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Basic Water Treatment

Fifth edition

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Preface

The fifth edition of this book, which has become one of the accepted primers on water-treatment, is an update of the fourth edition to reflect changes in water treatment over the past few years, and to introduce some of the new processes which are likely to become more common in the future. A section has also been added on the institutional structures of water supply across the British Isles. However this remains a relatively slim volume covering water treatment, a very broad subject and thus a degree of subjectivity has had to be exercised over what should be covered.

Basic Water Treatment is aimed at university students, at practising water treatment engineers, for whom it will be a useful reference book, and at mechanical engineers and chemists who need to put their specialised knowledge into a broader context; it will also be of interest to those managers and non-technical staff who wish to understand some of the quality and technical issues relating to water treatment. It is not a prescriptive handbook on water treatment plant design; rather it provides essential background and is a useful first choice reference book for many aspects of water quality and treatment.

It is important that all involved in water quality and water treatment do not adopt too blinkered an approach. In western Europe we now have to focus very closely on compliance with prescriptive water quality standards, although the Water Framework Directive and public concerns over the environment require a wider consideration of water resources and water demands. However, over much of the world, water engineers should consider, or at least be aware of, much wider issues, weighing the costs of high-quality water against the overriding need to supply safe water to as many as possible. *Basic Water Treatment* primarily relates to water treatment in western Europe and North America but it also takes account of treatment in other developed areas, and in developing and less developed areas.

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Chapter 1
Introduction and structure of the British water industry

Water-supply engineering is a large and growing field of engineering. Up until the second half of the twentieth century, water supply was rather a simple field. Water was abstracted, preferably from a reservoir with a protected catchment, or from wells, and after generally minimal treatment was put into the distribution network. Some supplies were derived from rivers and these supplies required more treatment. However, where relatively poor-quality lowland rivers were used as a source, it was common to provide extensive storage prior to treatment, to allow self-purification of poor-quality water, and the subsequent treatment was often slow sand filtration, a process that is both robust and simple to operate.

Distribution systems were also simple. In the UK, we generally pumped water to water towers or service reservoirs located at high points, and from there, the water served the supply zones by gravity. There was little or no flow measurement, except at treatment works, and leakage was generally not a major concern, not least because leakage rates could not be determined. However, as far back as 1885, Alexander Binnie, a forefather of one of the authors, was working to introduce metered water-supply zones with measurement and recording of night flows in Bradford to allow wastage and leakage to be reduced (Binnie, 1885). Bradford used inspectors to visit houses to check water pipes and fittings.

The water-quality requirement was that the water be wholesome. In practice, the only quality standards were microbiological and regular water sampling and analyses were limited to microbiological examination and testing of chlorine levels. Universal chlorination of public drinking water was not introduced in the UK until after the Croydon typhoid outbreak of 1936.

The major concern during most of the twentieth century was to meet the ever-increasing demand for water. This was driven both by a steady increase in population served and by increasing per capita demands as consumers became ever more affluent and acquired indoor toilets, baths, washing machines, cars, and garden hoses. Water supply was a public service and was seen as a basic right. Public funding was available and used to construct the infrastructure to meet projected demands. Water supplies were generally un-metered and domestic consumers had no economic incentive to reduce water use; environmental concerns had not developed; and there was little public objection to the construction of additional infrastructure. The infrastructure was often of the highest quality, much of it constructed between the late nineteenth century and the mid twentieth century, and continues to serve water consumers today. A similar situation prevailed over much of Europe and other developed countries.

There were enormous changes over the period 1975–2005. In the UK, some of the factors responsible include

- the adoption of level of service indicators for public water supply – notably water-quality standards and standards for pressure of water supply
- the rationalisation of much of the water-supply industry into river catchment-based groupings
- the introduction of numerical standards for water quality, derived from the 1980 European Drinking Water Directive
- the privatisation of the water industry and the introduction of the associated regulatory structure
- the widespread introduction of water metering, now universal for industrial and commercial supplies and used for a significant and growing proportion of domestic supplies in England and Wales
- increasing environmental awareness among the general public leading to extreme difficulties in promoting major infrastructure works and in increasing the quantity of groundwater used in water supply
- the impacts of the Water Framework and Habitats Directives (Council of European Communities Directive, 1992; 2000) which are now being felt, particularly in new limits on abstractions from both surface sources and groundwater
- two major droughts affecting large areas of the country.

The consequence has been a near total transformation of the UK water-supply industry. Most other developed countries have had or are having a similar transformation in their water-supply industries.

What effect have these changes had on water engineers? Arguably the most important change has been the need to adopt a holistic approach to water supply. It is no longer possible or acceptable to consider the various components making up a water supply in isolation. The need for environmental protection and the difficulties of constructing new dams has meant that it is no longer possible to meet the water demands that would arise from an unfettered growth in demand. Thus, means have had to be adopted to reduce demands and to lower the amount of water lost through leaks. The adoption of extensive new physical and chemical water-quality standards, which apply at the point of delivery to the consumer, has meant not only additional water treatment, but also the close examination of water-distribution systems, to ensure that the water entering the distribution system does not deteriorate unacceptably as it travels to the point of use. Environmental considerations also impact on treatment processes used, with the problems and costs associated with the acceptable treatment and disposal of wastes arising from treatment becoming ever more significant.

Water supply in the developing world reflects some of the above, but with other problems derived from the particular financial, social and institutional constraints applying to each country. In particular, there are often very high rates of demand growth associated with increasing urbanisation, high rates of population growth and increased wealth, leading to increased ownership of water-using appliances. Common problems include: very high per capita water production, a shortage of economic water resources, and low water prices. These problems combined with a lack of metering and poor revenue collection, leads to waste by consumers and a lack of money for operating and maintaining systems.

In much of Europe, there may be complaints about the cost of water, but it is generally accepted that it is essential to have the highest quality water, almost regardless of cost. Notwithstanding this, in March 2011 it was stated by the EU, that for drinking water in small supplies (serving less than 5000 people) no more than 60% of the small water supply zones deliver water which is entirely compliant with the requirements of the Directive (according to a study produced at Commission's request). However, this is certainly not the case for the UK.

In poorer parts of the world such a philosophy is not possible and standards and the costs of meeting standards may have to be looked at in the context of limited resources and comparative risks. Whilst the derivation of risk-based standards is complex, in essence the World Health Organisation (WHO) has set guideline standards for carcinogens on the basis of one additional cancer case per 100 000 population receiving water at the guideline value over 70 years, and Australia has used a criterion of one additional case per million population (Australian Drinking Water Guidelines, 2004; WHO, 2006).

In countries with limited resources, and short life expectancies, it may be difficult to justify investing heavily in expensive water treatment in preference to other needs. For example, it may be preferable to invest in water distribution or in additional water resources. There may be more benefits in supplying a larger number of people with a lower quality of water than in supplying a smaller quantity of high-quality water to a limited population.

This book concentrates on water treatment, only a small part of the water-supply system. It is, however, most important for water-treatment engineers, scientists and managers not to adopt a blinkered approach but to look outwards and consider the entire system from source to consumer, paying due attention to wastes produced. Chapter 2 addresses water quality. Subsequent chapters are devoted to treatment of water and wastes from water treatment. In order to put water treatment in context, Chapter 14 considers the particular issues relating to private supplies. Chapter 15 considers water safety plans, which are now required for public supplies in England and Wales, and Chapters 16 and 17 consider some of the factors relating to the sizing of water treatment plants, water demands and demand management. Demand management is an increasingly important area as many countries approach the point at which additional water resources become scarce and increasingly difficult and expensive to exploit.

1.1. Structure and regulation of the water industry in the British Isles

Whilst this book concentrates on the technical aspects of water treatment, it is useful to consider the regulatory aspect of the industry, being the framework within which water is extracted, treated, delivered, and, finally, the wastewater disposed of. The following sections concentrates on the position in the British Isles, but water and the environment are areas for which various European Community Directives apply, and national legislation and regulations have to be consistent with European Directives.

1.1.1 England and Wales

Prior to privatisation in 1989, provision of water supply and sewerage was the responsibility of either the Regional Water Authorities (RWAs) or, for water supply only, privately owned water companies. The RWAs had three main areas of responsibility: water supply, sewerage, and the water environment. As well as being responsible for sewage treatment, they also issued consents for the discharges. Potable water quality was regulated to a very limited extent by a government ministry. The arrangement was considered to have a number of shortcomings: the RWAs responsibility for both sewage treatment and the water environment was seen as unsatisfactory; the RWAs were state owned, limiting their ability to invest in the infrastructure required; and EU regulations were about to drive a vast investment to improve both drinking water quality and the treatment of wastewater which needed to be funded whilst keeping charges down.

In 1989, the industry was privatised and the present structure was established. There have been some minor changes since then: the following summarises the current (2013) roles of the main players involved.

- Private-sector companies provide water and sewerage services, with ten large water and sewerage companies whose area of coverage is based on river catchments, and 12 water-only companies supplying water but not providing sewerage services. Since 1989 the number of water-only companies has reduced significantly due to take-overs and mergers, and this consolidation will surely continue.

- The government, the Department for Environment, Food and Rural Affairs (Defra) in England, and the Welsh Assembly in Wales, sets high-level policies and drafts relevant legislation and regulations.
- Companies are overseen by Ofwat the economic regulator. Ofwat is responsible for, amongst other things: setting charges paid by customers; monitoring relationships with consumers; agreeing and monitoring investment plans; collecting data on companies' performance and promoting economic efficiency and competition. Ofwat significantly reduced the quantity of data reported by the companies in 2012, in line with its stated aim of reducing the regulatory burden it places on companies.
- The Environment Agency (EA) regulates the various discharges companies make to the environment, mainly regulating discharges of treated sewage to water bodies, and is also tasked with promoting sustainable development.
- The Drinking Water Inspectorate (DWI) is responsible for ensuring that drinking water treatment and the water supplied to consumers complies with legal requirements. It also investigates water quality incidents and carries out inspections of companies.
- The Consumer Council for Water represents consumer interests and can investigate consumer complaints that have not been satisfactorily resolved by companies.

The complex structure of the industry is largely because the provision of water and sewerage services in a geographic area is a natural monopoly, and thus the regulatory structure has been designed to ensure that the private sector companies providing these services operate efficiently and are responsive to consumers' needs.

1.1.2 Scotland

In Scotland, the water industry has not been privatised. Responsibility for the provision of water and sewerage services lies with Scottish Water, a government owned company established in 2002. The regulatory structure in Scotland is similar to England and Wales with the main players shown below.

- Scottish Water, the government-owned company responsible for provision of water and sewerage services.
- The Drinking Water Quality Regulator for Scotland, responsible for ensuring that drinking water treatment and the water supplied to consumers complies with legal requirements. It also investigates incidents and carries out inspections of Scottish Water.
- The Water Industry Commission for Scotland, responsible for setting prices, monitoring performance, and facilitating competition, a role similar to that of Ofwats in England and Wales.
- The Scottish Environmental Protection Agency, the environmental regulator for Scotland, with a responsibility to protect and improve the environment.

- Consumer Focus Scotland, from 2011, became the advocate for water consumers in Scotland. Its task is to ensure that the needs of consumers are at the heart of the Scottish water industry. It has a range of statutory powers and duties.
- The Scottish Public Services Ombudsman has the final responsibility for handling complaints about public services in Scotland, including complaints about water and sewerage. It considers complaints after they have been through the formal complaints process of the organisation concerned.

1.1.3 Northern Ireland

The institutional arrangement in Northern Ireland is similar to Scotland. However, until recently, water and sewerage services were paid for through general taxes, and the full transition to all consumers, both domestic and commercial, paying for water and sewerage services through a water bill is still in progress, and will not be completed before 2013 at the earliest. These are the key bodies involved in water supply in Northern Ireland.

- Northern Ireland Water Ltd is a government owned company responsible for the provision of water and sewerage services in Northern Ireland. It was established in 2007.
- The Department of Environment has overall regulatory responsibility for water supply and sewerage in Northern Ireland. This includes licensing of any abstractions. The Northern Ireland Environment Agency is an Agency of the Department.
- Within the Environment Agency.
  1. The Water Management Unit which has responsibility for protection of the water environment, including the management of discharges.
  2. The Drinking Water Inspectorate which is responsible for regulating drinking water quality.
- The Consumer Council for Northern Ireland investigates complaints that have not been resolved by Northern Ireland Water and have been escalated up to the Council.

1.1.4 Ireland

Ireland currently has a very different system to the rest of the British Isles with the provision of water services largely being the responsibility of city and county councils. This includes the taking and analysis of water samples to monitor potable water quality. Ireland also has a significant number of local water supplies, both private and public. Approximately 10% of the population are serviced by privately run Group Water Schemes and this along with private wells and other supplies mean that one in five households in Ireland are not connected to the public mains supply.

The DoECLG (Department of the Environment, Community and Local Government) has the role of developing and implementing Government water supply policy, making sure that funding is made available, and monitoring physical and financial progress on schemes. Other responsibilities of the DoECLG include: ensuring good quality drinking water is available to all consumers and promoting water conservation.

The Environmental Protection Agency monitors potable water quality, using data provided by the water suppliers, and publishes annual summary reports. It also is responsible for issuing directions to ensure that water quality complies with standards, carrying out audits at water treatment plants, and has an oversight role across the sector. It also has power to prosecute where corrective action to address a deficiency is inadequate, resulting in the supply of unwholesome water.

The Irish water sector is to undergo a significant reorganisation. It is planned to establish Irish Water, this will be an independent state owned company. It will be responsible for providing water and wastewater services to homes and businesses connected to a public water supply. It is planned that Irish Water will take over the operation of local authority water services on a phased basis, starting in 2015. Currently domestic users are not specifically charged for water in Ireland and one of Irish Water's priorities will be to introduce water charges.

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Chapter 2
Quality of water

2.1. Origin and type of impurities

Absolutely pure water is never found in nature and it is increasingly rare to encounter a source of water that requires no treatment before being used for potable-water supply. Water contains both biological and inorganic matter. The matter found in water is affected by: the source of the water, impacted by the geology of the route taken from rainfall to point of extraction; vegetation and animal impacts; and human impacts, reflecting activities such as application of agricultural chemicals and waste discharges. It is normal to classify the impurities found in water in one of three progressively finer states – suspended, colloidal and dissolved. The method of treatment required for the removal of impurities, or their reduction to acceptable limits, depends in part on the fineness of the material. Whilst the matter found in raw water may render it unfit for human consumption, treatment can also adversely affect water quality by introducing pollutants, or by modifying chemicals that are harmless prior to treatment.

2.1.1 Suspended matter

Running water obviously will carry floating debris, but it also has the capacity to pick up and transport solid particles of greater density than water; the higher the velocity the bigger the particle that can be transported. Rivers are normally at their most turbid during flood, because of the increased water velocity. Table 2.1 indicates the sizes of solids that are transported at different velocities. Large rivers in flood often run at velocities in excess of the maximum shown in Table 2.1 and are capable of carrying high quantities of suspended material.

2.1.2 Colloids

Colloids are fine particles that do not settle and which are electrically charged. The particles have a similar electrical charge, normally negative, which prevents them from coalescing together to form larger settleable particles. They are invisible to the naked eye, but can impart colour and turbidity to the water.

2.1.3 Dissolved solids

In its passage over or through the ground, water may dissolve a wide variety of chemicals. Common cations are aluminium, calcium, sodium, potassium, iron, and manganese. Common anions are bicarbonate, chloride, sulfate, and nitrate. A soft upland water will

Table 2.1 Transportation velocities of particles (adapted from Fox, 1949)

Material	Diameter of particle: mm	Velocity of water: m/s
Fine sand	0.4	0.15
Medium sand	1.1	0.23
Coarse sand	2.5	0.3
Gravel	2.5–25	0.76
Shingle	25–75	1.2

typically have a total dissolved solids (TDS) of between 70 and 150 mg/l and a hardness of 30–100 mg $CaCO_3$/l. Lowland rivers have a TDS typically of between 200 and 400 mg/l and a hardness of 100–250 mg $CaCO_3$/l. Groundwater can have a TDS of up to 500 mg/l or even higher and a hardness of 400 mg $CaCO_3$/l or more. Gases also may be present in groundwater, particularly carbon dioxide (CO_2). Naturally occurring dissolved solids are rarely objectionable in low concentrations.

As well as dissolved solids derived from minerals, water may also contain man-made pollutants notably agricultural chemicals, such as pesticides or herbicides. Where these are present, concentrations normally vary over the year, dependent on times of application and rainfall/runoff. Although concentrations of these chemicals are typically very low, they can be very expensive to remove.

2.1.4 Organic pollution

Pollution from organic matter can be serious, particularly for groundwater sources that receive little treatment. Faecal pollution, whether from animals or humans is of particular concern, given the risks of disease transmission. The presence of ammonia, nitrates and nitrites, which are products of decomposition of organic wastes, indicates the possibility of faecal contamination. The presence of particular indicator bacteria is taken to confirm faecal contamination.

Water derived from peaty upland catchments can have high levels of organic colour, often associated with humic and fulvic acids. It is often difficult to treat such waters, and chlorination can lead to the production of trihalomethanes (THMs). Waters containing algae may also be prone to high levels of THMs after chlorination. Chapter 12 discusses the formation of THMs due to chlorination. Chloroform is the most common THM.

Water from lowland rivers generally contains man-made pollutants derived from domestic and industrial sources, such as contraceptive pills, plastics, and other materials and activities. This is the case even where wastes are treated prior to discharge: such chemicals can also be found in groundwater where there are waste discharges within catchments.

A particular concern to the public and the media is the potential or perceived presence of endocrine disrupters within the water environment, and by implication within the public potable water supply. An endocrine disrupter is a chemical that affects the function of the endocrine system in living organisms and consequently causes adverse health effects in either the organism, or its progeny. Thus, an endocrine disrupter may impact on fishes, say, rather than humans. There are hundreds of organic chemicals that have been identified as potential endocrine disruptors, and worrying stories about their potential impact on the aquatic environment.

The sources of endocrine disrupters are manifold, including biocides, pesticides, plastics, and birth control pills. Biocides may be in direct contact with water, notably in anti-fouling coatings; pesticides are applied to the cultivated land, and many disrupters are present in sewage, derived from humans or the substances used in houses and industries.

Endocrine disrupters are of interest to those responsible for setting potable water quality standards, as would be expected given their high profile. However, the consensus is that they are not of concern in potable water supplies in Europe as where they have been found; concentrations have been so low as to not impact on human health in any way. This is not to say that there are no concerns over their impact on the aquatic environment, in particular downstream of waste discharges, and this appears to be the major concern at present.

2.1.5 Algae and algal toxins

Algae are relatively slow-growing autotrophic micro-organisms which use nutrients present in water, carbon dioxide, water itself, and sunlight to grow. They are a problem because they are often difficult to remove from raw water, and a potential concern because some species produce toxins. In practice, the concentration of algae present depends upon the concentration of nutrients in the water, adequate sunlight, temperature, and the time available for them to grow. Thus, in unpolluted upland rivers, algal concentrations are minimal, but concentrations are higher in the lower reaches of rivers, where farming and sewage treatment works have introduced nutrients, particularly at times of low flow when there is less turbidity and a longer residence time. Generally, in the UK, algal concentrations in rivers are not a problem but, unless control measures are taken, algal concentrations in reservoirs are.

Where water is extracted from lakes or reservoirs, there can be a risk of the presence of algal toxins produced by blue-green algae (cyanobacteria). There are three classes of toxins potentially of drinking water significance.

- Hepatotoxins – which affect the liver.
- Neurotoxins – which affect the nervous system and may lead to respiratory arrest.
- Lipopolysaccharides – some of which are irritants.

Dermatotoxins are also produced which may cause severe dermatitis to swimmers. The greatest concern is compounds of the microcystin family. These are produced by several species of blue-green algae and have been implicated in several incidents of liver problems, mainly in Australia (Chorus and Bartram, 1999). For sources with a history of algal blooms, the Australian water quality guidelines (Australian Drinking Water Guidelines, 2004) require water suppliers to inspect raw water twice weekly at times of high risk and consider the need for toxicity testing when the concentration of algal cells exceeds 2000 cells/ml. WHO has proposed a provisional Guideline Value of 1 μg/l for total microcystin-LR (free plus cell-bound), and this has been adopted as the potable water standard by several European countries.

