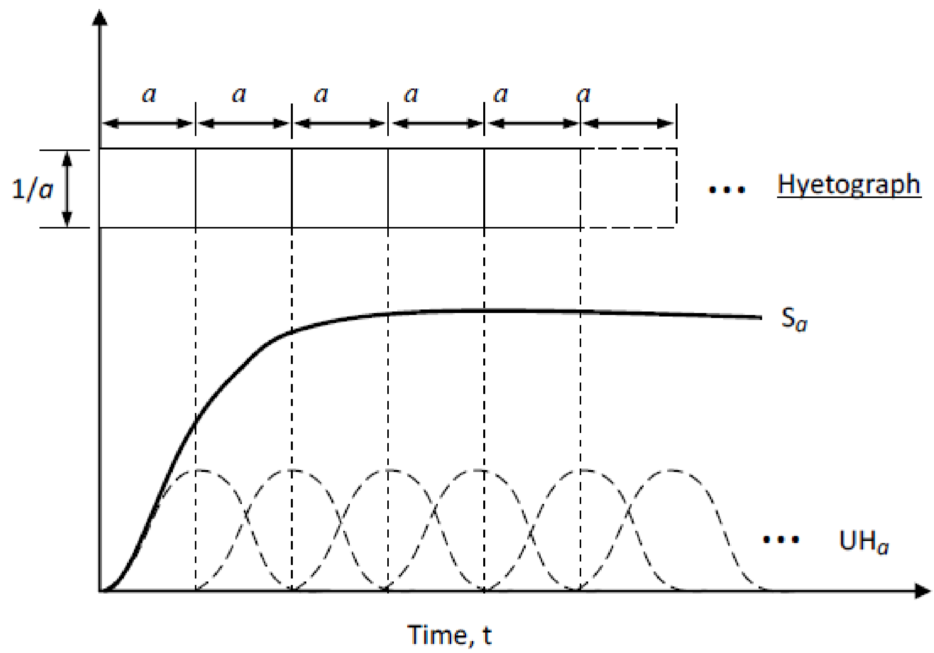

Figure (2.12). The S-Hydrograph.

Uses of unit hydrograph

The unit hydrograph obtained from the steps described here theoretically gives you the runoff that can be expected per mm of effective rainfall in one hour. In order to use the unit hydrograph for predicting a storm it is necessary to estimate the ‘effective rainfall’ that will result from the storm rainfall. This is not an easy task and is one of the main hurdles in using the method. In deriving the unit hydrograph the assumption has been made that ‘effective rainfall’ is the rainfall which does not infiltrate but is routed to the stream as overland flow (Hortonian). The same assumption has to be made when utilizing the unit hydrograph. To do this it is necessary to have some indication of the infiltration characteristics for the catchment concerned, and also of the antecedent soil moisture conditions. The former can be achieved through field experimentation and the latter through the use of an antecedent precipitation index (API). Engineering textbooks give examples of how to use the API to derive effective rainfall. The idea is that antecedent soil moisture is controlled by how long ago rain has fallen and how large that event was. The wetter a catchment is prior to a storm, the more effective rainfall will be produced.

Once the effective rainfall has been established it is a relatively simple task to add the resultant unit hydrographs together to form the resultant storm hydrograph. The worked example shows how this procedure is carried out.

Limitations of the unit hydrograph

The unit hydrograph has several assumptions that at first appearance would seem to make it inapplicable in many situations. The assumptions can be summarized as:

• The runoff that makes up stormflow is derived from infiltration excess (Hortonian) overland flow. As described in Chapter 5, this is not a reasonable assumption to make in many areas of the world.

• That the surface runoff occurs uniformly over the catchment because the rainfall is uniform over the catchment. Another assumption that is difficult to justify.

• The relationship between effective rainfall and surface runoff does not vary with time (i.e. the hydrograph shape remains the same between the data period of derivation and prediction). This would assume no land-use change within the catchment, as this could well affect the storm hydrograph shape.

Given the assumptions listed above it would seem extremely foolhardy to use the unit hydrograph as a predictive tool. However, the unit hydrograph has been used successfully for many years in numerous different hydrological situations. It is a very simple method of deriving a storm hydrograph from a relatively small amount of data. The fact that it does work (i.e. produces meaningful predictions of storm hydrographs), despite being theoretically flawed, would seem to raise questions about our understanding of hydrological processes. The answer to why it works may well lie in the way that it is applied, especially the use of effective rainfall. This is a nebulous concept that is difficult to describe from field measurements. It is possible that in moving from actual to effective rainfall there is a blurring of processes that discounts some of the assumptions listed above. The unit hydrograph is a black box model of stormflow (see end of this chapter) and as such hides many different processes within. The simple concept that the hydrograph shape is a reflection of the static characteristics and all the dynamic processes going on in a catchment makes it highly applicable but less able to be explained in terms of hydrological theory.

The synthetic unit hydrograph

The **synthetic unit hydrograph** is an attempt to derive the unit hydrograph from measurable catchment characteristics rather than from flow data. This is highly desirable as it would give the opportunity to predict stormflows when having no historical streamflow data; a common predicament around the world. The Institute of Hydrology in the UK carried out an extensive study into producing synthetic unit hydrographs for catchments, based on factors such as the catchment size, degree of urbanization, etc. (NERC, 1975). They produced a series of multiple regression equations to predict peak runoff amount, time to peak flow, and the time to the end of the recession limb based on the measurable characteristics. Although this has been carried out relatively successfully it is only applicable to the UK as that is where the derivative data was from. In another climatic area the hydrological response is likely to be different for a similar catchment. The UK is a relatively homogeneous climatic area with a dense network of river flow gauging, which allowed the study to be carried out.

In areas of the world with great heterogeneity in climate and sparse river monitoring it would be extremely difficult to use this approach.

Module – III

Reservoir management: Fixation of reservoir capacity, Ripple's mass curve, sequent peak algorithm, allocation of storage space for various uses, reservoir sedimentation and its control, determination of sediment yield at a reservoir site.