2.1.6 Microbiological parameters

The greatest short-term threat to human health from drinking water derives from pathogenic micro-organisms. A prime objective of water treatment is therefore to produce water free from such organisms. One approach would be to analyse for specific disease-causing organisms. However, there are difficulties and risks in cultivating such bacteria, and it is not practicable to analyse directly for viruses. The approach has therefore been to look for the presence of easily identified bacteria that are known to be present in human faeces, and to treat their presence as an indication of possible faecal contamination. If these indicator organisms are not present, then the water is assumed to be free of human pathogens and suitable for human consumption.

Traditionally there have been three or four main bacteriological parameters used to assess the quality of drinking water and alert water suppliers and the quality regulators to the potential presence of organisms pathogenic to humans. The parameters which are normally analysed are

- coliform bacteria
- faecal coliforms
- colony/plate counts
- faecal streptococcus.

The 1998 EU Drinking Water Directive (Council of European Communities Directive, 1998) included a standard for surface waters for the sulfate-reducing bacterium *Clostridium perfringens*, which is another indicator of faecal pollution: the appropriateness of this standard has been questioned and it may be removed from the next revision to the Directive. British standards did include a standard for *Cryptosporidium parvum* (known colloquially as crypto). This was introduced because in the 1980s there had been increasing concern over the presence of protozoan oocysts, notably *Cryptosporidium* and *Giardia lamblia*, both of which cause severe gastroenteritis: however this standard was later removed by a revision to the UK Regulations.

The potential transmission of viruses is a concern, but it appears that viruses are effectively inactivated by disinfection; and there is little evidence for viral transmission in properly treated water.

The significance of the above parameters is discussed further below. However, it is important to note that coliform bacteria are widely found in nature and do not necessarily indicate faecal pollution. The presence of either faecal coliform or faecal streptococcus is taken as evidence of faecal contamination. In practice, all surface water sources have coliforms present and most have faecal coliforms. However, groundwaters are often of high bacteriological purity. Plate counts give an indication of the overall level of bacterial activity in a sample and do not directly relate to faecal contamination.

2.1.7 Tastes and odours

Under certain conditions algae may be present in surface waters, particularly in water from reservoirs or lakes. These can lead to objectionable tastes and odours in treated water; these are particularly associated with chlorination. The removal of algae is essential and often difficult. Certain other substances may also cause taste even when present in incredibly low concentrations of as little as 1 μg/l. After chlorination, phenols may give rise to taste from chlorophenols when present in concentrations down to as low as 0.01 μg/l (Degremont, 1991).

2.1.8 Hardness

The soluble salts of calcium and magnesium commonly found in water cause hardness. Hardness forms insoluble precipitates with soap and requires more soap to be used to obtain lather; it also causes boiler scale. In the past, it was common to soften hard waters. However, there is a view that soft waters are associated with heart disease and thus softening is less common now. However, in the UK, there is sometimes a legal requirement to soften supplies to specific areas; such requirements are often associated with Acts of Parliament used in the past to promote water-supply schemes. Hardness and water softening are discussed in more detail in Chapter 11.

2.1.9 Iron and manganese

Iron and manganese impart colour, and can lead to staining of washing. Iron may be derived from raw water or from corrosion of iron water mains. Manganese is derived from raw water. These elements are common in some groundwaters, and may also occur in water taken from the lower levels of reservoirs. In the UK, both iron and manganese are removed by water treatment where necessary. Thus, the presence of iron in drinking water is therefore normally associated with corrosion of the distribution system. Where iron and manganese are found in drinking water, the cause is normally disturbances to the distribution system causing re-suspension of old deposits; typically, such events are due to pipe bursts, work on the distribution system involving changes in flow patterns, or the use of fire hydrants by the fire brigade or others.

2.1.10 Sulfates, chlorides, bromides, and fluorides

If present in excess, the sulfates of magnesium and sodium act as laxatives. Chlorides in concentrations above 600 mg/l tend to give the water a salty taste. Bromides have only recently become recognised as a potential raw-water-quality problem; this is because when water-containing bromide is ozonated, there is a risk of bromates forming, for which the allowable concentration is very low. Fluoride in low concentrations of approximately 1.0 mgF/l provides protection from tooth decay and fluoride is dosed in some areas for this reason. However, levels above 1.5 mg/l are undesirable and may affect bones and cause mottling of teeth.

2.1.11 Less common parameters

One of the effects of the introduction of increased numbers of standards combined with the introduction of regular sampling and analyses has been to identify a small number of UK supplies with unusual inorganic contaminants. Such problems relate normally to groundwater. The most common contaminants are probably arsenic and fluoride, but nickel has also been identified in some supplies. Arsenic present in groundwater in Bangladesh has caused major health problems due to water having elevated concentrations, with approximately 25% of shallow tubewells having concentrations greater than 50 μg/l, and concentrations of over 200 μg/l being found in almost 10% of shallow tubewells (Ahmed, 2003).

2.1.12 Corrosiveness and plumbosolvency

Waters with high carbon dioxide concentration, low pH value and low alkalinity are generally corrosive. This is of importance, as corrosive water will attack unprotected concrete. It can also attack materials used in the distribution network and domestic plumbing systems and can lead to elevated concentrations of iron, copper, and lead. A commonly used measure of whether water is corrosive is the Langelier saturation index (LSI). The LSI is the difference between the actual pH value of the water and the pH value at which the water would be saturated with calcium carbonate (pH_s). It, therefore, indicates whether particular water will dissolve or precipitate calcium carbonate. If the index is positive, the water will precipitate calcium carbonate, tending to protect pipework from corrosion. However, while the LSI is useful, it is only a very crude indicator of potential corrosion problems. Corrosion is far more complex than this.

Plumbosolvency refers to the tendency of water to dissolve lead, which is present in many older domestic plumbing systems: in lead pipes, in lead joints in cast iron pipes, and in the solder used to join copper pipes. Excessive lead concentrations are associated with a range of health problems, particularly in infants and young children, including a reduction of IQ (WHO, 1998), and as a result there is a requirement to greatly lower lead levels in water. Because of the tightening of lead standards in the 1998 EU Drinking Water Directive, it has become very common to adjust the pH value of water and dose phosphate-based chemicals to reduce corrosion. Among natural waters, soft, peat-stained moorland waters are generally corrosive, but many other waters also require treatment to reduce plumbosolvency in order to meet the lead standard.

2.2. River and source quality

In the early years of public water supply, many towns and cities established upland reservoirs within protected catchments. This ensured that the quality of the water in the reservoirs was bacteriologically good and that water from the reservoirs could be supplied directly to consumers at low risk. As water-treatment technology developed, the availability of potable-quality water at a source was seen to be less important, and abstraction points moved steadily downstream, but with full treatment. The development of water-quality standards has resulted in standards for many organic chemicals found in low concentrations. This has led to increased treatment costs and a renewed appreciation of the importance of high-quality sources for drinking water. It has also led to the realisation that it can be more economic to control the pollutants entering the aquatic environment than to treat water to remove them. Thus, the use of a herbicide that is less effective, but that rapidly and harmlessly degrades, may have a lower overall cost to society and may be preferable to a more effective but persistent alternative that requires expensive water treatment to remove it. This sort of calculation is increasingly being done in the regulation of agricultural and other man-made chemicals.

River quality can be assessed by chemical-based parameters, including biological oxygen demand, and by biological indicators that take into account the presence of indicator organisms and the diversity of life found in water. Attempts have been made to relate source water quality to the degree of treatment required. A 1975 EU Directive (75/440/EEC), now repealed, defined three categories of water and specified that

- category A1 water required simple physical treatment and disinfection
- category A2 water required normal physical treatment, chemical treatment, and disinfection; and
- category A3 water required intensive physical and chemical treatment, extended treatment, and disinfection.

Classification was mainly on the basis of chemical and microbiological quality, without any biological indicators. Whilst this rigid approach has fallen from favour, replaced by a less prescriptive risk based assessment, it does represent a starting point for specifying the degree of treatment required for a new source.

2.2.1 Water Framework Directive

In December 2000, the EU introduced a Framework Directive (2000/60/EC) to cover EU policy in relation to all waters, including groundwater, rivers, estuaries, and coastal waters. It sets environmental objectives and a target date of 2016 to achieve them. Member States have to prepare river basin management plans for all basins. The new Directive repealed seven existing Directives including that referred to above which classified waters used for drinking water abstraction. The long-term implications for water supply are still uncertain, but two areas stand out: there will be an improvement in the quality of some rivers, making them more suitable for use for public water supply; and the Directive will lead to some restrictions on abstractions where they adversely affect the quality or ecology of waters.

2.3. Potable-water standards

For much of the twentieth century, the key requirement for water quality in the UK was that the water should be wholesome. In practice, this meant that the key standards were that the water be free of coliform bacteria and in particular *Escherichia coli*. Development of prescriptive water-quality standards, notably by the WHO, covering a range of physical and chemical quality parameters for drinking water, started in the 1950s and still continues. Additional parameters have been added as knowledge was gained on the health effects on humans of chemicals in water, and also as analytical techniques developed, allowing the identification of large numbers of organic chemicals in water. At the time of writing, in Europe there is a perception in the water industry that the increasing proliferation of standards and analyses needs to be managed better. There are signs of a move to reducing the number of parameters analysed for all supplies, with analyses of additional parameters required on a case-by-case basis for individual supplies determined from risk analyses and water safety plans.

2.3.1 WHO standards

The first edition of WHO's *International Standards for Drinking Water* was published in 1958, and the second edition in 1963. The 1963 standards set coliform standards for microbiological purity, health-based maximum allowable concentrations for seven metals, and maximum acceptable and maximum allowable concentrations for a further 18 parameters. They also considered acceptable limits for fluoride and nitrate, without setting any standards, and suggested radiological limits.

In 1984 and 1985, WHO first published *Guidelines for Drinking Water Quality*. These stepped back from setting maximum allowable concentrations; rather they proposed guideline values to ensure aesthetically pleasing water posing no significant risk to health. The guidelines were published in three volumes, with the basis of the guideline values made clear in the supporting data. The introduction to the guidelines made it clear that the first priorities were to make water available and to ensure its microbiological quality. The 1984 *Guidelines* proposed guideline values for 43 parameters.

In 1993, the first volume of the second edition of the *WHO Guidelines* was published (WHO, 1998). This recommended new guideline values, again based on the need to provide aesthetically pleasing water providing no significant risk to health. The number of parameters for which guideline values were given increased to around 140, including approximately 90 organic chemicals of health significance.

WHO published the third edition of WHO *Guidelines for Drinking-Water Quality* in 2004, with an addendum in 2006 (WHO, 2006). This reviewed and updated guideline values for individual chemicals, and added others, in response to available scientific information. Summary statements were included for specific pathogens and individual chemical hazards of actual or potential concern. The third edition also includes more information on application of the *Guidelines* in specific circumstances (e.g. small community supplies, in large buildings, in emergencies and disasters). The fourth edition was published in 2011 (WHO, 2011). The current published version of the *Guidelines* is available on the WHO website. WHO has aimed to provide transparency as to the logic behind the guideline values and thus they are supported by extensive documentation and references. The WHO does not define guideline values for parameters with no direct health significance or where insufficient data are available, however such parameters are discussed in the document and guidance is generally provided. The recent tendency has been to reduce the number of guideline values for inorganic parameters.

The 1963 WHO standards were of great importance in the development of water-quality standards in much of the world. They were arguably the first standards widely used by engineers and governments to set quality standards for new water-treatment plants and as the basis of national water-quality legislation.

2.3.2 European standards

In the UK and the EU, the defining moment in the development of prescriptive water-quality standards was the adoption of the 1980 Drinking Water Directive (80/778/EEC). This set the minimum water-quality standards to be adopted by the member countries, a population of over 350 million. It also set sampling and analytical frequencies, to check whether the standards were being met. The Directive covered 62 parameters, divided into microbiological, toxic substances, substances undesirable in excessive amounts, physico-chemical parameters, and organoleptic parameters. For the majority of the parameters, it set both guide levels and maximum allowable concentrations, but for toxic substances and microbiological parameters only maxima were set, and for some parameters only guide levels were given. The Directive allowed limited latitude to national governments in respect of relaxations of standards, although governments were able to impose more stringent standards. The Directive applied to public water supplies, and not to bottled water.

The 1980 Directive was responsible for an enormous improvement in both the quality of drinking water across Europe and in the data available on drinking water quality. It was also responsible for a vast investment to meet its standards. The 1980 Directive was the subject of some criticism; notably, that it over-regulated some harmless substances, such as silica; that it set some standards, notably for pesticides, based on ideals rather than an assessment of health risks; and that it set statistically unsound absolute standards. More recently, it became increasingly apparent that it was too lax for some parameters, notably lead. By the early 1990s, pressure was mounting for the Directive to be revised and after much discussion a revised Directive was adopted.

The revised Directive on the quality of water intended for human consumption was adopted in 1998 (98/83/EC) (Council of European Communities Directive, 1998). The new Directive adopted the principle of subsidiarity, focusing on essential health and quality parameters and allowing a degree of latitude on other parameters. The Directive divides the quality parameters into three parts: part A sets mandatory microbiological parameters, part B sets mandatory chemical parameters, and part C sets indicator parameters, chemical, physical, microbiological, and radioactivity. Part C contains many parameters that were previously mandatory. The part C parameters are set for monitoring purposes and a failure to meet these standards need not require remedial action unless there is a risk to human health. The Directive allows only a limited period for derogations (which are effectively relaxations of the mandatory standards).

The 1998 Directive represents a dramatic change in focus. The 1980 Directive had led to a large and on-going investment to meet the standards for iron, manganese, and aluminium. Under the new Directive, not all of this investment would have been mandated. The 1998 Directive allows European governments to relax water-quality standards in some areas. However, it seems unlikely that this will be politically acceptable. In other areas, there is a significant tightening of standards, most notably for lead and bromate. The 1998 Directive covers bottled water as well as piped supplies. Table 2.2 compares the old and new European Drinking Water Directives for some key parameters and Appendix 2 provides more detail. As of late 2012, there appears to be no immediate prospect of a further revision of the Drinking Water Directive with respect to quality criteria before 2016. Any revision would add quality standards for a small number of additional parameters, and should emphasise the importance of risk analyses and water safety plans in deciding which parameters should be monitored for a particular supply.

2.3.3 UK and Ireland regulations

It should be noted that England, Wales, Scotland, Northern Ireland, and Ireland all have their own separate regulations covering potable water quality. All have to incorporate the requirements of the 1998 European Drinking Water Directive into national law. Up-to-date water quality regulations are on the websites of the national regulators.

Table 2.2 Comparison of WHO, European, and USA drinking water-quality standards (all values in μg/l unless otherwise indicated)

Selected parameter	WHO guideline value (2012)[a]	EU 1980 (80/778/EEC)	EU 1998 (98/83/EC)[g]	US Federal Standards (state standards may be tighter)
Turbidity (NTU)	None set[c]	4	Acceptable (C)	1[f]
Colour (mg/l Pt/Co)	None set[c]	20	Acceptable (C)	15[d]
Lead	10	50	10 (B)	15[e]
Iron	None set	200	200 (C)	300[d]
Manganese	None set	50	50 (C)	50[d]
Aluminium	None set	200	200 (C)	50–200[d]
Cadmium	3	5	5 (B)	5
Arsenic	10	50	10 (B)	10
Nitrates (as NO_3)	50 mg/l	50 mg/l	50 mg/l (B)	62 mg/l
Sulfates	None set	250 mg/l	250 mg/l (C)	250 mg/l[d]
Bromates	10	–	10 (B)	10
Cyanide	None set	50	50 (B)	200
Lindane	2	0.1[b]	0.1 (B)	0.2
Aldrin	0.03[h]	0.1[b]	0.03 (B)	–
Vinyl chloride	0.3	–	0.50 (B)	2
Bromoform	100	–	Total THM 100 (B)	Total THM 80
Chloroform	300	–	Total THM 100 (B)	Total THM 80

NTU, nephelometric turbidity units

[a] WHO has only set guideline vales where these are merited for health reasons. For other parameters there may be aesthetic reasons for achieving a low value but no guideline value is set

[b] All individual pesticides have a limit of 0.1 μg/l. The total concentration of all pesticides must not exceed 0.5 μg/l

[c] For turbidity WHO state that no more than 1 NTU is required for effective disinfection. For colour, WHO suggest less than 15 True Colour Units is acceptable

[d] Secondary standard – non-enforceable although individual states may have as an enforceable standard

[e] Action level – if exceeded by more than 10% of samples, treatment is required

[f] Surface water or groundwater under influence of surface water requires filtration such that no sample should exceed 1 NTU and 95% of samples must be below 0.3 NTU

[g] 'B' indicates mandatory chemical parameter, and 'C' indicates indicator parameter

[h] The WHO guideline value is for aldrin and dieldrin combined NTU, nephelometric turbidity units

The current (2013) Regulations for each country are listed below.

- England and Wales: *The Water Supply (Water Quality) Regulations 2000* – these have been amended by a number of subsequent Statutory Instruments, most recently in 2010. A consolidated version of the current Regulations is provided on the Drinking Water Inspectorate's website.
- Northern Ireland: *Water Supply (Water Quality) Regulations (Northern Ireland) 2007.*
- Scotland: *The Water Supply (Water Quality) (Scotland) Regulations 2001.*
- Ireland: S.I. No. 106/2007 – *European Communities (Drinking Water) Regulations, 2007.*

The English and Welsh Regulations are based upon the 1998 Directive, but carry forward some standards based on the 1980 Directive. The 2000 Regulations included a standard for cryptosporidium that was removed by an amendment in 2007.

As well as the standards set out in the Regulations, there are also paragraphs in the UK Regulations that require materials and chemicals used in drinking water supply to be approved. In England, this is done by a list of approved products and processes (UK Drinking Water Inspectorate, 2012). This approach has been used to set standards for the concentrations of some chemicals arising from water treatment additional to those in the Regulations. An example is the limit on by-products of chlorine dioxide generation. In England and Wales, this list is derived from the requirements of the 2000 Regulations.

2.3.4 USA standards

The US standards are the current *National Primary and Secondary Drinking Water Regulations* (USEPA, 2012). Primary standards are national legally enforceable standards that regulate contaminants that can adversely affect public health. Secondary standards relate to aesthetic parameters and are non-enforceable nationally although individual states may adopt them as enforceable standards. The US standards include the concept of a legally enforceable 'treatment technique'; this applies to several parameters that it is difficult to set absolute standards for, including lead, copper, acrylamide (polymer), and *Giardia lamblia*.

2.3.5 Comparison of standards

Table 2.2 compares the WHO, US Federal, and EU water standards for a selection of commonly found contaminants. UK standards are based on the 1980 and 1998 European Directives. WHO has only defined guideline values where there are health implications and adequate data are available: examples of parameters with no guideline value are aluminium, where insufficient data were available, and colour and turbidity, where there are no direct health implications. *WHO's Guidelines* does consider parameters with no guideline values and effectively propose values but it would be misleading to enter these in the table. Since the third edition of this book, there has been some convergence in standards, with a consensus emerging for most parameters. Appendix 2 contains further details of these standards.

2.4. Analysis of parameters

As well as leading to great improvements in water quality, the recent regulations have led to a massive increase in the numbers of samples analysed. This, together with increasing cost pressures, has led to a transformation in analytical techniques over recent years. Nowadays, virtually all analyses are carried out by automated equipment.

2.5. Key water-quality parameters

2.5.1 Turbidity

Turbidity is a measure of the amount of scattering that occurs when light passes through water. Although it is caused by the presence of fine suspended solids in the water, it is not directly related to the concentration of suspended solids. The former is a measure of the total weight of dry solids present, whereas turbidity is an optical effect, stated in turbidity units, which also reflects the fineness, colour, and shape of the dispersed particles. For this reason, there is no constant linear relationship between turbidity and suspended solids.

There are several methods of measuring turbidity and, in the past, the results were frequently expressed in different units. Nowadays, nephelometric turbidity units (NTU) are almost always used. Turbidity values in NTU are for all practical purposes interchangeable with those expressed as Jackson turbidity units (JTU), formazin turbidity units (FTU), or the APHA turbidity unit.

2.5.1.1 Methods of measurement

There are a number of types of turbidimeter, most of which are now only of historic interest. In the past, measurement of turbidity meant using a Jackson turbidimeter. In this method, a standard candle is viewed through a column of water under test, the length of which is increased until the flame disappears. The length of the column defines the degree of turbidity, which is stated in JTU. Later, these were calibrated by using suspensions of formazin polymer with the results stated in formazin units. These units were widely used until superseded by instruments reading directly in NTU. NTU and JTU can be taken as numerically equal.

Turbidity is measured in NTU by directly measuring the scattering of light in a laboratory instrument or an on-line continuous monitoring instrument. These involve shining a light of specified wavelength through a sample of water in a cell through which water is continuously passed, and they immediately record turbidity. It is now normal practice to have several on-line turbidimeters at water-treatment works to continuously monitor raw, filtered and treated water. A practical problem with many on-line turbidimeters is fouling of the cell through which the water passes; as the cell becomes dirty, the instrument becomes inaccurate. This can be avoided by shining the light through a falling column of water, avoiding fouling, but requiring precise control of the column of water.

2.5.2 Colour

Colour is a measure of the light absorbed by the water. True colour excludes the effects of any scattering of light due to turbidity. Colour in drinking water is normally measured by a comparison method. This defines a unit of colour, equal to a degree Hazen (°H), as that produced by 1 mg of platinum/l in the form of the chromoplatinate ion. Colour is measured by comparing the sample against a set of standard colour solutions. There are also automated instruments which measure colour directly. Normally, colour is very pH dependent and it is good practice to also report pH value.

There is also a spectrophotometric method that measures colour more precisely in terms of light wavelengths; but this is only used in water treatment for research.

2.5.3 Pesticides and other organic chemicals

The standards for many organic chemicals have been derived in parallel with the development of powerful analytical techniques. As techniques have developed, more and more chemicals have been found to be present in water, and standards have been developed to cover many of these chemicals. There are clear pressures on water regulators associated with: the presence of many organic chemicals introduced into the water environment by human activity; our increasing ability to analyse for minute concentrations; the ability of water treatment processed to remove such chemicals; and concerns over potential health impacts. Newspapers and the internet report the presence of a whole range of chemicals in rivers and lakes and concern over potential impacts. In most instances, the potential impacts are uncertain and it seems unlikely that there will be major impacts on either potable water quality standards or on water treatment processes, but this is a difficult area.

An early concern was pesticides; the 1980 EC Drinking Water Directive specified a blanket limit of 0.1 μg/l for all pesticides; this was intended to be effectively a zero limit. In 1980, the standard was below the level of detection for many pesticides, and this led to the development of analytical methods to measure pesticides at such low concentrations. These techniques were also appropriate to the analysis of many other complex organic chemicals. As more data has become available the health risks of most pesticides are better understood, and health-derived standards can be set for most pesticides, often higher than the European standard. A notable example is the pesticide metaldeyhde, used for killing slugs and snails. This has been found in water supplies in the UK at concentrations well above the maximum allowable concentration of 0.1 μg/l but well below the concentration that would be justified by health considerations. It is not easily removed by current water treatment processes and it appears that the most sensible options are strict controls on use or a relaxation of the current standard: at the time of writing this is an on-going problem for water suppliers, quality regulators, and legislators.

The standard methods for analysing many of the organic chemicals present in water at μg/l concentrations involve extraction and concentration of the chemical from water followed by gas chromatography or mass spectrometry. The equipment is complex and these analyses are carried out using automated equipment.

2.6. Microbiological parameters

2.6.1 Coliform bacteria

These are rod-shaped bacteria which are widely found in the natural environment. Water-quality testing is particularly interested in those bacteria that may be derived either from the human gut or from the gut of other warm-blooded animals, which thus prosper under the conditions found in the human digestive system. This is an acid environment at a temperature of approximately 37°C.

Coliform bacteria are defined based in part on the method historically used to detect them. They are rod-shaped bacteria that ferment lactose, forming gas and acid, in the presence of bile salts at a temperature of 35–37°C within 48 h. The old method of analysis used multiple dilutions of the water being tested, which were incubated in tubes. The number of positive tubes at the different dilutions gives the most probable number (MPN) of bacteria present. The result was expressed as a number of coliforms per 100 ml. The test was labour-intensive and has largely been superseded by methods using membrane techniques that filter out bacteria and then grow colonies of coliform bacteria, allowing a direct estimate of the number of coliforms present in a sample. However, these membrane techniques are in turn being replaced by enzyme substrate methods that permit simple analyses of both coliform and *E. coli* organisms; these are discussed further below.

2.6.1.1 Escherichia coli (*E. coli*)

If coliform bacteria are present, it is necessary to distinguish those that may derive from other sources from those that could only derive from the gut of warm-blooded animals. The *E. coli* test confirms that the coliforms are of faecal origin. *E. coli* or faecal coliforms are bacteria that can produce gas from lactose at a temperature of 44.5°C. Their presence is taken as confirmation of faecal contamination. Again, both multiple tube methods and membrane filtration methods can be used, and both methods are being replaced by enzyme substrate methods for routine analyses.

As coliform bacteria can arise from sources other than human faecal contamination, the presence of such bacteria is permissible in a small proportion of samples, normally no more than 5% of samples. The presence of *E. coli* is unacceptable and none should be detected in public potable-water supplies. Where coliform bacteria are found, it is usual to then check whether they are *E. coli* by additional analysis. Some suppliers outside of the UK sample from customers' external taps, and where this is done regular positive analyses can be expected for *E. coli*, due to contamination of the sample tap.

2.6.2 Enzyme substrate analyses (Colilert®)

The method involves the use of commercially available substrate formulations that are used as a substrate by coliforms and *E. coli* bacteria. If coliforms are present, a yellow colour is produced, and if *E. coli* are present, a fluorescent chemical is also produced. The formulation is added to dilutions of the water being tested, which are then incubated and examined under normal and fluorescent light to confirm the presence of coliforms and *E. coli* bacteria. The normal procedure is to put the water and reagent into a plastic disposable tray containing a large number of cells. The tray is sealed and incubated for around 24 h. Counting the number of positive cells, either yellow or yellow and fluorescent, can estimate the number of coliforms and *E. coli* using MPN techniques.

Such methods allow rapid and easy enumeration of bacteria; water companies now commonly use them for day-to-day monitoring of drinking water quality. The great advantage of this method is that, if coliforms are present, it avoids the need for further confirmatory analyses to test for *E. coli* (Graham, 1999).

2.6.3 Plate counts

Plate counts examine the total number of viable bacteria by mixing a sample of water with a medium and then allowing the mixture to incubate at a temperature of either 22°C or 37°C. The plate will develop colonies, indicating the number of viable bacteria and these can be counted. The test is useful for detecting changes in microbiological quality. Values found in tap water will vary from place to place but are normally consistent and low, typically 10–100 colonies/ml. Changes can indicate pollution of the distribution system, lack of chlorine, or a water-treatment failure. No standards are normally set for plate counts, but, in the UK, they are a required test on statutory water-distribution samples.

2.6.4 Faecal streptococci

These are species of bacteria that have been identified in the guts of warm-blooded animals. A particular subgroup is the *enterococcus* group that is referred to in the EU Water Directive. The prefix 'faecal' indicates that they can thrive in conditions present in the human gut. They are generally accepted to be more persistent than *E. coli* and coliform bacteria, and may indicate remote pollution. Faecal streptococci can grow at 45°C in concentrations of bile and sodium azide which inhibit coliform organisms.

Analytical methods are based on this. Membrane filtration methods are mainly used nowadays.

2.6.5 *Cryptosporidium parvum*

This is a very persistent protozoa that gives rise to severe diarrhoea which cannot be medically treated. It is extremely difficult to detect and determine the concentration of the organism in water. In 1999, the UK introduced a quality standard for *Cryptosporidium parvum* of no more than one organism in 10 l. This standard was removed by the 2007 revisions to the Regulations but it is likely to continue to be considered an informal benchmark.

2.6.6 *Clostridium perfringens*

This is a bacterium associated with faecal pollution. A standard of zero was introduced in the 1998 EU Drinking Water Directive. The standard was introduced as an indicator of faecal pollution, as a surrogate that could indicate the presence of pathogens resistant to disinfection.

2.7. Conclusions

While the development of water-quality standards over the past few decades has led to greatly improved water quality in the richer countries, it can also be argued to have had some negative impact. In the past, there were no hard and fast rules as to the acceptable quality for potable supplies, but clearly the water had to be wholesome. The position now is very different over much of the world; usually there are now extensive standards. In the richer countries, this has led to a vast investment in water treatment to meet the often very strict water-quality criteria. In poorer countries, the development of water-quality standards has arguably diverted funds into a limited number of modern-treatment plants, while water in large areas of many poorer countries receives only very limited treatment and water fails to meet even the most essential microbiological parameters. The introduction to the 1993 WHO *Guidelines for Drinking Water Quality* stressed the importance of prioritising water-quality standards and the absolute priority that should be given to parameters affecting public health. However, aesthetic parameters are also important in that if water is unpalatable, consumers may use alternative unsafe but more palatable supplies.

A very significant proportion of the recent investment in water treatment in the UK has been to meet the standards for organic chemicals introduced into the aquatic environment by human activity, most notably pesticides. A figure of US$2.25 billion was cited (Fawell and Miller, 1992) in 1990 as the cost per life saved in meeting the USA drinking water standard for the pesticides simazine/alachlor. While this figure may well be inaccurate, it does highlight two important points: it is very expensive to treat for some chemicals once they are in the environment; and over much of the world meeting such standards would represent a misallocation of resources.

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Chapter 3

Overview of water treatment

Arguably, it is possible to produce potable water from virtually any source of water. In practice, of course, there are a number of restrictions on the quality of water that can be treated. The usual reason for not using a particular source is cost but there are other reasons, notably ecological/environmental constraints and aesthetics – direct potable re-use of wastewater is usually unacceptable at present, although it has been practised at Windhoek in Namibia since the late 1960s, as discussed below. Although ecological and environmental constraints can also be valued, aesthetic objections tend to have a value above mere money.

There is a whole range of treatment processes that can be used to treat a particular type of water. The actual processes selected will depend on the nature of the raw water, the space available to construct the plant, a consideration of operating costs and capital costs, and, often, personal or company preferences. There may well not be a single correct process train to treat particular water, but a number of possible options. Often engineers are further restrained by the need to upgrade an existing plant, resulting in further limitations on the processes that can be used.

This chapter considers water treatment under three main headings: traditional; modern; and the future. There are also brief considerations of developing world treatment, highlighting some of the different considerations, and water re-use. The classification of processes under such headings is obviously subjective, reflecting the authors' age and experience, but it does offer a convenient way to sub-divide the various processes. In practice, water suppliers increasingly provide what is perceived to be the most economic treatment process within the existing physical constraints, taking account of risks and reducing them to an acceptable level. Mixes of traditional and advanced treatment will be provided to treat particular problems. One important issue is how risks are handled; it may be acceptable to a particular company or country to design a plant with a higher risk of failing particular quality standards than to incur the additional costs of reducing the risk. As the Water Safety Plan approach, discussed in Chapter 15, becomes more widely used, the processes used at water treatment plants will need to be focused on the significant water quality risks, particularly those with health significance. In the UK, the quality of the treated water is paramount, with only a low level of risk acceptable, but in poorer parts of the world financial pressures may mean that higher levels of risk have to be accepted.

3.1. Traditional water treatment

What processes can be classified as traditional water treatment? For surface waters, conventional water treatment consists of some or all of the following main processes

- catchment control
- raw-water storage
- removal of coarse solids by screening
- sedimentation
- aeration (also common for groundwaters with high levels of iron)
- chemical dosing
- coagulation and flocculation
- clarification
- slow sand filtration
- rapid gravity filtration
- pressure filtration (also common for groundwaters)
- chlorination (also common for groundwaters)
- chlorine contact tank (also common for groundwaters)
- settlement and recycling of filter washwater
- disposal of sludge to lagoons
- dewatering of sludge.

For groundwaters, the main problems tend to be elevated levels of iron and manganese and most groundwater treatment plants treated for these parameters. Less commonly, groundwater treatment plants softened waters.

In the past, the core of most water-treatment plants comprised settlement/clarification basins and filters. These arguably remain the most important processes, but with the development of membrane-based processes their use is no longer essential.

Most water-treatment works constructed in the UK before the 1970s were unlikely to have processes other than those listed above. The particular process used varied from works to works and there were often important differences between works using similar processes. For example, rapid gravity filtration could be conventional sand filtration, dual media filtration, or even upward-flow filters. The processes are predominantly, although by no means solely, physical. For many relatively unpolluted sources such treatment produced water of acceptable quality, reducing turbidity, colour, suspended solids, and iron and manganese to acceptable levels and producing bacteriologically acceptable water.

Slow sand filtration became important partly as a result of the English Metropolis Water Act of 1852 (1852). This prohibited the extraction of water from the tidal Thames and required that all surface-derived water supplied to London should be treated by sand filtration. This requirement gave an enormous boost to the construction of water-treatment plants to serve London. Despite this, it is surprisingly recently that there has been a move towards universal treatment of water, other than by disinfection. At the time of writing there are still cities in the British Isles (e.g. Manchester) that receive some surface water from protected catchments that has essentially had no treatment other than disinfection, and in the USA most of New York's water is untreated save for disinfection, fluoridation, and corrosion control (Nickols, 2000).

The supply of untreated water was predicated on the idea that collecting water from controlled catchments would ensure a safe water of satisfactory quality. Thus, catchment control involved limiting human access to, and prohibiting most agricultural activity in, the catchment. The view that catchment control alone provides water that is safe after disinfection is no longer acceptable; there have been too many outbreaks of waterborne diseases associated with protozoan parasites, specifically *Giardia lamblia* and *Cryptosporidium parvum*. Furthermore, the introduction of absolute quality standards for colour, turbidity, iron and manganese means that untreated water may be unacceptable on account of failure to meet these predominantly aesthetic parameters. In addition, there is increasing pressure for recreational access to protected catchments. The supply of surface water untreated save for disinfection has become very uncommon.

3.2. Modern water treatment

In parallel with the development of water-quality standards, water-treatment processes underwent a period of development. In the UK and Europe, the adoption into national laws of the water-quality standards in the first (1980) EU Drinking Water Directive led to the introduction of new treatment processes and the further development of traditional treatment processes to meet these standards. Modern treatment processes include

- improved coagulation control
- dissolved air flotation (DAF)
- advanced clarifiers (lamella separators and advanced 'sludge-blanket' systems)
- ozonation
- granular activated carbon (GAC) adsorption
- membrane-based processes
- air stripping of volatile organic chemicals
- advanced disinfection (ultraviolet, ozonation, and chlorine dioxide).

Most of the above processes are used to deal with organic chemicals present in water, either naturally occurring compounds that give rise to colour, taste, or chlorination by-products, or pollutants arising from human activities, notably pesticides and herbicides. In addition, the introduction of computer-based systems has led to great improvements in the monitoring and control of the traditional processes, leading to higher and more consistent quality of water produced from traditional plants.

3.3. Overview of processes and process selection

Table 3.1 lists the main current water-treatment processes and some specific water-quality problems for which the processes may be used. The table indicates the processes that may be applicable for specific water-quality problems. Typically, for any water-quality problem, there are options that can be used, depending on particular circumstances. This table is not intended as a guide to process selection as the real world is more complex. It is always essential to approach water treatment with an open mind, looking at the particular problems of the water to be treated; the amount of money available; plants treating similar water; and, where a plant is to be upgraded, the performance of the existing plant. It is also often essential to consider carefully the best source of the water to be treated if the most economic solution is to be proposed; it may be that by

Table 3.1 Main water-treatment processes and water-quality problems addressed

Process	Quality problems[1]													
	1	2	3	4	5	6	7	8	9	10	11	12	13	14
Coarse screening	×													
Fine screening	×													
Microfiltration	×											×		
Raw-water storage		×								×				
Preliminary settlement		×												
Aeration					×	×								
Air stripping									×					
Coagulation and flocculation			×	×		×				×		×		
Gravity clarification			×	×		×				×		×		
DAF			×	×		×				×		×		
Slow sand filtration			×	×		×				×		×		
Rapid gravity filtration			×	×		×				×		×		
Microfiltration			×	×						×		×		
Ultrafiltration				×						×				
Reverse osmosis							×	×	×		×			
Activated carbon adsorption				×	×			×						
Pre-ozonation				×	×			×	×			×		
Post-ozonation								×	×					×
Ion exchange							×							
Chemical oxidation						×								
pH control						×							×	×
Phosphate dosing													×	
Chlorination						×								×

[1] 1, debris; 2, high-sediment load; 3, turbidity; 4, colour; 5, taste and odour; 6, iron and manganese; 7, nitrate; 8, pesticides; 9, volatile organic chemicals; 10, *Cryptosporidium*; 11, salinity; 12, algae; 13, plumbosolvency; 14, microbiological quality.

carrying out works at the source or by modifying an intake the quality of the raw water can be improved to reduce the treatment needed.

3.4. Typical process streams

Given that there are many ways to treat a particular water, what processes are applied in practice? Figures 3.1–3.4 are typical process streams for a selection of waters: again these should not be considered as prescriptive.

Figure 3.1 represents the treatment water from the lower reaches of a large English river such as the Thames or the Severn. Such water requires extensive treatment to remove, in particular, the pollutants derived from upstream human activities and the treated wastewater that has been discharged upstream. Despite the number of processes shown, the water is normally relatively easy to treat.

Figure 3.2 is an alternative approach to Figure 3.1, using bankside storage of raw water and slow sand filters.

Figure 3.3 represents a possible treatment process for water extracted from the upper reaches of a river draining upland moors. Such waters are likely to be low in pollutants derived from human activities, but could well be soft with high natural colour and be of concern with respect to *Cryptosporidium* oocysts. These waters are difficult to coagulate, forming a light floc that does not settle well. Chlorination may lead to trihalomethanes (THM) formation and is delayed as long as possible. If the source was assessed at high risk of animal-derived *Cryptosporidium* oocysts being present in high numbers, the poor-coagulation characteristics of such waters may lead to the addition of an extra treatment stage specifically for *Cryptosporidium* removal.

Figure 3.4 represents a possible treatment process for water extracted from boreholes or reservoir waters containing iron and manganese. The key problem is to remove both the iron and manganese, which require different chemical environments for most effective removal. The process shown includes two-stage filtration, but this is not always necessary, as discussed in Chapter 11.

3.5. Raw- and treated-water quality

Conventional water treatment is appropriate for most surface or groundwaters that are free of serious organic contamination derived from human activities, as well as for many waters that are not. Conventional treatment can have difficulty treating some waters, notably soft, coloured waters derived from moorland and bogs. The difficulty of treating such waters has become more important as water-quality standards have developed. Thus a conventional treatment plant might well have produced water that had long been considered potable, but might now no longer be acceptable because of, for example, a failure to comply with the colour or THM standards.