Reservoir Characteristics

Self-functioning sediment reservoirs are a relatively new addition to the reclamation toolkit.

This type of reservoir is designed to discharge water through a sand filter that is part of the embankment. Self-functioning reservoirs operate without pumping, sampling, or logistical activities in all weather and all conditions at all times of the day and night. Periodic maintenance is required to remove sediment from the reservoir side of the sand filter.

If built according to appropriate designs, the discharge from a self-functioning sediment reservoir will be cleaner than required to meet limits for suspended solids, so sampling may not be required. This should be authorized by the appropriate regulatory agency prior to permitting and construction. Self-functioning reservoirs, as with any regulated structure, should be designed by a properly qualified professional. Passive sand filter sediment reservoirs have been in use for several years.

4. Backfill Impoundments

Applicability

Water structures are needed on a mine site for water treatment and storage. Impoundments created in backfill voids are usually more cost effective than impoundments on unmined areas.

Special Considerations

Uncompacted backfill slopes may not be stable enough to support equipment if the slopes become saturated.

Techniques

a. Design Considerations

Incised backfill impoundments are created by leaving a void area in the backfill and grading the slopes. Though simple, the following basic design features need to be considered: size, shape, cells, inlet structures, Mine Safety and Health Administration (MSHA) criteria, stability, location, function, and construction. The function of a reservoir will determine the extent that the design incorporates features for water treatment (shallows, cells, flat slopes, length approximately four times width) or water storage (deeper, steeper sides, length to width ratio variable).

In general, reservoirs should be designed with water treatment in mind, as good water quality is important for any function, and the incremental cost of a treatment reservoir is low compared to reworking or repairing any features of an inexpensive water storage reservoir. Backfill impoundments primarily intended to store water can be deeper, with shape dictated more by available topography, and with a reduced high-water line area to minimize evaporation.

(1) Size

The structure must be large enough to handle the design runoff event plus any pit or process water, and may include additional capacity for dust control or to provide a convenient location to hold excess runoff and minimize offsite discharges. Backfill impoundments with at least 100 acre-feet of water storage are commonplace for large surface mines. This large capacity provides flexibility in pumping and holding large volumes of water in the structure until a discharge is convenient or can be avoided. A larger size reservoir is more conducive to treatment and recycling of water from washing operations. Such a reservoir provides the treatment and storage capacity for dry periods when other sources of water become less reliable.

(2) Shape

If water treatment is the primary purpose of the reservoir, it will be the most effective in a long, shallow (two to three feet deep) reservoir with the inlets as far as possible from the water discharge location. This assures the shortest distances for sediment to drop to the bottom, and limits short circuiting (inflow water traveling directly to the pond outlet). As a guide, a backfill reservoir intended primarily for water treatment should average 5 to 30 feet deep, 200 to 500 feet wide, and 500 to 4000 feet in length, depending on the volume to be treated. Side slopes should be 3H:1V to 4H:1V to accommodate access by people and wildlife (Figure B-4-1). Angle of repose slopes 1.5H:1V that become saturated will likely fail. Mobile heavy equipment should be kept off the saturated, uncompacted slopes characteristic of this type of impoundment.

(3) Cells

Cells should be strongly considered to control any oil spill that might enter the reservoir, limit wave fetch, and eliminate short circuiting caused by temperature inversions (i.e. warm plant water). The connections between cells typically would be culverts installed below the water surface.

All cells should be designed to be at the same elevation regardless of the reservoir volume. Because the base and sides of the reservoir and the dividing embankment are not compacted, cells with differing water levels will likely have piping or seepage from cell to cell. As the soil settles, failure between cells is likely. Impoundments constructed primarily for water storage reservoirs do not necessarily need a dividing embankment.

(4) Inlet Structure

Inlet structures must be considered for any reservoir that is expected to function more than one year. Surface or pipeline water that is released above the high-water line and washes down an unprotected slope will cause significant erosion in one season and more quickly if there is a continuous water flow. The washing of slope soil into the reservoir bottom will quickly fill the lower reservoir volume up to, or possibly clog, a connecting culvert or pump culvert, and will compromise the already marginal stability of the uncompacted slope.

One option is a culvert to deliver inflow below the water surface. If the pipeline inlet is on the reservoir floor, it will generally stir up sediment and decrease the reservoir's water treatment effectiveness. Using the same culvert for inflow and outflow can be done, but should only be considered if the quality of the source water is very clean. Pumping sediment-laden water into the pump culvert will cause pump problems (Figures B-4-2, 3, and 4).

(5) MSHA

Generally, MSHA has jurisdiction over any water structure that stores 20 acre-feet or more of water or slurry behind an embankment that is five feet or higher, as measured from the upstream toe to the emergency spillway, or has an embankment that is 20 feet or higher. A 100 acre-feet or larger reservoir normally falls within these criteria.

An incised backfill reservoir does not have a compacted embankment storing water above the surrounding ground level. This generally eliminates such impoundments from MSHA jurisdiction. MSHA and the mines have used the following rule of thumb to qualify the reservoir as incised: backfill 200 to 300 feet wide between the reservoir and the pit, and higher than the reservoir. In

1993, MSHA qualified this by stating that for a structure to be incised and not require design approval, the minimum backfill slope from the high-water line to the pit floor had to be at least 10H: 1V.

(6) Stability

Uncompacted, saturated backfill is usually unstable when used to contain water or support weight. When used with very flat slopes such as MSHA suggests (10H:1V), stability should not be an issue.

The reservoir site must be conducive to water retention. This means the side slopes and reservoir bottom must be soils that exhibit low permeabilities.

Zones of sand or waste coal in or in close proximity to the base or side are likely to cause significant water loss through seepage. A very porous soil backfilled to the reservoir may generate a seep on a backfill bench, if one is nearby.

(7) Location

The location should be chosen to maximize life and assure stability. The longer the reservoir is used, the more cost effective it is. Designing an impoundment to be non-MSHA is a major help in timing and cost. Position the reservoir where MSHA jurisdiction is not required, and the saturated backfill does not become a future obstacle to mining.