Figure 3.1 Treatment schematic diagram for lowland river water

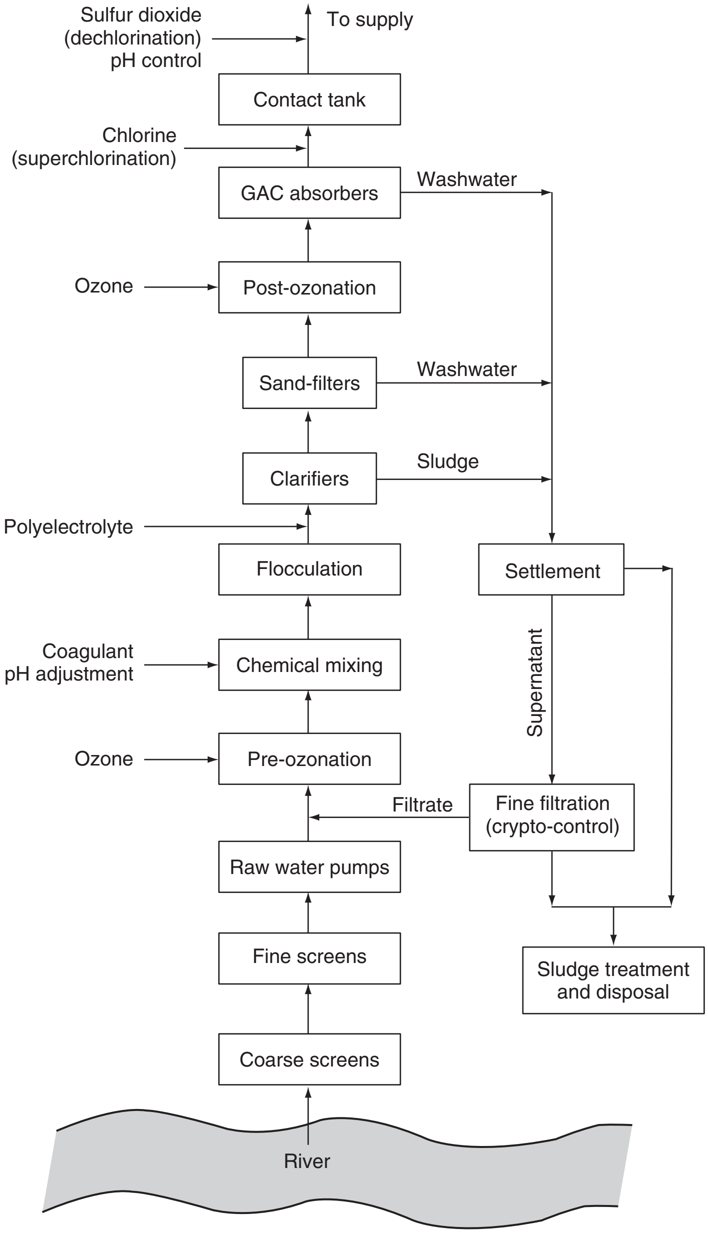


Figure 3.2 Treatment schematic diagram for lowland river water using slow sand filters

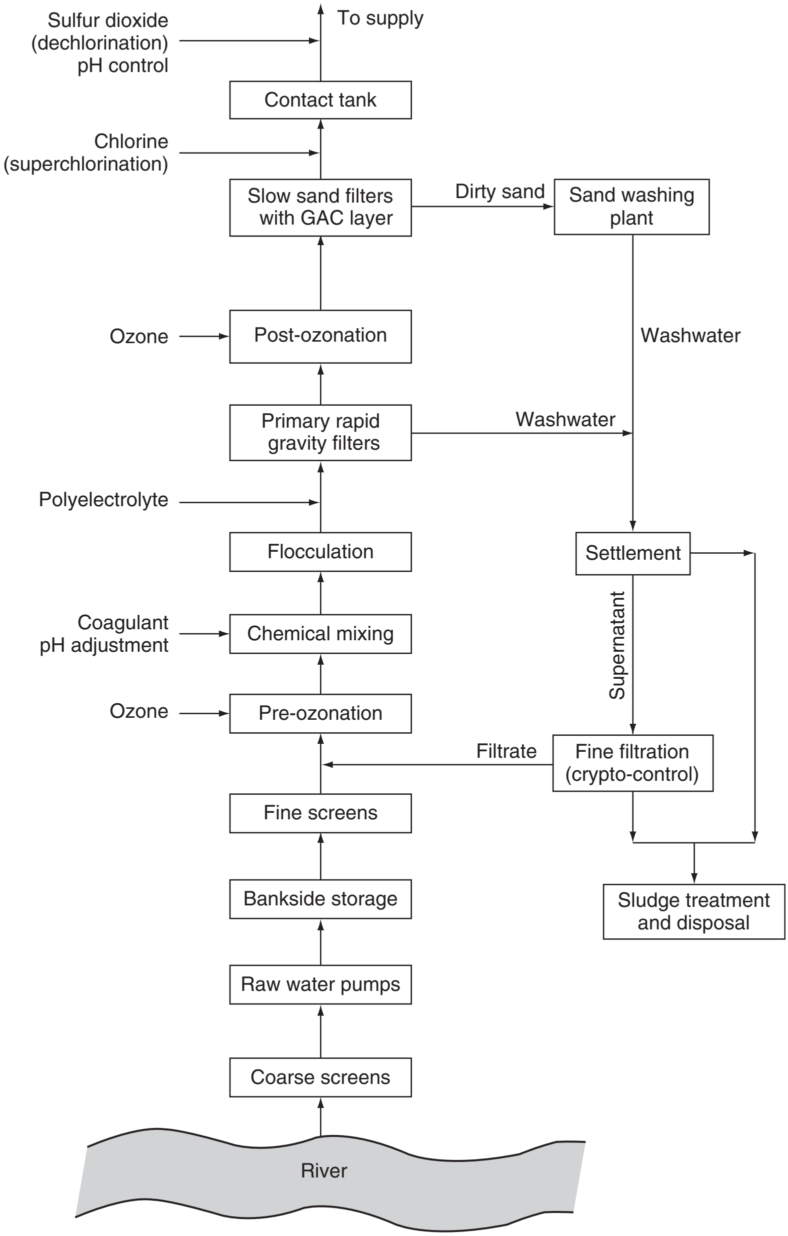


Figure 3.3 Treatment schematic diagram for upland river water

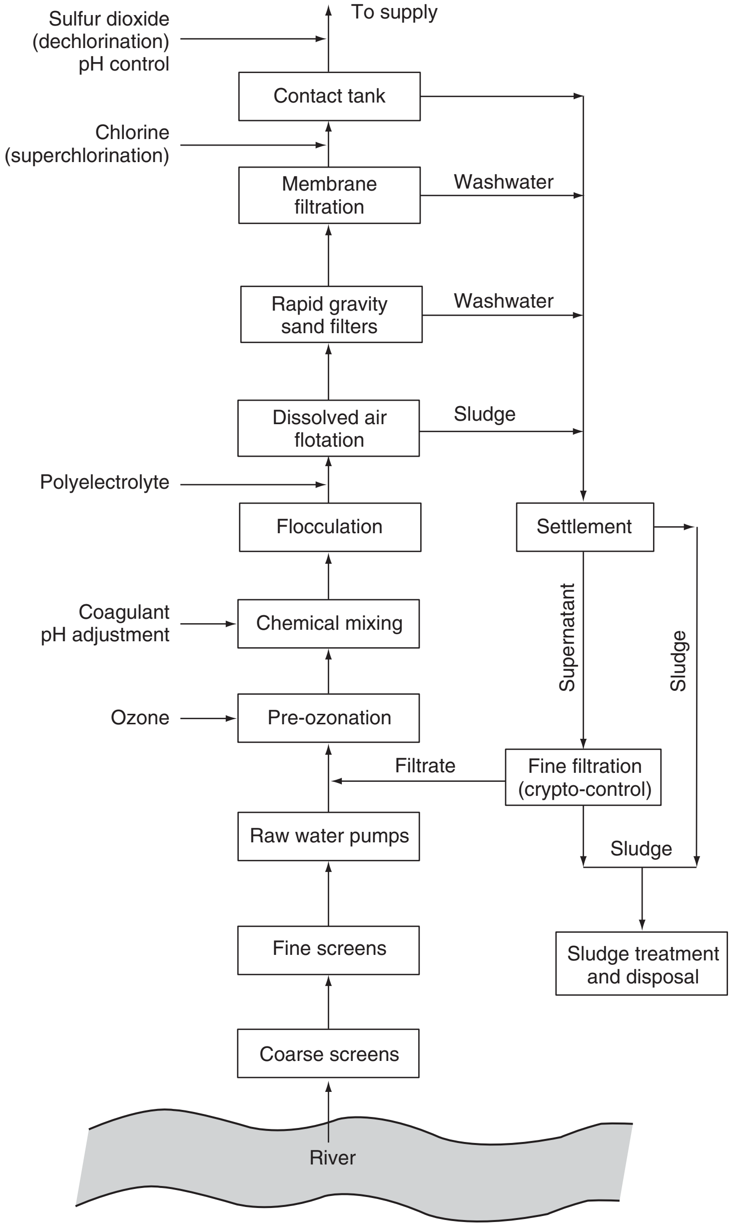
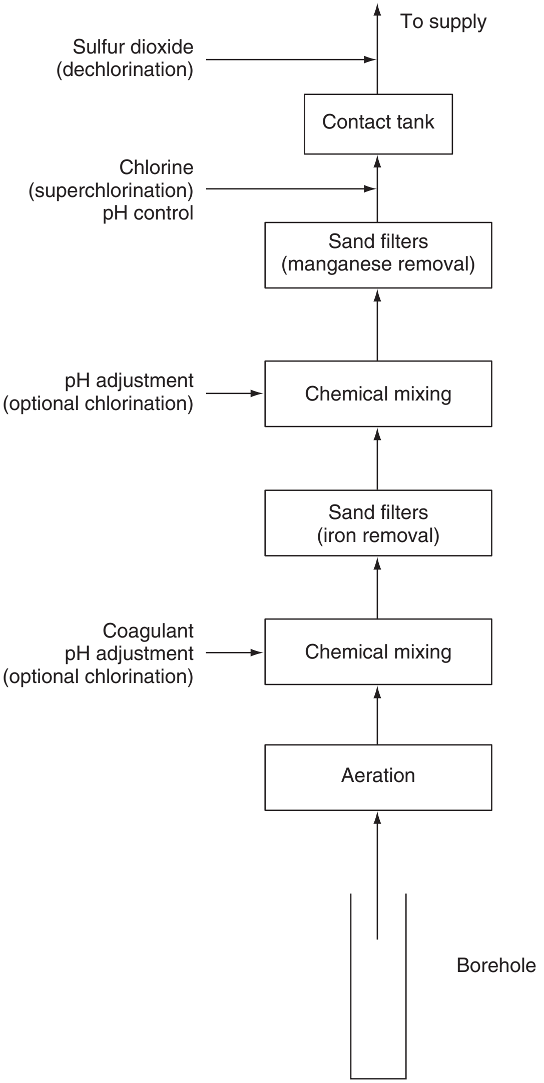


Figure 3.4 Treatment schematic diagram for borehole water



There have been many attempts to codify conventional water treatment, particularly for surface waters. These take into account key raw-water-quality parameters and define the treatment process required. Such broad approaches are of limited use. They may define a suitable starting point but often in practice the water engineer is either working within constraints that may severely limit the applicability of the solutions produced by such an approach, or the quality and nature of the raw-water may be more complex than allowed for by a simple approach.

3.6. Re-use of water

Formal re-use of water is now providing a water resource for domestic supplies in the UK and many overseas locations, including South Africa, California, Australia, and Singapore. In the UK, informal re-use has long been practiced, associated with water supply intakes on rivers being downstream of sewage works discharges, and this should be acknowledged in water safety plans where appropriate. Re-use can be classified as direct or indirect, with direct use considered as either potable or non-potable. It is almost certain that direct re-use of water in the UK will become more common over the next few years as suppliers consider how to maintain reliable supplies as the risk of droughts associated with climate change increases, and as environmental concerns lead to reductions in licensed abstractions in some locations. This section considers some of the issues relating to domestic water reuse. Many of the treatment issues and the processes which need to be considered prior to re-use can be considered to be upstream of potable water treatment, being required at the source of the water to be re-used.

There are many potential benefits of water re-use. These include the source generally being close to where the water is required, often reducing the need for pumping water long distances, with associated savings in infrastructure and energy. The source is always available, with a greatly reduced risk of reduced availability during extreme drought conditions, which in many areas of the world are expected to become more common due to climate change. These benefits are likely to become increasingly important in the future. On the other hand re-use does require extensive treatment and greater control to ensure the safety of the water supplied, and has a high carbon footprint.

Direct re-use of water for potable use is very unusual, the only significant scheme is in Windhoek, Namibia, where a small scheme was commissioned in 1969 and where there have been a number of expansions since. Direct re-use for non-potable domestic use is also uncommon, but has been implemented in parts of Australia and is likely to become more common in wealthy countries with a high per capita water use. Indirect re-use is relatively common and will become increasingly widespread in areas where water resources are stressed.

Detailed consideration of the types and details of the processes used to treat wastewater prior to re-use are outside the scope of this book, although many of the processes are also used for potable water treatment. This section discusses the Windhoek plant and touches on the approach to indirect potable re-use and direct non-potable re-use.

3.6.1 Direct re-use at Windhoek (Du Pisani and Petrus, 2005)

The original Windhoek reclamation plant was initially commissioned in 1969 with a capacity of 4600 m^3/day with the capacity increased over the years, to 7500 m^3/day in 1997. In 2002, a new plant was commissioned with a capacity of 21 000 m^3/day. The scheme treats a blend of surface water from a dam and treated effluent. The effluent is from a wastewater treatment that treats predominantly domestic wastewater.

Treatment in the old plant was essentially conventional using the following processes.

- Coagulation and flocculation.
- Dissolved air flotation.
- Rapid gravity filtration.
- GAC adsorption/filtration.
- Disinfection.
- Water into supply blended with water from other sources.

The old plant would now be considered relatively unsophisticated and such a process would generally not be considered acceptable. (The water produced from the old plant is now used for irrigation, with the GAC adsorbers decommissioned.) In 2002, a new plant was commissioned. This involved a far more sophisticated water treatment plant but the overall system also includes the upstream elements: the wastewater collection system and the wastewater treatment plant. The concept adopted was the provision of multiple barriers taking into account the entire system. These are the wastewater side precautions.

- Exclusion of industrial effluents from the domestic sewage system.
- Monitoring of the sewer system to reduce the risk of unacceptable discharges.
- Monitoring of the quality of the untreated sewage arriving at the wastewater treatment plant.
- Tertiary treatment at the wastewater treatment plant, including nutrient removal and maturation ponds.
- Monitoring of the treated effluent produced by the wastewater treatment plant.

Where a risk is identified of the treated effluent being unsuitable for re-use then appropriate action is taken to address the source of the risk.

The new reclaimed water treatment plant is far more sophisticated, using the following processes.

- Powdered activated carbon dosing when required.
- pH adjustment and dosing of coagulant.
- Pre-ozonation.

- Coagulation/flocculation.
- Dissolved air flotation.
- Rapid gravity filtration.
- Ozonation, with hydrogen peroxide addition.
- GAC filtration/adsorption with biologically active medium.
- Membrane ultra-filtration.
- Chlorination/pH adjustment.

The reclaimed water is blended with other water such that the reclaimed water is no more than approximately 35% of the water supplied. Although it has operated for 40 years, the Windhoek scheme remains unique; over much of the world cultural and social mores make it difficult to envisage direct potable re-use becoming widely used in the first part of the twenty-first century.

3.6.2 Indirect potable re-use

Indirect potable re-use is widespread and increasingly common. Most schemes involve injection of treated water into an aquifer followed by later abstraction, the other approach is to discharge treated water to a reservoir. The main requirements for indirect potable re-use are production of a high quality wastewater effluent, and sufficient retention time between production of the effluent and re-use. The treatment required for such reclaimed water will normally comprise several stages of advanced treatment such as de-nitrification, GAC adsorption, ozonation, or ultrafiltration. The particular processes used will depend upon the risks identified for the water being treated, the particular re-use pathway, and the requirements of the regulatory authorities.

In all instances, it would be normal to carry out risk assessments and define critical control points and parameters right back to the sources of the wastewater, and to ensure that potentially harmful discharges are excluded from the sewer network or are treated at source prior to discharge. It will also be necessary to appropriately monitor the water body to which the discharge is made, be it groundwater or surface water, to ensure that quality is maintained.

Draft Californian regulations (2011) set out the State's requirements for the use of treated wastewater to replenish groundwater, with sub-surface application only permitted where treatment uses reverse osmosis and oxidation processes as defined by the Regulations. Surface application is permitted where advanced treatment is not provided. In both instances extensive monitoring is required and minimum retention times are required between introducing the water into an aquifer and later abstraction.

Australia has two guideline documents relating to water reuse, the first (National Guidelines for Water Recycling, 2006) providing a generic 'framework for management of recycled water quality and use' that applies to all combinations of recycled water and end uses, and also specific guidance on the use of treated sewage and grey water for purposes other than drinking and environmental flows. The second (National Guidelines for Water Recycling, 2008) provides guidance on the use of recycled water (treated sewage and storm water) to augment drinking water supplies. These two documents provide a good overview of a risk-based approach to water re-use. The risk-based approach mirrors that set out in the Australian Drinking Water Guidelines (2004).

The trend in regulation of indirect re-use is towards extensive risk assessments that in turn lead to complex monitoring and sampling programmes. These are justified by the need to allay any concerns the general public may have, and to minimise any increase in risk to public health. Regulations covering re-use are always complex and the documentation and operational aspects of operating such schemes are also complex and have to be prepared on a case-by-case basis.

3.6.3 The Langford Recycling Scheme (Walker *et al.*, 2004)

The first large re-use scheme in the UK which provides additional treatment to treated wastewater for indirect re-use is the Langford Recycling Scheme, operated by Essex and Suffolk Water and opened in 2003. This provides advanced treatment to effluent from Chelmsford sewage treatment plant (which is otherwise conveyed through a 15 km long treated effluent pipeline discharging into tidal waters). The treated water is discharged to the River Chelmer, 3 km upstream of intakes where water is extracted for reservoir storage and for potable water treatment. The scheme can return up to 40 000 m^3/d to the river.

The water discharged to the river has to meet very stringent quality criteria, including limits on biochemical oxygen demand (BOD), nitrogen in its various forms, phosphorus and dissolved oxygen, and also requires being UV disinfected. The treated water is closely monitored and, if it fails to meet any of the consent criteria, discharge to the river is stopped, and it is diverted back into the effluent pipeline.

The treatment provided at the effluent treatment plant involves the following processes.

- Phosphate removal following ferric dosing.
- Nitrate removal in a de-nitrifying filter, using methanol as a carbon source.
- Ammonia removal in a nitrifying filter.
- Disinfection using UV light.
- Powdered activated carbon can be dosed to remove organic chemicals.

Water extracted from the Chelmer is pumped either to a large reservoir prior to treatment at one of the company's works, or direct to another treatment plant, where there is several days bank-side storage prior to treatment. Both of the potable water treatment plants which treat the water extracted from the Chelmer provide full treatment with pre- and post-ozonation and GAC absorption; they also soften the water.

3.6.4 Direct non-potable domestic re-use

This requires the provision of a separate non-potable water supply system, with the water used externally and for toilet flushing. Such systems are being provided in some Australian states where high per capita water usage and limited water resources combine to make them attractive. As there should be no intentional ingestion of such water, the needs are to produce an appropriate quality of water and ensure there is no significant risk of the re-used water entering the potable system. Treatment is aimed at producing a microbiologically safe supply, but also that is of adequate quality to ensure acceptability and not to give rise to quality problems within the distribution system. In order to minimise the risk of cross-contamination it is necessary to closely control domestic plumbing and also advisable to operate the potable system at a higher pressure than the non-potable. The Australian guideline document requires a risk-based approach mirroring that used for potable water, with extensive and site-specific documentation.

3.7. Developing world treatment

In the developing world, there are important differences in priorities. Without doubt, the overriding requirement is to provide a microbiologically safe water supply. Waterborne diseases are one of the greatest health hazards across much of the world and the high incidence of such diseases is one of the key factors causing high infant mortality rates in tropical countries with poor water supplies. This is not to say that all physical or chemical quality criteria should be ignored, but they should be considered with due regard for the overriding need for reliable supplies of microbiologically safe water at an affordable cost. There is no point in supplying safe water if it is aesthetically so poor, or so expensive, that other less safe supplies are used in preference.

The priorities in the developing world can be argued at length. A high priority should be to provide water from a source that requires little or no treatment, and to ensure the source is protected, rather than to use a lower-quality source and provide water treatment (Cairncross and Feachem, 1993). This ensures that when treatment fails, as it inevitably will, there is a far lower risk of waterborne disease transmission. A reasonable list of priorities are shown here.

- Wherever possible to utilise a safe source of water – often groundwater from a deep well – and ensure it remains protected.
- Provision of effective disinfection, ensuring that the community is protected from acute waterborne diseases.
- Ensuring a continuous supply of water.
- Provision of an aesthetically acceptable water.

- Provision of high-quality water which ensures that the community is at low risk over a long period.

Where treatment is required, the first short-term aims are normally basic treatment to provide an aesthetically acceptable water and effective disinfection. This should ensure that the water supplied is safer than any alternative sources. Longer-term treatment objectives should be to comply with the health related standards set out in the *WHO Guidelines for drinking water quality*.

A concept encouraged by WHO (1993) is that of multiple barriers, with respect to removal of pathogens. A single treatment stage will be prone to significant failures to supply water of an acceptable quality. By using several treatment stages a safer supply will be ensured. Important 'barriers' are storage, sedimentation/clarification, filtration, and reliable disinfection. It is vital that the failure of any single stage or barrier does not inevitably lead to unsafe water being supplied.

3.8. Future treatment processes

Looking into the future is more liable to result in future smiles than in accurate predictions. The current major water quality concerns arguably are lead, *Cryptosporidium*, disinfection by-products, and low concentrations of organic chemicals associated with contraceptive pills and plastics.