(8) Function

Multiple functions of water treatment, water storage for dust suppression, wildlife use, and post-mining land use should be considered in the given priority.

(a) Water Treatment

Water Treatment for pit, plant, or disturbed area water sources is usually the primary concern. This reservoir is not likely the final treatment, but the initial step to reduce the Total Suspended Solids (TSS).

(b) Water Storage

A second concern is water storage for use during dust control activities, for storage of water awaiting treatment, or for simply providing a holding area to minimize discharges or provide a water source in extended dry periods.

(c) Wildlife

A reservoir can be made to have some wildlife features at a minimal cost (the subsection entitled "Wetlands"). These features should be included to show compatibility of the mining operation with wildlife.

(d) Post-mining Land Use

It is an added bonus if the structure (or part of it) can be made part of the post-mining land use. Keep in mind that water sources may deposit coal fines that should be covered before final reclamation. You may want to design the reservoir to be partly buried by final reclamation to cover unsuitable sediment.

(9) Construction

The construction cost-saving in this structure is the use of uncompacted fill for water containment. Sides are dumped in and dozed to final slope (Figure B- 4-1). Berms are required by MSHA to prevent a vehicle from entering the impoundment. A spillway needs to be installed to provide six to ten feet of freeboard. Freeboard is from the high-water line when spilling to the top of the containing backfill bench.

Do not consider a compacted embankment on top of uncompacted backfill. Differential settling of the backfill will negate the purpose of the compaction and possibly cause failure of the embankment. For reservoir access, an old haul road ramp is good (if one is present) for skid mounted pumps or other equipment, because the base will remain firm. Compaction (minimum 90 percent standard proctor is suggested) should be performed around dewatering pipe stands.

(10) Dewatering

a. Gravity Discharge

The simplicity of gravity discharge should be considered because no pump is required. The downside is that it may not always provide surge capacity, there may not be a good way to shut the system off, and a pump may be required to load water trucks or recycle water. Also, gravity discharge is often not possible because the reservoir is usually lower than the surrounding area.

b. Pumps

Installation of two pumps should be considered to maintain reservoir capacity and discharge system reliability. Pumps with durable, slower speed impellers are better because they will last longer. Existing pump stock may dictate the pump to be used. Pumps that are in continuous use for long periods should be electrically driven, if power is available, because of the lower maintenance and cost compared to diesel engine driven pumps.

Figure B-4-4 illustrates a desirable pump evacuation method for longer duration installations and the float and pump method, which is usually much cheaper for up to three years duration. Access, safety, winter ice, and maintenance make the land-based design much more desirable when it must be used for many years.

4. Planning and Constructing Permanent Post-mining Impoundments

Applicability

Post-mining impoundments are constructed for a variety of reasons. This includes replacement of pre-existing impoundments and construction of new impoundments, where feasible, to take advantage of the resulting reduction in backfill replacement movement. Wetland replacement is another common reason for constructing post-mining impoundments. However, wetlands and dam construction techniques are discussed elsewhere in this section. This subsection will deal more with the special considerations faced in planning and constructing permanent post-mining impoundments.

Special Considerations

Function of post-mining impoundments will be influenced by water supply and quality, chemical characteristics of backfill underlying the reservoir basin, and sediment inflow. From a permitting standpoint, changes in impoundment function or size from pre-mining conditions may trigger a requirement from the regulatory agency to similarly modify post-mining land use descriptions in the Surface Mine Control and Reclamation Act (SMCRA) permit prior to approval. Under SMCRA, reservoir feasibility is scrutinized heavily both in design and in performance (via monitoring). If the impoundment is not a replacement feature, the agency likely will request an alternate reclamation plan to be implemented if the structure does not function as planned.

Technique

a. Design and Permitting Considerations

Permanent post-mining impoundments are as much a permitting project as a construction project. Typically, permitting and construction of smaller features, such as stock ponds, are not as rigorous or costly as similar work for much larger bodies with a capacity of several hundred or several thousand acre-feet. These larger impoundments are heavily scrutinized from the design and performance standpoints.

(1) Uses

Design and permit the impoundment for its anticipated uses. Larger impoundments intended for wildlife, fish, and recreation, in addition to stock watering, will have different characteristics than small stock ponds alone.

Consult established guidelines for appropriate reservoir characteristics prior to design, so you and your management know where you're headed. For waterfowl or fish, for example, guidelines give recommended percentages forlittoral zone (shallows), as well as deeper water, to make the impoundment more conducive to healthy populations.

(2) Topsoil

Make sure a reasonable and protective topsoil handling plan is put forth.

Without ample topsoil, compliance problems will arise immediately. Typically, for impoundment drainage areas that are actively disturbed or reclaimed but still bonded, the reservoir basin should contain no topsoil below the high water line. This may cause contamination from overland flow.

(3) Water Supply

Permit the impoundment for an expected water supply. Stock ponds can be permitted to contain the mean annual flood (or some other relatively frequent event, to keep fresh water available) estimated by modeling. Larger impoundments may require operational studies to determine a size that will operate properly given the intended use.

Assess such parameters as spilling frequency, range of water level fluctuations, and possibly a salt (TDS) balance to evaluate water quality over time and especially during drought periods. These studies will have to take downstream, senior water rights into account, and may result in construction of a low flow bypass or other mechanism (or administrative arrangement) toA water right should be obtained for the anticipated capacity of the impoundment; but remember that, upon construction, the final as-built capacity cannot exceed the permitted capacity. This is good to know as construction is commencing so no surprises are encountered. A beautiful pond may never function if insufficient attention is paid to water supply.

(4) Freeform Work

As much as possible, leave flexibility in your permitting language to do free form work in the field. The freedom to add cost-effective shoreline undulations, small islands, riprap extensions, or simply keep working if unexpected conditions are encountered is invaluable if, as a situation arises, you don't have to convene the legislature to continue. But behave yourself; unlimited flexibility is dangerous when it reduces reliance on planning.