Lead is, at the time of writing, a major operational and treatment concern for many European water suppliers. The lead standard is to reduce from the current level of 25 μg/l to 10 μg/l at the end of 2013. This is causing water suppliers to focus very carefully on the changes that often occur as water passes through a distribution system, and on precise control of chemical dosing and pH value control. A key problem with lead control is often that the water put into a distribution system changes as it passes to customers. This may arise from biological action within the distribution system or it may be that the water entering the system is chemically unstable, or some combination of the two. Thus, the pH value and alkalinity of the water may change. Also if phosphate is dosed to inhibit lead uptake, this may be lost as the water passes through distribution.

Lead control cannot be considered independently of other water-quality and treatment issues. In particular, there can be conflict between the pH value required for disinfection (less than approximately 8.0) and that required to minimise plumbosolvency which is often 8.0 or higher, and this may place constraints on where chemical dosing for control of pH value can be applied. In the longer term the solution may be to remove lead from plumbing, but this is a major exercise that will take many decades.

Since the 1990s, there has been increasing acceptance of membrane treatment and it appears certain that the use of membrane treatment will increase in the future. The introduction of widespread use of membrane treatment in the UK was driven to a large extent by the standards for *Cryptosporidium* and THMs. Under the UK Regulations (now superseded) the use of an approved 1 μm membrane filtration process removed the requirement to sample and analyse for *Cryptosporidium*. Such sampling and analyses significantly increased the cost of operating plants, particularly small plants that would otherwise require very little operating effort. The use of membranes to treat soft peaty upland waters has also become common. Such waters are coloured, difficult to treat, and often have unacceptable levels of THMs after chlorination; membrane treatment offered an effective way of removing THM precursors. In the future, membrane treatment will increasingly be used in preference to conventional treatment, offering capital cost savings, ease of construction, and the production of high quality water.

Concern over disinfection by-products is likely to greatly influence water treatment. It has long been recognised that chlorination gives rise to the formation of THMs. More recently, the formation of bromate as a by-product of ozonation has been identified as a major concern. There are also concerns over chlorite levels where chlorine dioxide is used as a disinfectant in place of chlorine. In the future, it is quite possible that other by-products or risks will be identified that will further restrict disinfection. It is possible, even though it seems almost inconceivable today, that the use of chlorine itself could be restricted.

As noted in Chapter 2, there have been concerns, particularly in the press, over the impacts of low concentrations of endocrine disrupters in the water environment. However, at the extremely low concentrations found these currently relate more to potential impacts on the aquatic environment rather than impacts on human health. In Europe, such chemicals are, or should be, monitored if they are considered a priority substance. If a problem were to be identified it is difficult to envisage it being widely addressed by treatment, either by wastewater treatment or water treatment, as the cost, and carbon, implications are significant.

The carbon footprint and overall sustainability of water treatment is becoming of increasing interest and concern. Conventional treatment uses relatively small amounts of energy and chemicals, with coagulation and flocculation being the areas where savings may be made in the future. Later, the development of ceramic membranes is discussed, and it is possible that these will in future be used to reduce chemical consumption in coagulation: however, the head loss through the membrane will partly offset the environmental benefits of lower chemical use. Conventional desalination is inherently energy intensive and although lower-energy processes are in development, the ‘greening’ of desalination is currently focussed on low-carbon energy sources to power plants.

Water treatment will continue to evolve, with more use of the modern treatment processes. However, it may well be that the most important changes over the period

2010–2030 will be the development of improved monitoring and control systems, covering the distribution system as well as the treatment plant. As the quality of water produced by treatment plants improves it will become unacceptable for it to significantly deteriorate in distribution systems. Thus, black boxes filled with sensing and control equipment will become more and more important and will spread from treatment plants across the entire water network, and also to water sources, to allow treatment to be better optimised to variations in raw-water quality. Improvements to monitoring are consistent with the introduction of water safety plans, as covered in Chapter 15, with appropriate monitoring at critical points allowing early identification of potential hazards.

3.9. Greenhouse gases (GHGs)

It is generally accepted that it is vital to reduce the environmental impact of human activities, and the British government has legislated to require the reduction of GHG emissions to the atmosphere. The need to reduce emissions has required an understanding of their sources and quantities and there have been a number of studies undertaken to quantify energy use associated with the production, distribution and use of water and the collection and treatment of wastewater, and the associated or arising GHG emission.

In 2008, the Environment Agency produced a report (Environment Agency, 2008) on GHG emissions associated with the English and Welsh water sector. This concluded that 89% of emissions associated with water supply and disposal is domestic, with the energy used for heating water in the home (but excluding central heating) being the major factor. Public water supply and sewerage activities accounts for 11% of water-related emissions. Of this 11%, approximately 40% relates to water supply and 60% to sewerage, and around half the water supply emissions relate to water resources and treatment, and half to distribution. Whilst these figures are approximate, and represent a high level overview, this report found that water treatment thus represents a small proportion of energy use within water supply and sewerage, around 2% of total emissions of the water sector. Water resource activities and potable water distribution account for a further 2%. Other studies have produced figures significantly different, but still suggesting that water treatment is a small proportion of energy use in water supply and treatment.

Notwithstanding the small contribution to GHG emissions associated with water treatment, there is still a need to minimise these emissions, both by minimising direct energy use and by minimising the use of materials, in particular the chemicals used. In most conventional water treatment plants, the most obvious areas for energy saving are associated with coagulation and flocculation and clarification, where chemical use, mixing energy, and the treatment of the sludge produced make a significant contribution to the total footprint of a treatment plant.

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Chapter 4
Preliminary treatment

Prior to raw water passing to the main treatment processes, there is normally some form of preliminary or pre-treatment. Processes classed as pre-treatment include raw water storage, screening, aeration, straining, preliminary settling, and pre-ozonation. Any of these processes might be found at a particular plant but it is improbable that all would be needed. Each performs a particular function and, unless the problem they are designed to eliminate is present in the raw water, they can be omitted.

It used to be common to chlorinate water prior to treatment, to reduce problems of attached biological growth within the main treatment process but this can generate trihalomethanes (THMs) and the standards for THMs mean this is now uncommon. Instead, it is increasingly common to pre-ozonate water, which disinfects, oxidises some of the complex organic chemicals, and, for some waters, also improves the performance of the subsequent clarification process. Pre-ozonation is covered in Chapter 11.

4.1. Coarse intake screens

Most rivers at some time have floating or submerged debris that would damage pumps or block inlets. It is, therefore, prudent to always provide coarse screens. Typically, these comprise substantial steel bars (typically about 25 mm dia. or equivalent) and are spaced about 100 mm apart. They are normally slightly angled from the vertical to facilitate raking. Sometimes the bars are mounted in frames that are duplicated so that one frame can be lifted for cleaning or repair, without unscreened water entering into the plant. The velocity of water through the screen openings should not exceed 0.5 m/s. It is almost universal to provide coarse screening on intakes from surface water intakes. In some parts of the world, there can be large and unpleasant items in the water, but in Britain it is most unusual to have to remove dead donkeys.

4.2. Raw water storage

The provision of raw water storage on river intakes is good practice. There are several reasons why this is so. On intakes on the lower reaches of rivers there is always a risk that there will be significant upstream pollution; for example, a vehicle accident leading to diesel oil in the river or the discharge of a toxic chemical. To provide some flexibility in dealing with such incidents it is good practice to provide storage prior to a works. It is then possible to stop withdrawing water while the pollution passes down the river. In recognition of this risk, the UK government recommended in the early 1970s that a minimum of seven days storage should be provided for treatment works the raw water intakes of which were downstream of effluent discharges. The seven-day requirement for all works has not been achieved, and now never will be. However, seven days is a reasonable period to allow for the worst pollution to pass, and to provide time to deal with the source of the problem.

A significant improvement in quality of poor-quality water can be achieved by storage alone, even if such storage is not required for any other purpose. This improvement results from settling of the suspended solids and a marked reduction in pathogenic organisms. The improvement in water quality depends on residence time, whether the reservoir is fully mixed, and the time of year. Reservoirs with several weeks storage will normally give a reduction of over 90% in coliform and *Escherichia coli* numbers, rising to around 99% in the summer (Poynter and Stevens, 1975). At the same time there will be reductions in colour, turbidity, ammonia, and many organic pollutants including pesticides and herbicides (Oskam, 1995). There will also be significant reductions in numbers of *Cryptosporidium* and *Giardia* oocysts. Table 4.1 presents some data illustrating the improvements that can be achieved; the storage in Grafham Water was of the order of three years.

Longer storage periods may be provided to balance the amount of water that can be abstracted from a river against demands, or to further improve the quality and consistency of the water supplied to a works for treatment. It is normal for abstraction licences to specify both the maximum quantity of water that may be abstracted and the minimum

Table 4.1 Quality of water before and after storage

Parameter	River Thames at Oxford (Crowe, 1974)		River Great Ouse at Grafham (Saxton, 1970)	
	Raw water	Stored water	Raw water	Stored water
Colour: °Hazen	19	9	30	5
Turbidity units (type not specified)	14	3.2	10	1.5
Ammoniacal nitrogen: mg/l	–	–	0.3	0.06
Oxygen adsorbed: mg/l	1.8	1.3	3.5	2.0
BOD: mg/l	–	–	4.5	2.5
Total hardness: mg/l	300	259	430	280
Presumptive coliforms MPN/100 ml	60 000	200	6500	20
E. coli MPN/100 ml	20 000	100	1700	10
Colony counts per 1 ml	–	–	50 000	580
3 days @ 20°C, 2 days @ 37°C	–	–	15 000	140

flow in the river downstream of the intake, with the latter taking precedence. Thus, for rivers with seasonal variations in flow it is often essential to provide large raw water reservoirs to allow sufficient water to be abstracted at time of high flow to cover for the periods when abstraction has to be reduced at times of low flow. Storage periods shorter than seven days will provide some protection against pollution but provide only limited other benefits.

Raw water storage can also lead to problems, particularly where the water being stored is high in nutrients; this can lead to high levels of algae in the spring and summer.

If the subsequent treatment process can handle this then there will not be a problem. If algae do cause problems then it will be necessary to take measures to limit their growth. Another problem can be silt deposition, which can be a particular problem on smaller reservoirs. While raw water storage is undoubtedly beneficial, due allowance must also be made for potential problems in both the subsequent treatment processes and in the design of the storage itself, with provision for periodic access to remove silt and for managing potential water-quality issues. Storage equivalent to 7–15 days of the average water demand is sufficient to reduce pathogenic bacteria while being short enough to minimise problems associated with growth within the reservoir.

Large dams and deep storage reservoirs have their own water-quality issues. Problems arise from thermal stratification (discussed below) and there are often significant water quality changes over time in dams often associated with a steady build-up of nutrients and iron and manganese within the dam sediment over time. There are two main approaches to deal with these problems: draw-offs can be provided at different depths, allowing some control over the quality of the water drawn off; or a destratification system, normally using aeration, can be installed.

In Britain, nowadays, it is not easy to provide new raw water storage. Where there are working or disused gravel pits in a river valley it may be possible to combine gravel extraction or rehabilitation of disused gravel pits with new bankside storage. In other areas, this is not possible; land is expensive, there may be environmental objections to disturbing old gravel pits, and it may be necessary to protect against pollution risks by installing additional treatment rather than by providing storage. An interesting new surface water treatment plant in the Netherlands, PWN's Andijk III plant uses innovative ion exchange and membrane technology, discussed further in Chapter 10, to replace conventional clarification and filtration. What is interesting is that this plant also has a large raw water reservoir, with an eight-day retention period, which is effectively a key part of the overall process. The raw water is aerated to control algae growth and sodium hydroxide is dosed to reduce hardness prior to treatment. Integrating raw water storage and pretreatment within the overall abstraction and treatment process offers clear advantages in terms of the quality of water passing to the main treatment plant.

4.3. Algae, algal control and reservoirs

Algae are microorganisms that contain chlorophyll α and require water, carbon dioxide, low levels of inorganic substances and, most importantly, light to grow and multiply. They are essentially very small aquatic plants. They are slow growing compared with most bacteria, partly because they grow by photosynthesis. Their growth rates depend to a large degree on the availability of nutrients, nitrogen, and phosphorus, in water, and on the energy available from sunshine. Thus, they tend to grow best in waters that have higher levels of nutrients. Because they grow relatively slowly they are more often associated with lakes and reservoirs rather than rivers, although long slow-flowing rivers may also have high algal concentrations. There are multiplicities of types, many of which, if present to excess, can cause treatment problems at water-treatment plants. Algae can be either attached to the sides and bottoms of reservoirs or rivers, or can be free-floating. Blue-green algae, or cyanobacteria, are a form of algae with some similarities to bacteria. Algae can normally be seen in affected water, but some (notably *Stephanodiscus*) can remain invisible except under a powerful microscope. Outbreaks tend to be severe and sporadic. Algal blooms are a common phenomenon.

These arise when the optimum conditions exist for a particular species of algae to grow. Algal growth is exponential and the species grows until there are large numbers present and growth is limited by lack of nutrients or by some other factor; the algae then die. Each year there tends to be a series of different species of algae that grow in large numbers in succession. A particular species ‘blooms’ and then dies; another species will then bloom and die, and so on. Algal blooms normally start in the spring, as increasing light levels lead to the beginning of optimum conditions for growth, and continue through the summer into the autumn. In an uncontrolled reservoir, it is only in the winter that algal numbers are low for any extended period.

Algal blooms have a long history. It is possible that the plague of blood referred to in the Bible’s book of Exodus refers to an algal bloom in the River Nile. Nowadays, they are extremely common across much of the world, brought on by increased concentrations of nitrogen and phosphorus in lakes and reservoirs. Algal growth and eutrophication of inland waters is recognised as a major problem, and is now being addressed in EU countries. Nutrient removal is becoming increasingly common in sewage-treatment works that discharge to water bodies prone to algal blooms. However, a major source of nutrients is agriculture, and this may minimise any improvements derived from nutrient removal from wastewaters.

In the past, algal problems have sometimes come as a surprise to engineers; when the building of an impounding reservoir on a previously clear stream has led to severe algal problems. Even when algae are not a problem with a new reservoir, as the reservoir ages there can often be a build-up of nutrients within the reservoir and over a period of time algal blooms may become a problem. Fairly alkaline waters containing appreciable concentrations of nitrates and phosphates are particularly prone to algal problems. Clarity is also a factor, as algae need sunlight for photosynthesis. Thus, in rivers with a high silt load there are rarely problems with excessive levels of algae. However, in lakes and reservoirs in much of the UK and indeed much of Europe and North America, algae are a problem. It is generally only the remoter uphill lakes and reservoirs, which do not receive significant nutrient inputs from agriculture or sewage, that are free from large numbers of algae. Where a new dam is proposed the probability of algal problems can be assessed from the level of nutrients in the water, but it is foolhardy to assume that there will not be problems in new reservoirs.

Algal control is not easy. In the past, it was common to control algal blooms by dosing copper sulfate before a bloom occurred. Copper is an algicide and the technique was effective in reducing algal blooms if properly monitored and applied. Copper was dosed at around 0.3 mg/l of copper sulfate. Except in very soft waters the copper was quickly removed by precipitation as copper carbonate. However, nowadays the dosing of relatively large quantities of copper into the aquatic environment is unacceptable because of its effect on other organisms. Copper sulfate is toxic to fish at a concentration similar to that needed to kill algae and, thus, fish kills were common. Algal growth is often limited by the availability of phosphorus and, thus, in smaller reservoirs, or reservoirs storing upland water low in phosphate, it may be practicable to limit algal growth by dosing a ferric salt to precipitate phosphate. However, this is relatively expensive and produces sludge that has to be removed periodically.

A problem with reservoirs more than around 10 m deep is stratification. In the spring and summer, deeper reservoirs tend to form two layers: an upper layer warmed by the sun, the epilimnion, with a high population of algae, and a cold lower layer, the hypolimnion, which is not warmed by the sun and being denser does not mix with the upper layer. The lower layer tends to become anoxic, as the oxygen in the water is used by the organisms within it, and reducing (anoxic) conditions develop. As a consequence, concentrations of dissolved iron and manganese are often high in the lower levels of stratified reservoirs. This is not an immediate problem providing it is possible to withdraw water from different levels. Water for treatment is normally drawn from the higher level of the reservoir, where oxidising conditions exist and dissolved iron and manganese concentrations are low, although algal levels may be high. However, as winter approaches and the upper layer cools down, there is often a marked deterioration of water quality as thermal stratification breaks down and the reservoir becomes mixed. This can result in poor-quality water in the entire reservoir in the period following de-stratification.

Reservoirs can be prevented from stratifying by installing mixing systems to mix the warm and cold layers. A common way of doing this is to pump air into the lower part of a reservoir, inducing an upward current that mixes the reservoir. Such systems can ensure a uniform improved water quality in the entire reservoir. Fully mixed reservoirs may also suffer from algal blooms as algae can control their buoyancy to ensure that they stay at a level at which they receive adequate levels of sunshine. Thus, de-stratification may not resolve the problem of algal blooms. However, there is evidence that intermittent operation of de-stratification measures in a reservoir leads to significantly lower levels of algal mass in a reservoir (Simmons, 1998). This may be related to the difficulty that algae have in adopting to changing environmental conditions, or to the encouragement of grazing by zooplankton (Hall and Hyde, 1992).

A recent approach to algae control is to use ultrasound; this involves installing ultrasound generators within a reservoir to kill algae. Ultrasound is particularly effective at killing blue green algae and kills most other algae. However, it is ineffective with macrophytic (plant-like) algae. It remains to be proven how effective and economic this approach is but it does appear promising, appearing to be effective at low power levels and able to control most species of algae.

Algae are a problem that will not go away, although there are measures that can be taken to reduce the problems they cause. It is only in the very long term that the control of nutrient inputs may alleviate the problem. Thus, it is necessary to ensure that water treatment can cope with algae.

4.4. Fine screens

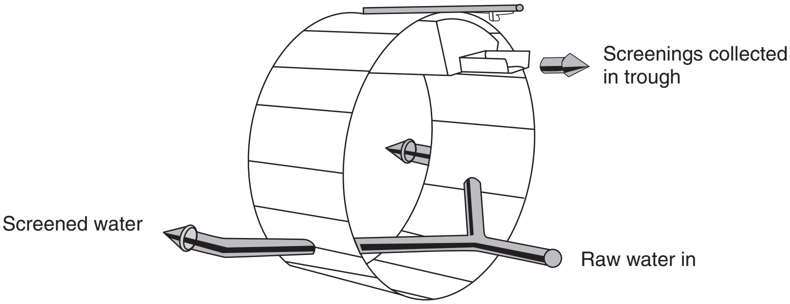
Fine screens are normally the first stage of treatment. The screens may be located at the intake or at the treatment works site, with only coarse screens at the intake. Typically, they have an effective aperture of approximately 10 mm and remove filamentous algae, waterweed, small debris, and the larger inhabitants of the raw water. However, drum screens can be constructed with far smaller openings, of around 0.1 mm, where the raw water quality is such that they will not be overloaded and blind. The screen aperture used will depend both on raw water quality and the subsequent treatment.

The openings in a fine screen mesh tend to clog very quickly if there is much debris in the water. It is possible to fit the fine mesh in frames and remove these for cleaning by hand. However, except on the smallest works, or where labour is cheap and plentiful, it is normal to have mechanically cleaned screens. These are often one of the many proprietary forms of mechanical screen constructed on the endless band or drum principle and cleaned continuously by water jets that wash the screenings away along channels.