(5) Reservoir Characteristics

Pay special attention to reservoir characteristics. Safety must be considered because of the inherent questions about the long-term stability and settlement characteristics of mine backfill. Impoundments in backfill are more safely operated if they are totally incised than if they include an embankment sitting on many tens of feet of uncompacted truck/shovel or dragline spoils. Plus, in the incised case, a much smaller spillway or overflow channel is needed because there is no embankment to protect. Be aware of the safety concerns associated with operating heavy equipment along the banks and shorelines of backfill impoundments as these will be areas of instability when saturated.

(6) Industrial Uses

Make sure the impoundment is permitted for at least some industrial uses.

Impoundments can be a convenient water storage feature your production department will want to use, or need to use, at some point. Plus, pit water or disturbed area runoff can enhance the speed with which the reservoir will fill, if it is not to be hydraulically connected to its ultimate drainage area for a period of time. This is a two-edged sword, however. While the extra water can speed maturation of the impoundment, the mining operation must be careful not to introduce unwanted sediment or poor quality water into a structure you hope to be a clean contributor to the post-mining environment.

Also, the extra water may bias your evaluation of the water balance for the reservoir, if one is necessary to prove its ultimate functions.

b. Construction Considerations

(1) Strip Topsoil

Much work will be going on in the vicinity of the reservoir basin and dam.

Because of this, a first step is to remove all remaining topsoil in the dam, spillway, reservoir basin, borrow, and access areas.

(2) Work Dry

Working in wet bottomlands generally can be avoided when creating impoundments in older backfill or small drainages. If you must work in boggy areas, be cognizant of special sediment control measures that may be needed. Also, make sure your equipment operator or contractor can handle the work. In tricky areas, for instance, a floatation dozer may be required.

When working in a wet drainage, coffer dams, a temporary diversion, or pumping will be required to keep the active areas workable. In ephemeral channels, most water encountered will be subsurface. A pre-construction borehole grid should awaken you to any potential construction problems.

(3) Survey Control

Maintain good survey control. This is especially important for the as-built drawings, spillway and dam crest elevation confirmation, and high water line delineation. Stake the high water line well in advance of construction to highlight problem areas that may not have been caught when working from maps or photos (i.e. was a power line installed recently, or are there trees that can be salvaged?).

(4) Material Sources and Quantities

Permanent impoundments in backfill environments, particularly the larger ones, will typically require the placement of a layer of "suitable" earth for a specified depth beneath the reservoir bottom. This is material that meets chemical quality criteria agreed to in the permitting process. The suitable zone may be four to eight feet in thickness or more, so a substantial amount must be available. When the time comes that this material is needed, the source quantity and quality must not be in question.

(5) Materials Testing

Do not skimp on materials testing. Since this structure will have to prove its performance during the bond period, and is intended to last many years beyond, thorough testing and documentation thereof is critical. Materials testing will include soil quality (paragraph D above), compaction, gradation (e.g. for riprap), and classification (for embankment zones).

(6) Monitoring

Especially for the larger impoundments, provisions should be made during construction for the water quantity and quality monitoring that will often be required as a condition of approval. This will include, at a minimum, one or more staff gages, survey monuments (for the embankment, and for the reservoir basin if constructed on backfill), and water quality monitoring station locations. Be sure to stash a johnboat in the brush nearby, if access is unpredictable, to allow for bathymetric surveys and depth-integrated sampling in the future. If the structure serves any industrial uses, it will likely end up as an National Pollutant Discharge Elimination System (NPDES) discharge point. In this case, additional equipment will include a controlled lower-level discharge (in addition to the service discharge and emergency spillway) and a flume or other measuring device at the outfall.

(7) Stabilization

Stabilize areas that will be affected by flowing water. Spillways, outlet channels, inlet channels, downwind shorelines, embankments, and mechanical outlet works should all have erosion protection in place and functional before they are needed. Especially steep sections, such as in inlets to incised structures and embankment slopes, will probably require durable riprap. Other areas can adequately be stabilized with vegetation.

To enhance the long term aesthetics and habitat characteristics of the structure, consider seeding or transplanting wetland vegetation along shorelines (which should be flatter near the high water line for this purpose, if possible), although many of these will recruit naturally. Do not plant trees in embankments, as their root systems may cause destabilization.

(8) Final Touch

Hold back some materials for the final touch. As construction of the impoundment nears completion, it is nice to have some extra suitable fill, riprap, and equipment time available to add meaningful and functional features to the reservoir basin. These need not be costly or extensive. By adding a small peninsula here, a small island or two there, and some additional shoreline sculpting or rock habitat, (all done under dry conditions) the lake will take on a much more natural appearance with more valuable habitat. And, given it will be there much longer than you, it will be much more likely to look like the natural feature it is intended to be.

The above view of the 26-SR-1 Reservoir at the Black Thunder Mine provides a look at islands placed in the reservoir basin following dam construction (view is from the embankment looking west). The dam, its spillway, and the islands received topsoil to aid re-vegetation establishment. Additional features such as these islands and the littoral zone around them serve as wildlife habitat enhancement features.

Module – V

Flood frequency analysis: Gumbel's method. Flood routing: Hydrologic channel routing, Muskingum equation, hydrologic reservoir routing: Modified Plus method, flood control measures.

FREQUENCY ANALYSIS

The analysis of how often an event is likely to occur is an important concept in hydrology. It is the application of statistical theory into an area that affects many people's lives, whether it be through flooding or low flows and drought. Both of these are considered here, although because they use similar techniques the main emphasis is on **flood frequency analysis**. The technique is a statistical examination of the frequency–magnitude relationship discussed in Chapters 2 and 5. It is an attempt to place a probability on the likelihood of a certain event occurring. Predominantly it is concerned with the low-frequency, high-magnitude events (e.g. a large flood or a very low river flow).

It is important to differentiate between the uses of flow duration curves and frequency analysis. Flow duration curves tell us the percentage of time that a flow is above or below a certain level. This is average data and describes the overall flow regime.

Flood frequency analysis is concerned only with peak flows: the probability of a certain flood recurring. Conversely, **low flow frequency analysis** is concerned purely with the lowest flows and the probability of them recurring.

Flood frequency analysis

Flood frequency analysis is probably the most important hydrological technique. The concept of a '100-year flood', or a fifty-year recurrence interval, is well established in most people's perceptions of hydrology, although there are many misunderstandings in interpretation.

Flood frequency analysis is concerned with peak flows. There are two different ways that a peak flow can be defined:

• The single maximum peak within a year of record giving an **annual maximum series**; or

• Any flow above a certain threshold value, giving a **partial duration series**.

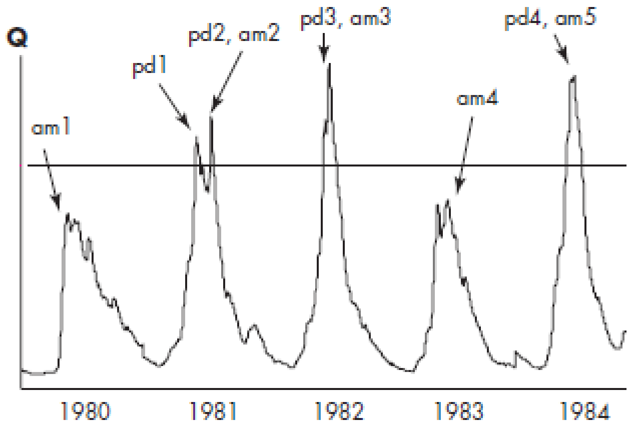

Figure 6.10 Daily flow record for the Adams river (British Columbia, Canada) during five years in the 1980s. Annual maximum series are denoted by 'am', partial duration series above the threshold line by 'pd'. NB In this record there are five annual maximum data points and only four partial duration points, including two from within 1981.

Source: Data courtesy of Environment Canada

Figure 6.10 shows the difference between these two data series. There are arguments for and against the use of either data series in flood frequency analysis.

Annual maximum may miss a large storm event where it occurs more than once during a year (as in the 1981 case in Figure 6.10), but it does provide a continuous series of data that are relatively easy to process. The setting of a threshold storm (the horizontal line in Figure 6.10) is critical in analysis of the partial duration series, something that requires considerable experience to get right. The most common analysis is on annual maximum series, the simplest form, which is described here. If the data series is longer than ten years then the annual maxima can be used; for very short periods of record the partial duration series can be used.

The first step in carrying out flood frequency analysis is to obtain the data series (in this case annual maxima). The annual maximum series should be for as long as the data record allows. The greater the length of record the more certainty can be attached to the prediction of average recurrence interval. Many hydrological database software packages (e.g. HYDSYS) will give annual maxima data automatically, but some forethought is required on what annual period is to be used. There is an assumption made in flood frequency analysis that the peak flows are independent of each other (i.e. they are not part of the same storm). If a calendar year is chosen for a humid temperate environment in the northern hemisphere, or a tropical region, it is possible that the maximum river flow will occur in the transition between years (i.e. December/ January). It is possible for a storm to last over the 31 December/1 January period and the same storm to be the maximum flow value for both years. If the flow regime is dominated by snow melt then it is important to avoid splitting the hydrological year at times of high melt (e.g. spring and early summer). To avoid this it is necessary to choose your hydrological year as changing during the period of lowest flow. This may take some initial investigation of the data.

All flood frequency analysis is concerned with the analysis of frequency histograms and probability distributions. Consequently the first data analysis step should be to draw a frequency histogram. It is often useful to convert the frequency into a relative frequency (divide the number of readings in each class interval by the total number of readings in the data series).

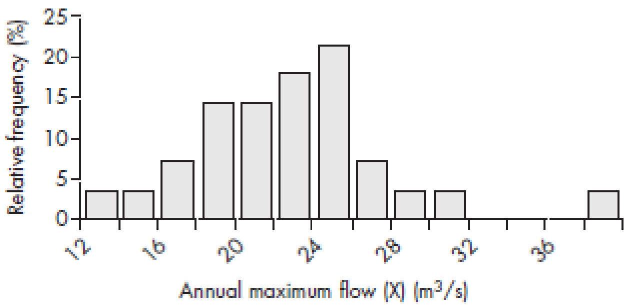

Figure 6.11 Frequency distribution of the Wye annual maximum series.

The worked example given is for a data set on the river Wye in mid-Wales (see pp. 113–114). On looking at the histogram of the Wye data set (Figure 6.11) the first obvious point to note is that it is not normally distributed (i.e. it is not a classic bell shaped curve). It is important to grasp the significance of the non-normal distribution for two reasons:

- Common statistical techniques that require normally distributed data (e.g. t-tests etc.) cannot be applied in flood frequency analysis.
- It shows what you might expect: small events are more common than large floods, but that very large flood events do occur; i.e. a high magnitude, low frequency relationship.

If you were to assume that the data series is infinitely large in number and the class intervals were made extremely small, then a smooth curve can be drawn through the histogram. This is the *probability density function* which represents the smoothed version of your frequency histogram.

In flood frequency analysis there are three interrelated terms of interest. These terms are interrelated mathematically, as described in equation 6.2 in the text below.

1 The probability of excedence: $P(X)$. This is the probability that a flow (Q) is greater than, or equal to a value X. The probability is normally expressed as a unitary percentage (i.e. on a scale between 0 and 1).

2 The relative frequency: $F(X)$. This is the probability of the flow (Q) being less than a value X. This is also expressed as a unitary percentage.

3 The average recurrence interval: $T(X)$. This is sometimes referred to as the return period, although this is misleading. $T(X)$ is a statistical term meaning the chance of excedence once every T years over a long record. This should not be interpreted as meaning that is exactly how many years are likely between certain size floods.

$$P(X) = 1 - F(X)$$
$$T(X) = \frac{1}{P(X)} = \frac{1}{1 - F(X)} \qquad (6.2)$$

It is possible to read the values of $F(X)$ from a cumulative probability curve; this provides the simplest method of carrying out flood frequency analyses. One difficulty with using this method is that you must choose the class intervals for the histogram carefully so that the probability density function is an accurate representation of the data.