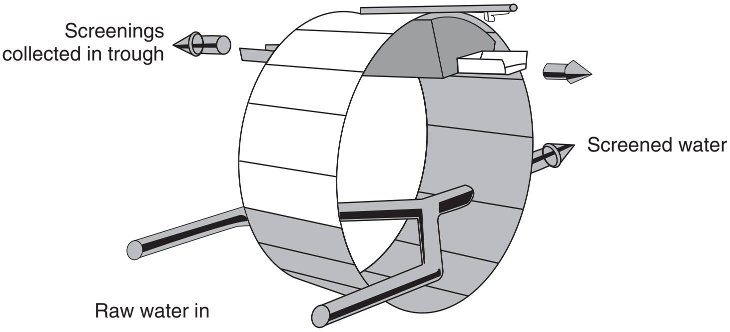
Drum and cup screens both consist of large diameter rotating drums, coated with a mesh screen around their perimeter. Water flows from the inside of the drum out through the mesh, then debris is removed from the screen and washed off into a collection trough as it reaches the highest point of the drum. A cup screen allows water only to enter from one

Figure 4.1 Principle of cup and drum screens (courtesy of Eimco Water Technologies)

Cup screen



Drum screen

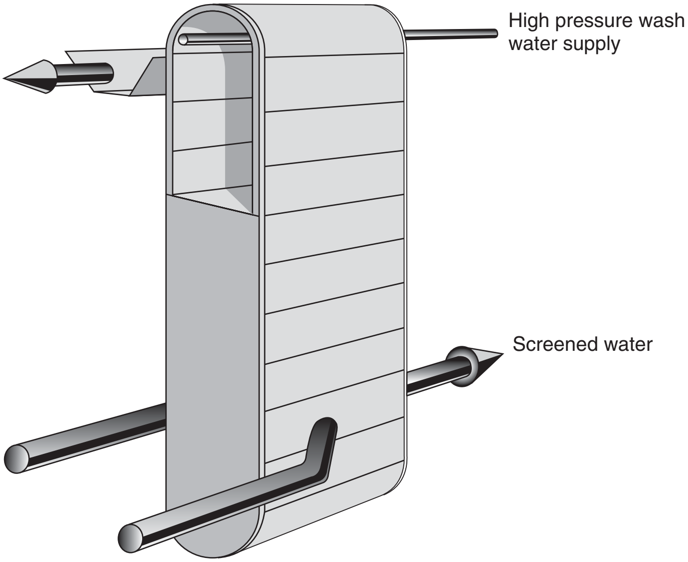


side, whereas a drum screen permits water to enter from both sides; the principles are illustrated in Figure 4.1.

For large flows a large diameter screen is required, depending on water levels and mesh size, a 300 000 m^3/day cup screen would be of the order of 5 m dia. with a screen width of 1.5 m. Extensive civil engineering works are required for large drum or cup screens. Band screens reduce the size of civil structure required by using flexible sectional bands. The principle of band screens is illustrated in Figure 4.2. They are particularly useful where there are large variations in water level, which would require a larger diameter drum than would otherwise be needed. A band screen to treat 300 000 m^3/day would require a screen around 1 m wide and a minimum immersion of 4–5 m.

Drum and band screens are proven and very common but they are relatively complex and expensive. They may also require a high pressure washwater supply. Screenings collected

Figure 4.2 Principle of band screen (courtesy of Eimco Water Technologies)

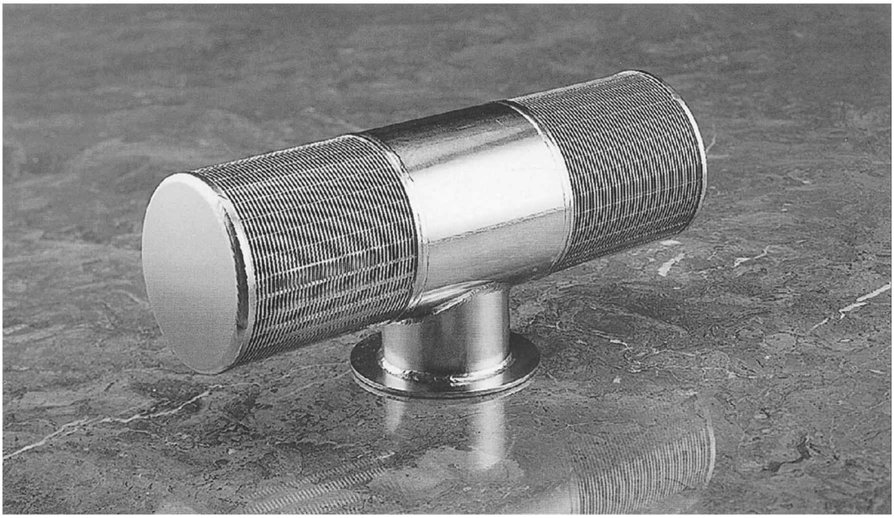


in a trough require a significant quantity of water for cleaning; typically this is less than 1% of the total works throughput.

Traditional mechanical screens are complex and two newer types of intake screen are sometimes used on new works in preference. One is a passive submerged screen using a screen derived from the wire wedge screens used in boreholes. The screen itself comprises a cylindrical section manufactured from wedge wire wrapped round a retaining structure. The slot allowing water to enter is tapered, with the narrower width on the outside of the screen to prevent clogging. Figure 4.3 shows a typical arrangement of such a screen. They are cleaned by injecting air into the screen, blasting out debris that accumulates between the wires. Thus, such screens must be submerged and have to be placed below the minimum possible water level. They are typically designed to remove particles down to 3 mm or less. Such screens are very simple; requiring relatively minor civil works, and can often be installed at locations where a drum or band screen would require a major structure. An advantage of these screens is that they have minimum impact on fish; only the very smallest young fish can pass through the screen. If they are located such that they protrude into a river, making them potentially vulnerable to being struck by boats or large items of debris, adequate protection will be needed.

Another form of screen uses rotating discs able to remove particles down to 3 mm or less (Figure 4.4). These screens are designed to return debris back to the river. They have the

Figure 4.3 Johnson passive screen (courtesy of Johnson Screens)



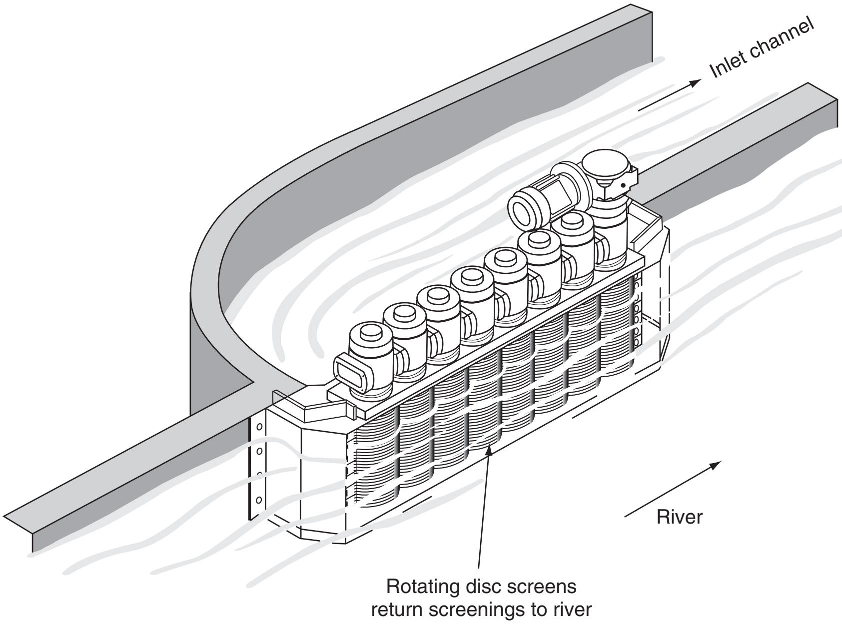
advantage of not requiring any cleaning water. Such screens are simple, but large installations require significant civil works to allow them to be periodically removed for maintenance.

Key factors to consider in screen design are the variation in river water level and the solids loading in the water being screened. It is common practice to locate river intakes upstream of a weir to provide a minimum water level that is largely independent of river flow. This removes any concern over minimum water level, but rivers can flood and the design must cater for this. Where there is potential for flooding all electrical equipment needs to be above flood level and secure access is required. Table 4.2 compares different types of screen.

4.5. Pre-chlorination

Formerly, it was common to dose chlorine prior to clarification in order to disinfect, at least partially, water passing through a works and in order to control attached algal and slime growth. Nowadays, this is rarely practised with surface waters because it maximises the formation of THMs. For groundwater with little or no risk of forming THMs, prechlorination may be used to encourage the oxidation of iron and manganese and to oxidise ammonia. For a typical groundwater containing iron and manganese a chlorine dose of around 0.8 mg of free chlorine per mg of iron or manganese should be dosed as the water arrives at the treatment plant (pH adjustment may also be required). If the objective is to oxidise ammonia a chlorine dose of eight to ten times the NH_3–N

Figure 4.4 Disc screen (Discreen) installation (courtesy of Mono Pumps)



concentration is required. (The theoretical quantity is 7.6 times the NH_3–N concentration but in practice this is a minimum.)

4.6. Aeration

Aeration can be used to oxygenate the raw water and to release carbon dioxide and hydrogen sulfide from the water. The rate of absorption or release of gas by water depends on the concentrations of the gas in the water and the air to which it is exposed, and the surface area across which the gas is transferred. Aeration is also used in treating high levels of iron and manganese. If present, they are normally in solution but, when oxidised, are precipitated and can be removed by filtration. Raising dissolved oxygen levels is an important process to speed oxidation. Cox (1964) lists ten different treatment processes that may be used for reducing iron and manganese concentrations; six of these incorporate aeration.

When water is in contact with the atmosphere, gases are absorbed or liberated such that the partial pressure of the gases in the water will move towards the partial pressure of the gases in the atmosphere. If the partial pressure of a gas in the water is higher than the partial pressure in the atmosphere, then the gas will be released from the water; conversely if it is lower, the water will absorb gas. Thus, water with a low dissolved oxygen concentration will absorb oxygen from the air, improving its taste; water containing carbon dioxide or hydrogen sulfide will tend to lose them, with benefits to the taste and corrosiveness of water. Hydrogen sulfide is essentially non-existent in the atmosphere and it will be almost totally removed in a properly designed aeration plant. However, the removal of carbon dioxide will only be partial, partly because it is present in the atmosphere and partly due to the chemical relationships between carbon dioxide and bicarbonate in the water being aerated.

Aeration is a cheap and valuable means of controlling tastes and odours due to hydrogen sulfide, of increasing the concentration of oxygen in water, and of reducing the corrosiveness of water. There are, however, limits on its practical effectiveness. Aeration is also used to remove volatile organic chemicals and this is covered in Chapter 11.

Aerators are commonly used if any of the following conditions are present in the raw water.

- Hydrogen sulfide (tastes, odours, etc.) – to liberate the dissolved gas.
- Carbon dioxide (corrosive tendencies) – to liberate excess gas, raising pH.
- Iron and manganese in solution – to lower the carbon dioxide concentration and increase the oxygen content to encourage oxidation of the reduced soluble forms of iron and manganese.
- Low levels of dissolved oxygen – to increase the oxygen content.

Aeration is commonly used on groundwater containing high levels of dissolved carbon dioxide or high concentrations of iron or manganese and on water drawn from reservoirs, where the lower-level water may be low in dissolved oxygen if the reservoir is not fully mixed.

Water from other surface sources is generally in equilibrium with the atmosphere and there is no need to aerate.

4.6.1 Types of aerator

To ensure aeration proceeds at a useful rate it is necessary to maximise the area of contact between the atmosphere and the water. There is a wide range of aerators used in water treatment; some of the common types are considered below.

4.6.1.1 Spray aerators

These use nozzles which produce thin jets of water, often directed against metal plates for greater efficiency, giving rise to a fine spray that exposes countless droplets of the water to the atmosphere. Spray aerators are very efficient. The nozzles are commonly of 25–35 mm dia. and discharge about 5–10 l/s. To aerate 10 000 m^3/day about 20 nozzles arranged within an area of 25 m^2 are needed. Care must be taken to shelter

Table 4.2 Advantages and disadvantages of different screen types

Type of screen	Size of solids removed	Advantages	Disadvantages	Typical uses
Manual bar	Down to around 25 mm	■ Simple ■ Cheap	■ Cost of manual operation ■ May blind if not cleaned ■ Unsuitable for deep intakes	As a very coarse protection to a mechanical screen
Mechanical bar	Down to 12 mm or less	■ Relatively cheap	■ Arguably more prone to failure than other mechanical screens ■ Cannot remove small debris ■ Unsuitable for deep intakes	Not normally used in water treatment
Drum (or cup) or band screen	Down to 1 mm, 5 mm more normal	■ Reliable ■ Drum screens relatively simple ■ Band screens have less extensive civil works and suitable for deep intakes and for sites with large changes in water level	■ Drum screens require more extensive civil works ■ Drum screens unsuitable for deep applications or for large changes in water level ■ Band screens more complex ■ Supply of washwater required ■ Screenings need to be handled	Both types widely used

Disc screen	Down to 2.5 mm	■ Simple and reliable ■ Screenings returned to main flow ■ No washwater required ■ Relatively fish friendly	■ Need provision for removing entire unit for maintenance ■ Unsuitable for changes in water level ■ Unsuitable for high solids loadings	Reaches of rivers where water level fairly constant
Passive well screens	Down to 25 mm	■ Very simple ■ Screenings returned to main flow ■ No washwater required ■ No power supply required at screen ■ Minimum maintenance requirements ■ Fish friendly ■ Minimum civil works required ■ Can be designed to allow screens to be lifted for inspection	■ Vulnerable to damage from boats and large debris unless adequately protected ■ Need air supply for cleaning	River intakes

the installation against wind, or the fine spray may be blown away from the collecting trays. Louvres set in a surrounding wall often shelter the area where the sprays are located.

4.6.1.2 Cascade aerators

These depend on the turbulence created in a thin stream of water flowing swiftly down an incline and impinging against fixed obstacles. The surface area of liquid exposed is rather limited and they are not very efficient.

4.6.1.3 Tray aerators

Typically these consist of up to five trays, increasing in size from top to bottom. The water falls from tray to tray through a height of around 0.5 m per tray. The total area of the trays in relation to the flow is generally about 0.1 m^2 per m^3/h. This type of aerator is apt to freeze in cold weather and encourage the growth of algae and other life in warmer climates. However, it is a simple and cheap method that has been widely used, particularly in the UK.

4.6.1.4 Slatted tower aerators

These were commonly used in the USA and consist of towers that contain a series of horizontal redwood slats. The water is introduced at the top and it cascades down over the slats. These aerators were generally used for waters high in dissolved iron; the iron tended to oxidise and build up on the wooden slats, from which it would periodically slough off. There is a tank beneath the tower to collect the water, and the oxidised iron settles in the tank and is periodically removed.

4.6.1.5 Diffused air aerators

These consist of tanks in which air is bubbled upwards from diffusers laid on the floor, the diffusers being sufficiently fine and numerous to promote a cloud of small bubbles. This type of aerator is efficient provided fine bubbles are produced and there is adequate water depth. The amount of air used can be regulated according to need. Aeration tanks are commonly about 4 m deep and have a retention time of about 15 min. Advantages of diffused air systems are minimal head loss and a lesser space requirement than other systems. The air blowers should deliver about the same amount of air in any given time as the throughput of water. For a throughput of 10 000 m^3/day an air blower of between 2.5 and 5 kW is required.

4.6.1.6 Packed tower aerators

These are towers packed with media. Water passes down through the media while air is blown upwards. These are highly efficient for gas transfer and are used for removal of volatile organic chemicals as well as for pre-treatment. They are discussed further in Chapter 11.

4.6.2 Relative effectiveness

Assuming a packed tower to be the most effective method of aeration, a very approximate measure of relative effectiveness, relative to the carbon dioxide removal effected by a diffused air installation, would be

- sprays 90%
- trays 70%
- cascades 50%.

In the UK, trays were the most commonly used method of aeration because of their cheapness, simplicity and reasonably high efficiency. They are still appropriate for oxygenation but where the prime aim is to remove carbon dioxide, modern plants generally would use packed tower aerators supplied by a specialist supplier. In some older treatment plants, where the need is only to oxygenate water, an eductor aeration system is sometimes seen; this passes water through a venturi nozzle, and air is introduced into the water at the venturi throat.

4.7. Algal treatment

Algae are notorious for causing problems with water treatment. If they are not removed they can lead to taste and odour problems in treated water. If they pass to filtration, they can quickly block up filters, leading to very short running times between cleaning. The first preference is to control their numbers in the raw water, either by reservoir management techniques, or by withdrawing water from reservoirs at levels where algae are only present in low numbers. The concentration of algae is also affected by wind, tending to be higher at the downwind end of a reservoir. If algae cannot be controlled in the raw water, the aim is to remove them as soon as possible in the treatment process.

Strainers can be used to remove algae, either rapid sand filters, or microstrainers. If the medium is coarse the former are sometimes known as roughing filters and can be worked at high loadings to reduce the load on the main filters. Such filters are effective at removing some algal species, but other species require coagulation to be effectively removed (Hall and Hyde, 1992).

Microstrainers are similar to drum screens but use a fine mesh to remove small particles. They are capable of excellent performance provided that the water is relatively silt-free. They can effectively remove filamentous or multicellular algae but do not cope as effectively with small unicellular algae. The mesh of a microstrainer has openings of around 20–40 μm for algal removal, and throughput velocities are around 10 m/h through the submerged area. The filter elements are cleaned continuously by jets and the volume of washwater is typically 1–3% of the throughput. The loss of head is dependent on mesh size and loading but is normally small (about 150 mm). They work well for some waters but not others; in favourable cases they will reduce the algal load by up to 80–90%.

Algae control their buoyancy and thus when living cannot successfully be fully removed by means of settling basins. Thus a prerequisite to removing them by settlement is to kill them. After this, they still require to be removed. In the past chlorination was commonly used to kill algae, but as mentioned earlier this is now uncommon. However, ozone will kill most algae and this can be used as an alternative to chlorination. Once killed, they can be removed in clarifiers: by dissolved air flotation (DAF), or by filtration. DAF works well for algal removal; it works best with a stable water quality and is well suited for use with reservoir water, where algae are more likely to be a problem, rather than river water.

4.8. Pre-settlement basins

Pre-settlement basins are not used in the UK, where high silt loads are unlikely to occur for extended periods. However, they may be used in countries where there can be long periods of high solid loadings associated with very large seasonal variations in river flow due to monsoons. Pre-settlement tanks, which are always horizontal flow tanks have the following advantages.

- Permit the use of upward-flow basins on rivers formerly considered too turbid.
- Improve the performance of existing upward-flow tanks over-loaded with high levels of suspended solids.
- May be built later if conditions on the catchment deteriorate (a common occurrence in developing countries where jungle-covered catchment is apt to be cleared for development).
- Can be used with horizontal-flow basins to reduce coagulant use.

Upward-flow basins become increasingly difficult to operate once the dry silt by weight exceeds 1000 mg/l. Where this is likely to happen it is helpful to put in a small, non-chemically-assisted, horizontal-flow basin immediately upstream of the vertical-flow basins to keep the peaks of suspended solids well below 1000 mg/l.

Early pre-settlement basins in the USA were designed largely in accordance with findings of Bull and Darby (1928), on the basis of 3 h nominal detention time and an overflow rate of 1.2–1.5 m/h. Such tanks have been used successfully on waters of up to 20 000 mg/l suspended solids. However, 3 h detention is more than is needed, and tanks with a detention time of not more than 1 h have been shown to operate well. Typically, effluents with considerably less than 1000 mg/l of suspended solids can safely be expected from such tanks, even though the incoming raw water might have suspended solids of 10 000 mg/l or more.

However, the use of pre-settlement tanks does not necessarily mean that all sources with high silt loads can be used. Some tropical rivers have such heavy silt loads during the monsoons that it is simply not practicable to treat water taken directly from the river.

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Chapter 5
Coagulation and flocculation

This chapter looks in some detail at what is often a vital treatment process: coagulation and flocculation. It focuses on the theory and practice of destabilisation of fine solids present in water; Chapter 6 considers the common types of coagulants used and their chemistry.