Too large an interval and the distribution may be shaped incorrectly, too small and holes in the distribution will appear.

One way of avoiding the difficulties of choosing the best class interval is to use a rank order distribution.

This is often referred to as a plotting position formula.

The Weibull formula

The first step in the method is to rank your annual maximum series data from low to high. In doing this you are assuming that each data point (i.e. the maximum flood event for a particular year) is independent of any others. This means that the year that the flood occurred in becomes irrelevant.

Taking the rank value, the next step is to calculate the $F(X)$ term using equation 6.3. In this case r refers to the rank of an individual flood event (X) within the data series and N is the total number of data points (i.e. the number of years of record):

$$F(X) = \frac{r}{N + 1} \qquad (6.3)$$

In applying this formula there are two important points to note:

1 The value of $F(X)$ can never reach 1 (i.e. it is asymptotic towards the value 1).

2 If you rank your data from high to low (i.e. the other way around) then you will be calculating the $P(X)$ value rather than $F(X)$. This is easily rectified by using the formula linking the two.

The worked example on pp. 113–114 gives the $F(X)$, $P(X)$ and $T(X)$ for a small catchment in mid-Wales (Table 6.3).

The Weibull formula is simple to use and effective but is not always the best description of an annual maximum series data. Some users suggest that a better fit to the data is provided by the Gringorten formula (equation 6.4):

$$F(X) = \frac{r - 0.44}{N + 0.12} \qquad (6.4)$$

As illustrated in the worked example, the difference between these two formulae is not great and often the use of either one is down to personal preference.

Extrapolating beyond your data set

The probabilities derived from the Weibull and Gringorten formulae give a good description of the flood frequency within the measured stream record but do not provide enough data when you need to extrapolate beyond a known time series. This is a common hydrological problem: we need to make an estimate on the size of a flood within an average recurrence interval of fifty years but only have twenty-five years of streamflow record. In order to do this you need to fit a distribution to your data.

There are several different ways of doing this, the method described here uses the method of moments based on the Gumbel distribution. Other distributions that are used by hydrologists include the

Log-Pearson Type III and log normal. The choice of distribution is often based on personal preference and regional bias (i.e. the distribution that seems to fit flow regimes for a particular region).

Method of moments

If you assume that the data fits a Gumbel distribution then you can use the method of moments to calculate $F(X)$ values. Moments are statistical descriptors of a data set. The first moment of a data set is the mean; the second moment the standard deviation; the third moment skewness; the fourth kurtosis. To use the formulae below (equations 6.5– 6.7) you must first find the mean ($\bar{Q}$) *and standard deviation* (σ_Q) of your annual maximum data series.

The symbol *e* in the equations 6.5–6.7 is the base number for natural logarithms or the exponential number (≈□2.7183).

$$F(X) = e^{-e^{-b(X-a)}} \quad (6.5)$$

$$a = \overline{Q} - \frac{0.5772}{b} \quad (6.6)$$

$$b = \frac{\pi}{\sigma_Q \sqrt{6}} \quad (6.7)$$

With knowledge of $F(X)$ you can find $P(X)$ and the average recurrence interval ($T(X)$) for a certain size of flow: X. The formulae above can be rearranged to give you the size of flow that might be expected for a given average recurrence interval (equation 6.8):

$$X = a - \frac{1}{b} \ln \ln \left(\frac{T(X)}{T(X) - 1} \right) \quad (6.8)$$

In the formula above ln represents the natural logarithm. To find the flow for a fifty-year average recurrence interval you must find the natural logarithm of (50/49) and then the natural logarithm of this result.

Using this method it is possible to find the resultant flow for a given average recurrence interval that is beyond the length of your time series. The further away from the length of your time series you move the more error is likely to be involved in the estimate. As a general rule of thumb it is considered reasonable to extrapolate up to twice the length of your streamflow record, but you should not go beyond this.

Low flow frequency analysis

Where frequency analysis is concerned with low flows rather than floods, the data required are an annual minimum series. The same problem is found as for annual maximum series: which annual year to use when you have to assume that the annual minimum flows are independent of each other. At mid-latitudes in the northern hemisphere the calendar year is the most sensible, as you would expect the lowest flows to be in the summer months (i.e. the middle of the year of record). Elsewhere an analysis of when low flows occur needs to be carried out so that the hydrological year avoids splitting in the middle of a low flow period. In this case $P(X)$ refers to the probability of an annual minimum greater than or equal to the value X. The formulae used are the same as for flood frequency analysis (Weibull etc.).

There is one major difference between flood frequency and low flow frequency analysis which has huge implications for the statistical methods used: there is a finite limit on how low a flow can be. In theory a flood can be of infinite size, whereas it is not possible for a low flow to be less than zero (negative flows should not exist in fresh water hydrology). This places a limit on the shape of a probability distribution, effectively truncating it on the left-hand side (see Figure 6.15).

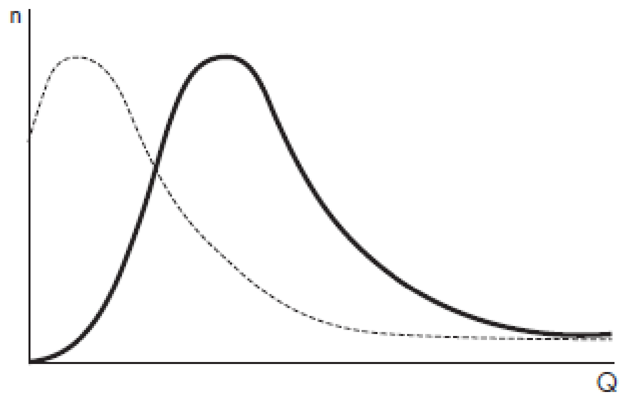

Figure 6.15 Two probability density functions. The usual log-normal distribution (solid line) is contrasted with the truncated log-normal distribution (broken line) that is possible with low flows (where the minimum flow can equal zero).

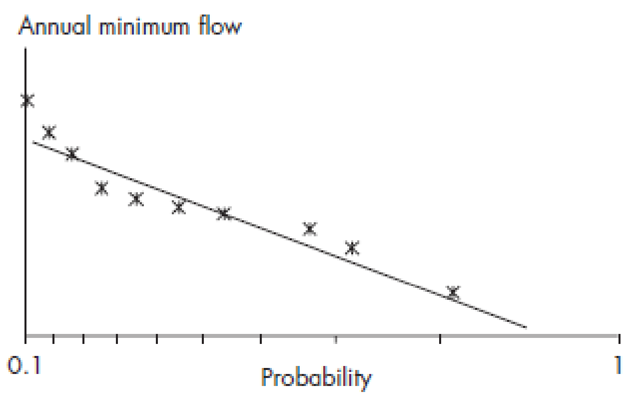

Figure 6.16 Probability values (calculated from the Weibull sorting formula) plotted on a log scale against values of annual minimum flow (hypothetical values).

The statistical techniques described on pp. 111–112 (for flood frequency analysis) assume a full log-normal distribution and cannot be easily applied for low flows. Another way of looking at this problem is shown in Figure 6.16 where the probabilities calculated from the Weibull formula are plotted against the annual minimum flow values. The data fit a straight line, but if we extrapolate the line further it would intersect the x-axis at a value of approximately 0.95. The implication from this is that there is a 5 per cent chance of having a flow less than zero (i.e. a negative flow). The way around this is to fit an exponential rather than a straight line to the data. This is easy to do by eye but complicated mathematically. It is beyond the level of this text to describe the technique here (see Shaw, 1994, or Wang and Singh, 1995 for more detail). Gordon *et al*. (1992) provide a simple method of overcoming this problem, without using complicated line-fitting procedures.

Limitations of frequency analysis

As with any estimation technique there are several limitations in the application of frequency analysis; three of these are major:

1 The estimation technique is only as good as the streamflow records that it is derived from. Where the records are short or of dubious quality very little of worth can be achieved through frequency analysis. As a general rule of thumb you should not extrapolate average recurrence intervals beyond twice the length of your data set. There is a particular problem with flood frequency analysis in that the very large floods can create problems for flow gauges and therefore this extreme data may be of dubious quality (see pp.89).

2 The assumption is made that each storm or low flow event is independent of another used in the data set. This is relatively easy to guard against in annual maximum (or minimum) series, but more difficult for a peak threshold series.

3 There is an inherent assumption made that the hydrological regime has remained static during the complete period of record. This may not be true where land use, or climate change, has occurred in the catchment (see Chapter 8).

FLOOD ROUTING

In a large watershed, the shape of a storm hydrograph will change as the flow moves from upstream to downstream locations. The reason is related to the different storage characteristics and time of water travel between the two locations. Routing is the derivation of outflow hydrograph from inflow hydrograph by considering water storage variation in between. The following unsteady continuity equation is used in hydrologic routing

$$\frac{dS}{dt} = I(t) - Q(t) \qquad (2.10)$$

where S= storage volume between upstream and downstream locations; I= inflow at the upstream location; and Q= outflow at the downstream location. Two problems of flood routing are commonly analyzed: reservoir routing, and channel routing. In reservoir routing the storage is considered a nonlinear function, i.e. S=f(Q) and Q=f(H); while in channel routing the storage is linearly related to I(t) and Q(t) (see Figure 2.14).