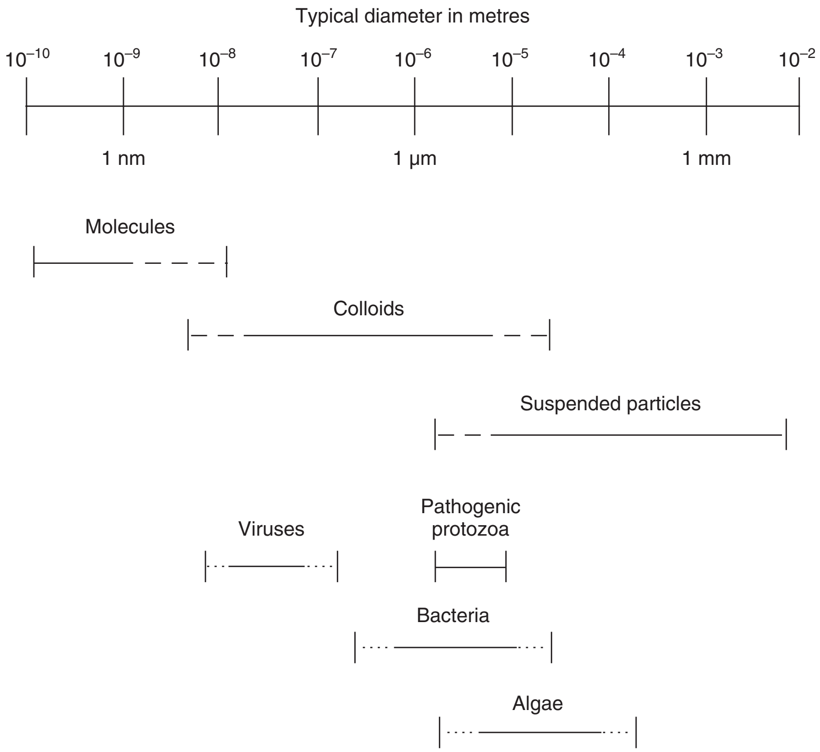
Coagulation and flocculation is an area of water treatment that developed greatly over the last 20–30 years of the twentieth century. Prior to this, plants often had very simple arrangements for dosing and mixing chemicals. These generally worked well with lightly loaded plants, at the cost of high chemical use. With the development of advanced flat-bottomed clarifiers and dissolved air flotation (DAF) much more attention has been paid to the destabilisation of water containing fine material and the conditions necessary for the formation of easily removed flocs. Also, economic pressures have meant that savings in chemical costs have become more important and the need to reduce energy use to minimise the carbon footprint of water treatment is also becoming significant.

5.1. Introduction

It is convenient to think of solids as being present in water in three main forms: suspended particles, colloids and dissolved solids. Suspended particles may be coarse or fine particles of, for example, sand, rocks, or vegetable matter. They range in size from very large particles down to particles with a typical dimension of 10 μm. Suspended particles will under quiescent conditions either settle or float. Colloids are very fine particles, typically between 10 nm and 10 μm. Finally, there are dissolved solids that are present as individual molecules or as ions. Figure 5.1 shows the size ranges of materials present in water. These size bands are approximate and some sources quote slightly different ranges. The colloid size range includes large organic molecules.

Coarse or fine particles are generally relatively simple to remove by either settlement or filtration. Dissolved solids cannot be removed by physical treatment save by reverse osmosis (although some may be removed where precipitation is possible). Thus, removal of colloids is often the main objective and most difficult aspect of conventional water treatment.

Figure 5.1 Size range of particles of concern in water treatment



Because the size of colloidal particles is so small they have significantly different characteristics to larger particles. If a 1 m cube of material were to be divided into cubes with a side of 10 nm, the surface area of the material would increase from 6 m^2 to 6000 km^2. A cubic centimetre of material would have an area of 600 m^2. As a result of the large surface area associated with small particles, surface phenomena predominate and gravitational effects are unimportant (Sawyer and McCarty, 1978).

Colloids themselves are split into two types: hydrophilic or water-loving colloids, and hydrophobic or water-hating colloids. Hydrophobic colloids are unstable; once the particles aggregate they do not easily reform as colloids. Examples are some clay particles and non-hydrated metal oxides. Other common examples of hydrophobic systems, although of liquid/liquid colloids, are emulsion paint and mayonnaise. Hydrophilic colloids include soap and wallpaper paste. When these materials are mixed with water they form colloidal solutions, which cannot be destabilised.

The reason that colloids stay as such small particles is that the particles have similar negative electrical charges, meaning that electrical forces keep the individual particles separate. The importance of surface electrical charge is associated with the very large specific surface area of the particles.

In order to remove colloids, it is necessary to form larger particles that can then be removed by physical treatment. For hydrophobic colloids, it is necessary to overcome the forces keeping the particles apart; once this has been done the particles coalesce into larger particles that do not reform into colloids. This process of particle destabilisation and formation of larger particles is called coagulation. Flocculation is the process of mixing which results in further collisions between the particles formed by coagulation, and results in the formation of relatively large particles that can be more easily removed.

Hydrophilic colloids cannot be destabilised in the same way as they would simply reform as colloids. They normally have to be removed by chemical precipitation, filtration or adsorption. However, some hydrophilic colloids can be removed from water by flocculation, specifically those composed of long organic molecules with multiple charges.

5.2. Definitions – coagulation and flocculation

Water engineers often use these terms interchangeably, but this is incorrect. Coagulation is the destabilisation and initial coalescing of colloidal particles, specifically hydrophobic colloids. Coagulation occurs extremely quickly. Flocculation is the longer term process of forming larger particles from the small particles formed by coagulation. Flocculation may also assist in removal of some hydrophilic colloids. Note, however, that sometimes ‘coagulation’ is used to cover only the destabilisation of the particles, and ‘flocculation’ to cover all agglomeration of particles. These may be more precise definitions but are less commonly used in water engineering.

A coagulant is the chemical that is dosed to cause particles to coagulate. Typically, these are metal salts. Polymers may also be used as coagulants, but are more often used to aid flocculation rather than for coagulation. When a polymer is used to strengthen or enlarge flocs formed by coagulation it is normally, if not entirely logically, referred to as a coagulant aid.

5.3. Types of destabilisation

There are generally accepted to be four main methods of destabilising colloid systems.

5.3.1 Double layer compression

This involves the addition of an electrolyte to water to increase the concentrations of ions. This has the effect of decreasing the thickness of the electrical double layer that surrounds each colloidal particle. This allows the particles to move closer to each other, meaning attractive forces have more chance of overcoming the electrical forces that keep them apart. The effectiveness of the coagulant depends on the change in ionic concentration and also exponentially on the charge on the ions added. Thus, ions with a charge of +3 are around 1000 times more effective than ions with a charge of +1. There are three points to note. First, this method of destabilisation only works when ions are present and thus where a metal salt is added which subsequently precipitates as a hydroxide floc, the effect is only present before the insoluble hydroxide is formed. Second, the effect is independent of the concentration of colloidal material. Finally, the effect is proportional to the change in ionic concentration.

5.3.2 Charge neutralisation

Adding ions with a charge opposite to that on the colloidal particles can lead to adsorption of the ions on to the colloidal material and reduction of surface charge. This reduces the electrical forces keeping particles apart and allows easier agglomeration. There are two points to note: the dose needed is proportional to the quantity of colloidal material present; and it may be possible, with some colloids, to overdose, leading to charge reversal on the colloidal matter. Where charge reversal occurs the colloid is not destabilised.

5.3.3 Entrapment in a precipitate

If soluble aluminium or iron salts are added to water at the correct pH value, they will precipitate as hydroxide flocs. If colloids are present then the hydroxide will tend to precipitate using colloid particles as nuclei, forming floc around colloid particles. Once the hydroxide floc has formed it may physically entrap other colloidal particles, particularly during subsequent flocculation. The point to note is that there is often an inverse relationship between the concentration of colloidal material to be removed and the coagulant dose required. This is explained by the concept that at high colloid concentrations the colloidal particles act as nuclei on to which the coagulant precipitates. However, at low colloid concentrations an excess of precipitated coagulant is required to entrap the colloid particles. The optimum coagulation pH value is dependent on the pH/solubility characteristics of the coagulant used.

5.3.4 Particle bridging

Large organic molecules with multiple electrical charges are often effective as coagulants. In water treatment, such chemicals are normally referred to as anionic or cationic polymers. These are believed to work by bridging between particles. It is interesting to note that both anionic and cationic polymers are often found to be capable of coagulating negatively charged colloid particles. Polymers are also often used during flocculation to aid in particle formation when they are referred to as coagulant aids. Excessive agitation of flocs formed using a polymer coagulant can lead to the flocs breaking up.

5.4. Coagulation and flocculation

The aim of coagulation and flocculation is to produce particles of a size that can be removed by settlement, flotation or filtration. Where small particles are formed by coagulation it is normally necessary to have a subsequent flocculation process to increase particle size to that required generally by the particular downstream clarification process. Flocculation occurs when particles collide with each other. Flocculation arises from three main processes: Brownian motion, stirring and differential settling. Because the particles involved in flocculation are much larger than colloid particles the effects of surface charges are much less and do not significantly affect flocculation.

For initial flocculation of particles smaller than 0.5 μm, Brownian motion is the main process. However, as particles increase in size it is necessary to encourage collisions by stirring. The amount of energy used for this needs to be appropriate to the size and strength of the flocs. Too little energy will result in low rates of floc formation but excessive energy input will lead to floc breakage.

5.5. Energy input to mixers/flocculators

The stirring of water creates differences of velocity and therefore velocity gradients. The energy input to a mixer is dissipated by velocity gradients and the unit rate of energy input is proportional to the velocity gradient established. Mixing is commonly either by a baffled channel, a mechanical mixer, or a weir. The velocity gradient in a shearing fluid is denoted by G and this is used to measure the intensity of mixing. A high G denotes violent mixing; a low G denotes gentle mixing.

A general rule is that coagulation requires high-energy mixing. This is because where metal salts effect coagulation by double layer compression and charge neutralisation their effectiveness is greatest when the salts are present as ionic complexes, and these complexes only exist for a very short time, of the order of a second or less. For effective economic coagulation by double layer compression and charge neutralisation, intense mixing is required to ensure that the metal coagulant is distributed rapidly through the water before insoluble salts are formed. The requirement for violent mixing is less where coagulation is effected by particle bridging and entrapment in a precipitate. This is because floc formation takes a few seconds reducing the need for rapid dispersion of coagulant and high-energy mixing (Amirtharajah and Mills, 1982).

Flocculation requires a lower-energy input. High-energy mixing will tend to break up large flocs, which is not at all what is wanted. Ideally the energy used for coagulation and flocculation will be adjusted to the particular process needs. Coagulation will have a high-energy input, with the energy input for flocculation decreasing as the floc size increases. The key design parameters for the design of coagulation and flocculation processes are the intensity of mixing, which the velocity gradient G is used to denote, and the retention time T. Because of the different energy requirements coagulation, or mixing, and flocculation are normally carried out as separate processes.

A conventional arrangement for flocculation is tapered flocculation. This involves the use of several basins in series, with the energy input to each basin decreasing as the water flows through the process. This is on the basis that the floc size increases as the water flows from basin to basin and thus to prevent break up of flocs the stirring rate needs to decrease. However, a recent research paper (Cairns *et al.*, 2012) on flocculation of a soft upland water concluded that tapered flocculation provided no benefit and two of the three flocculators could be turned off without any detrimental effect on floc characteristics or downstream clarification, which used DAF, and halving power cost. This only considered a single source of water but it does highlight that often there may be opportunities to optimise processes when they are subjected to close and critical examination.

5.6. Velocity gradient

The basic equations relating to mixing and flocculation were published some years ago in the USA by Camp and Stein (1943). They showed that the power dissipated per unit volume of a fluid is given by

$$P/V = \mu G^2 \tag{5.1}$$

where G is the velocity gradient (s^{-1}); V is the volume of fluid (m^3); P is the power consumption (W); and μ is the dynamic viscosity of water (Pa s).

Thus, G is equal to $(P/\mu V)^{0.5}$. This applies either to a mechanical mixer or a flocculator, where the power P is the power transferred from the mixing blades to the water. However, it is also common to dose chemicals into the flow over weirs and to flocculate in baffled or sinuous channels. In this case the power input to the water is determined by the head loss over the weir or in the channel. The power needed to transfer a flow of liquid equal to Q m^3/s across a height difference of h is given by:

$$P = \rho g h Q \tag{5.2}$$

where P (W); Q is the flow (m^3/s); ρ is the density of the liquid (kg/m^3); and g is acceleration due to gravity. Thus, for sinuous or baffled flocculation channels:

$$G = (\rho g h/\mu T)^{0.5} \tag{5.3}$$

where T is the detention time.

Similarly, where coagulation is induced by injecting chemicals into the flow over a weir equation (5.3) applies, where h is the fall over the weir and T is the retention time in the chamber into which the weir discharges. However, often the weir is located in a channel. In this case there is no defined chamber and the volume within which the energy is dissipated will have to be estimated by the designer.

5.7. *GT*

The intensity of mixing is one of the key design parameters in mixing and flocculation. The second parameter is the time for which the mixing is maintained. This is easy to calculate, being the volume in the mixing zone divided by the throughput. In practice it is normal to specify both G, the velocity gradient, and GT, the velocity gradient multiplied by the retention time. For mixing, GT has less importance than G, but for flocculation GT is an important parameter. Appendix 1 includes a sample calculation for GT.

5.8. Types of rapid mixers (for coagulation)

The purpose of rapid mixing is to speedily disperse the chemical being dosed throughout the water being treated and, where appropriate, to agitate the water sufficiently to allow coagulation by double layer compression or charge neutralisation. Where destabilisation is by particle bridging there is no need for such violent agitation.

It appears clear from much research work that in general the higher the G-value in a rapid mixer the better. High G-values have been shown to minimise the coagulant dose required (Meyer, 1995) and to maximise floc size under a particular set of circumstances. It is also clear that most water-treatment plants work well using very low G-values. There are, however, two points that should be noted: first, chemical usage is minimised by maximising the effectiveness of coagulation and flocculation, and second, for more advanced clarification processes, plate settlers and DAF, effective coagulation is essential as retention times in the clarification process are short and the feed water needs to have been properly conditioned.

Mixing devices for metal coagulants should be designed for a high G, of the order of $1000\ s^{-1}$. For minimum chemical use, where destabilisation is by charge neutralisation or double layer compression, a G of over $5000\ s^{-1}$ is required to minimise chemical dosage. Polymers require a lower value of around $400–1000\ s^{-1}$ (Ghosh *et al.*, 1985). Detention time in the mixing zone can be very short providing there is effective mixing. Dispersion of simple metal coagulants is assisted by using carrier water to dilute them to a concentration where the density and viscosity of the coagulant solution is close to that of water; however, for polymerised salts (see Chapter 6) this is not recommended as it reduces their effectiveness.

Where pH control is needed to adjust coagulation pH value, the chemical used for adjusting pH should be added prior to dosing coagulant. Chemicals used for pH adjustment need to be properly dispersed but there is no specific requirement for high-intensity mixing.

5.9. Types of rapid mixers

There are many forms of rapid mixers (Hudson, 1981). Some are considered below.

5.9.1 Weirs or flumes

It is common to find coagulant dosed above a weir. The mixing takes place in the violently agitated water below the weir. An advantage of this arrangement is that the unit energy input will be largely independent of flow. The main problem is that the coagulant should be distributed as uniformly as possible along the weir. This is actually quite difficult to achieve for what is normally a relatively low coagulant flow rate. It is extremely common to see poor coagulant distribution where this method is used. Poor distribution may require more coagulant use, but often in practice this is not a major concern. If a flume is used then there should be a hydraulic jump with the coagulant dosed in the flume throat immediately upstream of the jump.

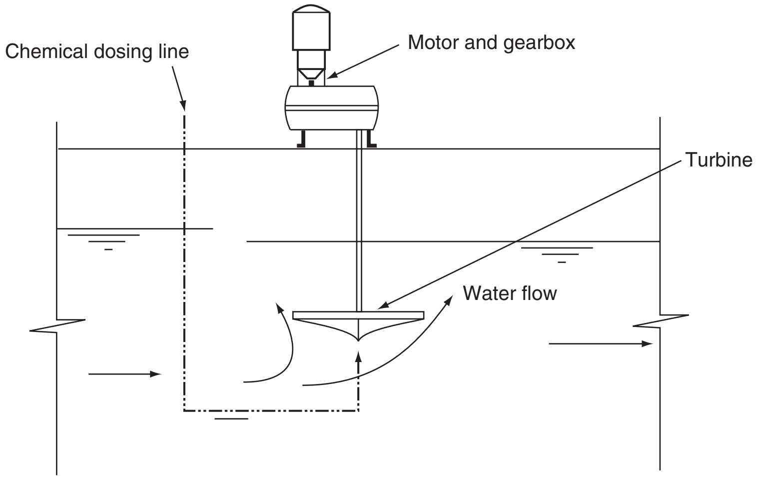
5.9.2 Paddle or propeller mixers

Comprising of a chamber containing a high-speed paddle or a submerged propeller. Where a high *G* is appropriate it is often found that the retention time should be extremely short, of the order of a few seconds, and this can present practical design difficulties. In practice, it may be necessary to use a lower *G* and a more realistic retention time. It is important to inject the coagulant into a region where it will be rapidly dispersed into the full flow.

5.9.3 Turbine mixer

Comprises of a propeller or turbine that imparts a high velocity gradient to the water before it enters the flocculation zone. It is a development of a paddle mixer creating the maximum *G* for a particular power input. Chemicals are injected at the turbine. Figure 5.2 shows a turbine mixer installed in a channel.

Figure 5.2 High-intensity mechanical mixer installed in a channel



5.9.4 Static mixer

This is a short length of pipe containing fixed blades that create violent agitation of flow passing over the blades. Static mixers were developed initially by the chemical process industry for intimate mixing of different chemicals. They have to be installed in a pressure pipeline. When used in water treatment they typically have a head drop of around 0.05 bar and have a length of two to four times pipe diameter. The great advantage of static mixers is that they are compact and highly effective in rapidly dispersing the coagulant. They are arguably less suited to large flow variations as they are optimised to treat a particular flow. Figure 5.3 shows a modern static mixer; some have more complicated blade arrangements than that shown but the principle is the same.

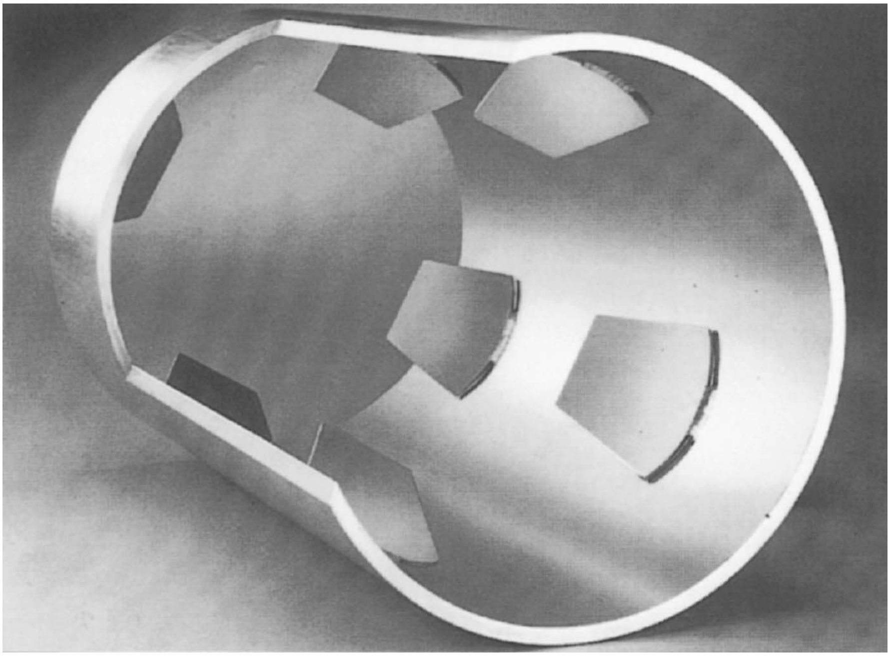
5.10. Types of flocculator

There are three main forms of flocculator: paddle flocculators, some form of sinuous or baffled channel, and sludge-blanket flocculators.

5.10.1 Paddle flocculators

Comprises of a series of two or more chambers, each containing a paddle mixer. As the flow passes from chamber to chamber the flocs grow in size, and the *G* should decrease to

Figure 5.3 Example of a static mixer (courtesy of Chemineer Ltd)



prevent the flocs breaking up. It is normal to have variable speed paddles to allow the *G* to be adjusted depending on water conditions and quality.

5.10.2 Sinuous or baffled channels

Sinuous channels are fairly common in some African and Asian plants but are unusual in the UK. Flow enters at one end and flows around the channels or baffles. Their advantage is that they are very simple with no moving parts. Their disadvantage is that they are inflexible and cannot easily be adjusted to take account of differing flow rates or flocculation needs.

5.10.3 Sludge-blanket flocculators

An alternative to flocculators is to pass coagulated water through a bed or blanket of flocs. As the water passes through the blanket smaller particles collide with the flocs to form larger flocs. The concept, developed from simple upward-flow clarifiers, requires a layer of sludge/floc in the bottom of the tank. This has been extensively developed to allow it to be used in flat-bottomed clarifiers (refer to Chapter 8 for more details). Sludge-blanket clarifiers are preceded by a coagulation stage and sometimes by a conventional flocculation stage.