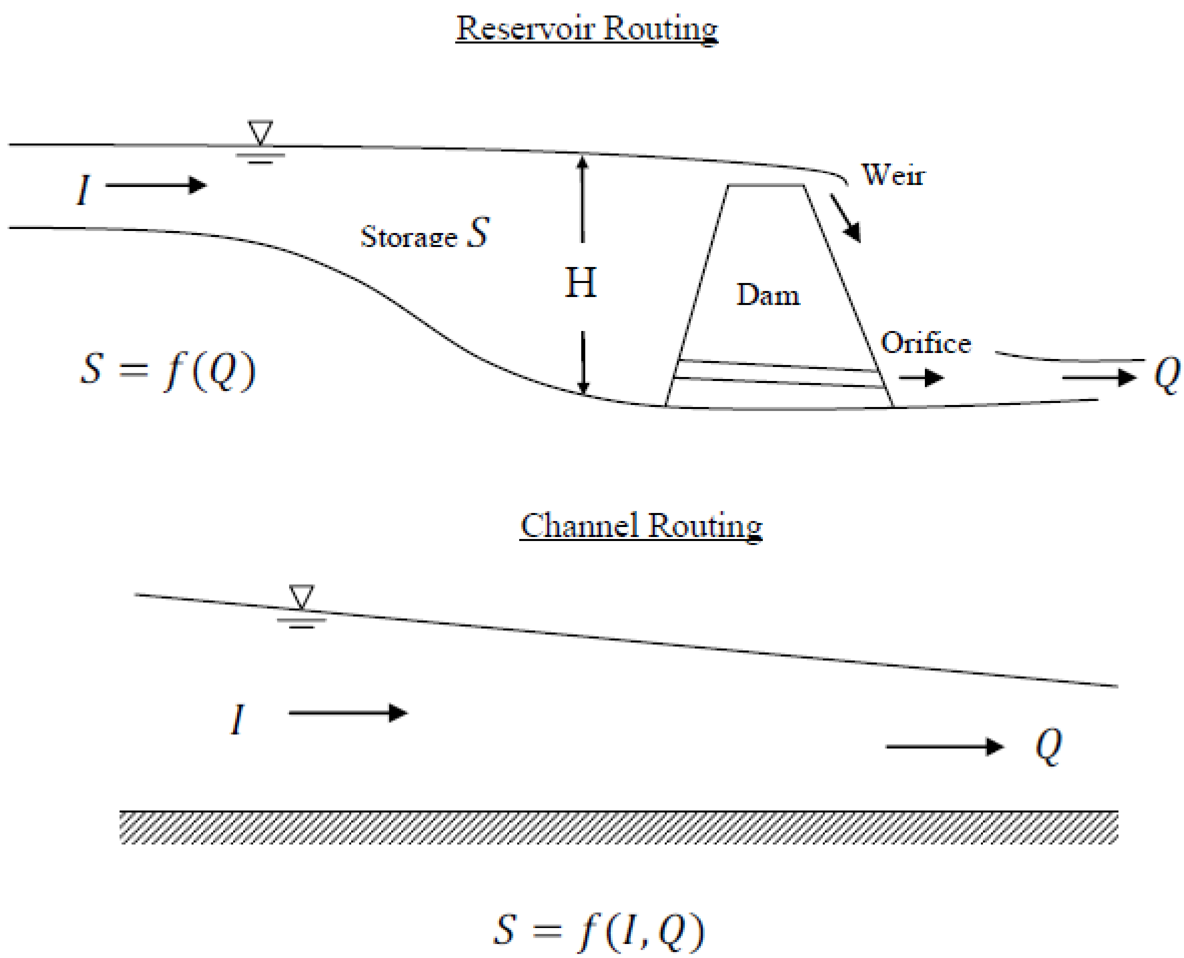

Figure (2.14). Difference between reservoir routing and channel routing.

Reservoir Routing

For computational purposes, Equation (2.10) can be written in a mathematical finite difference form as

$$\frac{S_{n+1} - S_n}{\Delta t} = \left(\frac{I_n + I_{n+1}}{2}\right) - \left(\frac{Q_n + Q_{n+1}}{2}\right) \qquad (2.11)$$

The indices n and n+1 refer to the values at the beginning and the end of time interval Δt, respectively. This equation can be conveniently rewritten as

$$(I_n + I_{n+1}) + \left(\frac{2S_n}{\Delta t} - Q_n\right) = \left(\frac{2S_{n+1}}{\Delta t} + Q_{n+1}\right) \qquad (2.12)$$

Typically, I is known for all n values, and S and Q are known for the initial time condition n=0. Consequently, the right side of Equation (2.12) can be determined. The storage indication curve is a plot of 2S/Δt +Q against Q. Thus, once the right side of Equation (2.12) is determined, the values of Q can be read directly from the storage indication curve. Values for 2S/Δt -Q for the left side are calculated by subtracting 2Q from 2S/Δt +Q. The computations are repeated until the entire outflow hydrograph is obtained. It should be noted that the storage indication curve must be prepared for a fixed Δt value same as that used in the routing procedure.

Channel Routing

Many methods are available to solve Equation (2.10) for channel routing and the most widely used is the Muskingum method. This method assumes that the storage is a linear function of weighted inflow and outflow as given by

$$S = K[XI + (1 - X)Q] \qquad (2.13)$$

where K= storage constant with the dimension of time that approximates the travel time of water through the channel; and X= dimensionless weighting constant, 0.5<=X<=1. Substituting Equation (2.13) into Equation (2.12) yields

$$Q_{n+1} = C_0 I_{n+1} + C_1 I_n + C_2 Q_n \qquad (2.14)$$

where

$$C_0 = \frac{\Delta t/2 - KX}{K(1-X) + \Delta t/2} \quad (2.15a)$$

$$C_1 = \frac{\Delta t/2 + KX}{K(1-X) + \Delta t/2} \quad (2.15b)$$

$$C_2 = \frac{K(1-X) - \Delta t/2}{K(1-X) + \Delta t/2} \quad (2.15c)$$

It should be noted that C0+C1+C2=1. The significance of this expression may be seen for steady flow condition, i.e. In=In+1=Qn=Qn+1, as Equation (2.14) becomes correct only when the sum of the constants is unity.

The parameters X and K may be estimated for a channel from given inflow and outflow hydrographs for a particular storm. The estimation can be achieved with reference to Equation (2.13) by plotting the storage S against weighted discharge XI+ (1-X)Q for several selected values of X. The Muskingum method assumes that this curve is a straight line. As seen in Figure (2.15), the selected values of X will result in loops. The value of X that gives the narrowest loop will be chosen as the X value to be used in future routing procedures. The inverse slope of the line of best fit for the narrowest loop will give the value of K.

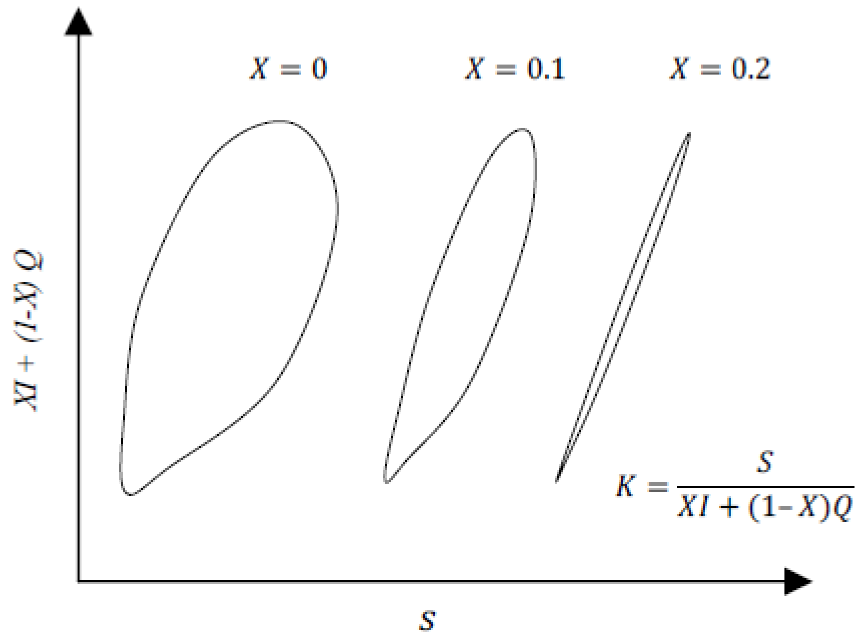

Figure (2.15). Estimation of the Muskingum X and K values.