5.11. Jar testing

An important piece of apparatus in a water-treatment plant is the jar testing equipment. This is used to mimic the coagulation and flocculation processes in a laboratory and is used to optimise chemical dosage and coagulation/flocculation pH value. It consists of four or six glass beakers each with a powered paddle that stirs the contents of the beaker. Normally, the paddles have a common drive and all rotate at the same adjustable speed. Each of the beakers has the same quantity of raw water added. The paddles are set to rotate at high speed. Then different quantities of coagulant, and possibly acid or alkali to adjust the pH value, are added to each jar. After a short period of intense mixing the paddle speed is reduced to reflect flocculation. After flocculation the jars are observed to assess the size and strength of the flocs formed. If the jars are allowed to settle and the supernatant is filtered through a fine filter paper, the quality of the filtered water often approximates to that which would be obtained after filtration through rapid gravity filters.

The jar test is important because it is not possible to predict optimum coagulation conditions based purely on water quality, and also because it is an easy and simple test that plant operators can use for process control.

5.12. Enhanced coagulation

Enhanced coagulation is a phrase increasingly common in water treatment, although strictly it relates to US regulatory requirements. The USEPA defines it as follows: ‘The term *enhanced coagulation* refers to the process of improving the removal of disinfection by-product (DBP) precursors in a conventional water treatment plant.' (USEPA, 1999) Under the American regulatory regime the USEPA is required to specify either maximum contaminant levels or a treatment technique to address contaminants. A treatment technique is a process or technique applied when it is impracticable to specify a required maximum concentration, and an example of a parameter of concern is total organic carbon (TOC). TOC concentration is a key factor in the formation of disinfection by-products, in particular chlorinated organic chemicals many of which are carcinogenic. Whilst the wording is complex, the essence of the requirement is that, at a water treatment plant, if coagulation means that the plant achieves the TOC removal required, which is dependent on the nature of the water, then enhanced coagulation is not required. If the required TOC removal is not achieved then coagulation has to be optimised and enhanced to achieve the maximum possible removal, with the process known as enhanced coagulation. Enhanced coagulation involves a bench scale investigation into the flocculation and coagulation process to determine the optimum conditions to maximise TOC removal and then seeking approval from a state for an alternative TOC removal criterion for a plant, and then implementing the treatment at the full size works. In the UK, arguably the term is used loosely to cover optimisation of coagulation within a water treatment plant, generally not focussing only on minimising turbidity.

The USEPA 'treatment technique' approach represents a pragmatic view of what can realistically and economically be achieved by treatment of a particular water. (Another area where this approach is used in the US is removal of *Cryptosporidium* and *Giardia*, where numerical standards are not applied but appropriate treatment is required. This contrasts with the former UK approach, now no longer used, of specifying a maximum concentration for *Cryptosporidium* oocysts.)

5.13. The future of coagulation?

The use of flocculation and coagulation followed by clarification is the conventional approach to the destabilisation and removal of colloidal material in raw water and is used at virtually all large water treatment plants in the UK. However, coagulation normally requires the use of a metal salt coagulant, and a significant proportion of the sludge produced in the preliminary treatment of water prior to filtration derives from the coagulant itself. There would be a significant reduction in sludge production, and a reduction in energy use if coagulation and flocculation could be replaced by a process that requires little or no coagulant to be dosed. Two processes which are coming into use are based on ion exchange resins: MIEX® and SIX.

The MIEX® (magnetic ion exchange) process uses magnetic ion-exchange resins that can be cationic, anionic, or both, depending on what the process requirements are. The cationic resin removes hardness and the anionic resin removes anions and negatively charged dissolved molecules containing organic carbon. The resins are dispersed within the reactor and then aggregate due to their being magnetic, enabling them to be easily removed for regeneration in a clarification process. In the UK, Yorkshire Water has used the MIEX process at a number of its treatment works.

The SIX process also uses a suspended ion-exchange resin to remove organic material and colloidal solids in raw water. It is a continuous process involving introducing resin into the feed water, which then passes to reactors, and then to lamella clarifiers where the solids are continuously withdrawn allowing the resin to be regenerated. The process was developed in the Netherlands where a large plant using the process has been constructed. This process is being trialled in the UK.

The advantages of both processes are that the sludge production is far less as there is no dosing of a coagulant. However, as with all new processes, extended operation of full-scale plants is required to assess their long-term attractiveness.

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Chapter 6
Coagulants and coagulant aids

The previous chapter looked at some of the background theory to coagulation and flocculation; this chapter considers the chemicals used as coagulants and chemicals used to assist in coagulation.

Coagulation theory proposes four methods of destabilisation of colloids. The theory suggests that coagulants should have properties to enable the following mechanisms of destabilisation.

- *Double layer compression* – compounds forming trivalent cations (positively charged) will be very effective. In practice, aluminium and ferric salts are used.
- *Charge neutralisation* – compounds forming cations are required. Again aluminium and ferric salts are used.
- *Particle bridging* – simple theory suggests that long cationic molecules would be most appropriate. In practice, both anionic and cationic polymers can often be used to destabilise negatively charged colloidal particles.
- *Enmeshment in a precipitate* – compounds forming a hydroxide floc or a carbonate precipitate may be used. Again, aluminium and ferric salts are appropriate but other metal salts can be used (e.g. calcium hydroxide or magnesium carbonate).

It is clear from the above that aluminium and ferric salts are suitable for three of the four methods of colloid destabilisation identified. This has the great attraction that it is not necessary to know the precise nature of the colloids or the destabilisation process, and that if the nature of the water changes it will normally be possible to deal with this by adjusting the coagulant dose and the coagulation pH value. This is not to say that all waters can be coagulated with a hefty dose of an aluminium or ferric salt; if the process of coagulation is charge neutralisation, overdosing of coagulant can lead to charge reversal and re-suspension of the colloidal material, leading to poorer filtered water quality. However, even if charge reversal occurs, enmeshment may yet rescue the process, although at a high cost in chemical usage.

Enhanced coagulation has the objective of maximising total organic carbon (TOC) removal and where this is used the coagulation conditions may differ from those required to maximise reduction of other parameters.

6.1. Coagulation with iron and aluminium salts

In the past, the common iron and aluminium coagulants were the trivalent compounds: aluminium sulfate, ferric sulfate, and ferric chloride. Table 6.1 summarises some data relating to these. Suppliers' data sheets should also be used to confirm properties of chemicals used in water treatment.

The addition of a ferric or aluminium coagulant to water sets in motion a complex series of reactions (Benfield, 1982). Initially, trivalent ferric or aluminium ions are formed. These then hydrate to form complexes of the metal with water molecules. In a series of further reactions, the water molecules are replaced by hydroxide ions giving rise to a further series of complexes. Dependent on pH values and concentrations, the iron and aluminium will eventually largely precipitate as a hydroxide floc. The points to note are that many of the intermediate complexes are very effective in double layer compression, and in charge neutralisation, and they also have a short life. This explains the importance of proper mixing when iron or aluminium salts are used as coagulants. If there is not effective mixing, then higher doses of coagulant may be required.

It is most important to have pH control during coagulation with iron and aluminium. The solubility of ferric hydroxide and aluminium hydroxide is lowest at particular pH values for a given water, and above these values concentrations of soluble iron and aluminium higher than the minimum will be carried forward to clarification and filtration. The floc formed by the hydroxides is important in enmeshment and it is normally most effective to maximise its formation during coagulation. However, the salts used for coagulation are strongly acidic, being salts formed from weak bases and strong acids. This makes control of coagulation pH complicated for most waters, and very difficult for unbuffered soft waters.

6.1.1 Coagulation pH

Coagulation pH controls the precipitation of metal hydroxides and also the charges on the intermediate products arising from dosing of metal salts. It is, therefore, most important in ensuring effective and economic coagulation. The usual aim is to coagulate at the pH value where the solubility of the ferric or aluminium hydroxide is at a minimum. This pH value can be theoretically derived from solubility calculations/diagrams for particular water at a given temperature. However, this is impractical in practice and jar testing is often used to determine the optimum pH value for precipitation of hydroxides. Figure 6.1 is a typical simplified solubility diagram for aluminium hydroxide showing how solubility varies with pH value. Aluminium hydroxide can exist in many forms but the diagram only shows the two species that normally determine solubility. Aluminium hydroxide has a minimum solubility at a pH of approximately 7; at lower pH values the solubility increases rapidly and at higher pH values the solubility increases less rapidly. Thus, if alum is used the pH value may be set at, or a little above 7 to avoid dissolved aluminium passing to the next treatment stage. However, if charge

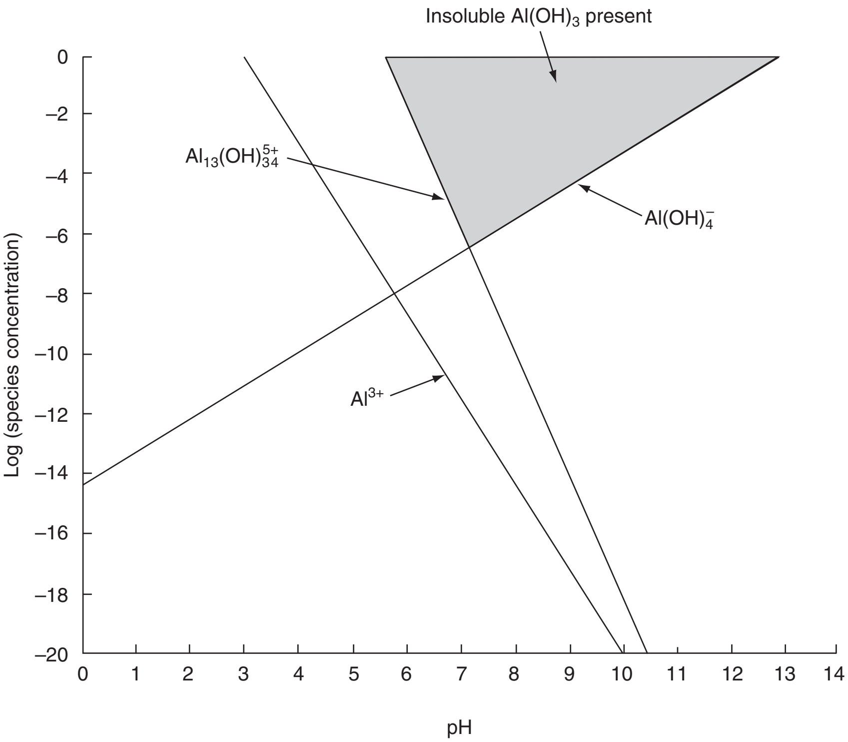
Table 6.1 Iron and aluminium coagulants

Common name/ chemical name	Chemical formula	Molecular weight	Physical forms	Specific gravity[a]	Comments
Alum/aluminium sulfate	$Al_2(SO_4)_3 \cdot 14H_2O$	595	Solid granules 8% Al_2O_3 liquid	~1.3 (bulk density) 1.32 at 15°C	Normally used in the UK as 7.5% or 8% Al_2O_3 liquid
Ferric sulfate	$Fe_2(SO_4)_3$	400	Liquid[b]	~1.5	Typically around 40% w/w ferric sulfate content
Ferric chloride	$FeCl_3$	162	Liquid[b]	~1.45	Typically around 40% w/w ferric chloride content
Polyaluminium chloride	$Al_x(OX)_yCl_z$	High	Liquid	~1.2	For 10% w/w Al_2O_3
Polyaluminum silicate sulfate	$Al_w(OX)_x(SO_4)_y(SiO_2)_z$	High	Liquid	~1.3	For 8% w/w Al_2O_3
Polymerised ferric sulfate	$Fex(SO_4)_y$	High	Liquid	~1.6	Approximately 45% w/w ferric sulfate content

[a] The precise specific gravity will depend on the supplier and the grade supplied. Refer to supplier data sheets

[b] Also available as solid but this is most unusual for potable-water treatment

Figure 6.1 Typical solubility diagram for aluminium hydroxide



neutralisation is the destabilisation process the optimum pH value may be around 5, because at this pH positively charged ions predominate. However, it should be borne in mind that the objective of controlling coagulation pH may not be to minimise carry-over of hydroxide but to remove solids. Thus, sometimes the coagulation pH value selected may not be that of minimum solubility, but where this is the case carry-over of aluminium has always to be considered.

Ferric hydroxide has a minimum solubility over the pH range 7–10, and the increase in solubility at pH values outside this range is less than for alum. Thus, ferric coagulants often require a less precise control of pH value.

The ease of pH control depends on the nature of the water. Waters that are high in dissolved salts and alkalinity generally are well-buffered against pH changes arising from dosing coagulants, or strong acids or alkalis to change the coagulation pH. For these waters, it is possible to adjust pH using strong acids or alkalis and the final pH value is relatively insensitive to changes in the dosage rates. On the other hand relatively large quantities of pH correction chemicals may be required leading to high chemical costs. Works operators like treating such waters as it is easy to control pH.

Waters that are soft with low dissolved solids have very little buffering capacity. Often, such waters have organic colour and are acidic. Such waters are poorly buffered and the pH is very sensitive to small dosages of strong acids and alkalis, and also to the metal salts used as coagulants, which are strongly acid. It is difficult with such waters to control the pH value within narrow limits and for this reason alum, with a narrow pH for minimum hydroxide solubility, may not be the preferred coagulant. For soft waters, the use of metal salts as coagulants will markedly lower pH, unless this is countered by dosing an alkali.

As discussed in Chapter 5, enhanced coagulation is an increasingly common phrase in water treatment. In practice, this will normally involve coagulating at a lower pH than would be used for straightforward turbidity removal in order to achieve a higher removal of organic material than would be achieved at a higher coagulation pH.

6.1.2 Aluminium sulfate (alum)

The data in Table 6.1 raise some issues relating to alum. It is possible to easily get confused over strengths of alum solutions and alum dosages. It is common to express the strength of alum as the percentage of aluminium oxide (Al_2O_3). Solid alum has an Al_2O_3 content of approximately between 14% and 17%. Normally, in the UK, alum is delivered as a solution; typically with a Al_2O_3 content of 7.5% or 8%. The theoretical formula of alum is $Al_2(SO_4)_3 \cdot 18H_2O$. However, the commercial composition of 17% Al_2O_3 alum is $Al_2(SO_4)_3 \cdot 14H_2O$, and this is the formula on which weights and dosage have been calculated in this chapter. An 8% solution has a specific gravity of approximately 1.3. It can then be calculated that to dose 1 mg Al/l to a flow of 1000 m^3/day, 12.6 ml/min of 8% Al_2O_3 solution is required. In calculating flow rates for other strength solutions, due allowance must be made for the specific gravity of the solution.

A simplified representation of the reaction of alum with natural alkalinity in water, shown as calcium bicarbonate (but normally expressed as equivalent $CaCO_3$), is:

$$Al_2(SO_4)_3 \cdot 14H_2O + 3Ca(HCO_3)_2 \rightarrow 2Al(OH)_3 + 3CaSO_4 + 6CO_2 + 14H_2O \qquad (6.1)$$

From this reaction, it can be calculated that each gram of hydrated aluminium sulfate requires 0.50 g of alkalinity (as $CaCO_3$) and produces 0.44 g of carbon dioxide. In order to buffer the pH, it is necessary that the natural alkalinity is in excess of that needed to react with the alum. If this is insufficient then alkalinity will need to be added. This is normally done by dosing lime (normally hydrated lime $Ca(OH)_2$) or sodium carbonate, Na_2CO_3. For each gram of hydrated aluminium sulfate, 0.37 g of

$Ca(OH)_2$ or 0.53 g of Na_2CO_3 is required to provide balancing alkalinity. Note that Equation 6.1 is not an accurate reflection of what actually happens and thus calculations based on it are only approximate.

An example of a calculation of alum quantities and sludge production is given in Appendix 1.

6.1.3 Ferric sulfate

In the UK, the use of ferric sulfate in water treatment increased in the 1990s. This was for several reasons: the 1989 quality regulations set a limit for aluminium in potable water that could be difficult to attain for some waters if alum was the main coagulant; there was concern over a possible connection between aluminium levels in water and Alzheimer's disease; and the 1989 Camelford incident (where a tanker load of aluminium sulfate was delivered into the treated water tank at a water-treatment works) made water suppliers wary of using alum. In fact, ferric sulfate can be a superior coagulant to aluminium for some waters and it is now more widely used on its merits. It is more pH tolerant than alum giving it advantages in certain processes such as decolorisation of waters of low pH value, removal of manganese at high pH value, and the clarification of water of low temporary but high permanent hardness.

The simplified reactions with natural alkalinity are:

$$Fe_2(SO_4)_3 + 3Ca(HCO_3)_2 \rightarrow 2Fe(OH)_3 + 3CaSO_4 + 6CO_2$$

or

$$Fe_2(SO_4)_3 + 3Ca(OH)_2 \rightarrow 2Fe(OH)_3 + 3CaSO_4$$

Ferric sulfate has similar properties to alum and most installations can use either without any particular problems.

6.1.4 Ferric chloride

Ferric chloride solution is more corrosive than alum and has a reputation for being difficult to handle, store, and dose. It is normally delivered as a liquid for potable use in the UK, although overseas it is sometimes delivered as a solid. The liquid is a little more difficult to store and dose than alum or ferric sulfate; its reputation most likely dates back to the days when it was normally delivered solid. Use of ferric chloride is now common in the UK. Its advantages are similar to those of ferric sulfate. It reacts with natural alkalinity similarly to ferric sulfate.

6.1.5 Polymerised aluminium and iron salts

In the 1960s, research into inorganic coagulants led to the development of partially hydrolysed polymeric aluminium salts. These are inorganic polymers with very high molecular weights. The salts are sold under various trade names. The formulation may also include other chemicals to assist in floc formation. Examples are polyaluminium chloride and polyaluminium silicate sulfate.

These coagulants are more expensive than alum but normally have a number of advantages including

- more effective at low temperatures
- faster floc formation
- lower dosage rates
- savings in pH adjustment chemicals
- possibly more effective with algae.

These chemicals can offer significant benefits at plants where there are poor dosing and flocculation facilities; the replacement of alum by a polymeric aluminium salt may greatly improve flocculation and overall treatment efficiency. Some operators have found it effective and economic to use alum when water temperatures are higher, switching to a polymeric salt in the winter when the water is colder. The salts are sold with an Al_2O_3 content similar to alum, making it easy to change coagulants.

Polymerised iron salts have also been developed offering the same advantages as the polymerised aluminium salts. They are used where they offer advantages over aluminium-based coagulants.

6.1.6 Organic polymers

Organic polymers are discussed below in the section on coagulant aids. However, for some waters they are used as the main coagulant. Their particular advantage is that they can be used over a wide pH range. Water-quality standards may limit dosages of polymer dose, as noted in Chapter 2.

6.2. Other coagulants

The coagulants discussed above are those normally encountered in potable-water treatment in the UK. However, a number of other coagulants may be encountered; these may be used in industrial water treatment or overseas. These include

- ferrous sulfate (also known as copperas) ($FeSO_4$–$7H_2O$) and chlorinated copperas, formed by chlorinating ferrous sulfate
- sodium aluminate, a compound of sodium oxide and aluminium oxide
- activated silica, reported to be highly effective as a coagulant aid. It is actually a freshly made sol (a liquid/liquid colloidal suspension) of silicic acid in water, prepared from sodium silicate ‘activated’ by one of several chemicals.

6.3. What coagulant?

It is very difficult to state reasons for selecting a particular coagulant. Table 6.2 compares in very simple terms the effectiveness of the three main sorts of coagulants for four different types of waters, using a broad categorisation of both waters and coagulants. This table also shows that there are waters that are easy to treat and waters that are difficult. The real problems of coagulant selection arise when water that is difficult to coagulate has to be treated.

Aluminium salts are the most commonly used coagulants, and by adjusting dosage and coagulation pH can be used to treat most waters. Iron salts are similarly flexible.

6.3.1 Other chemicals used in coagulation and flocculation

While hard and fast definitions are difficult, there are three main types of chemicals other than primary coagulants used in coagulation and flocculation: coagulant aids, chemicals for pH correction and weighters.

6.3.1.1 Polyelectrolytes

The term polyelectrolyte is used as a generic term to describe high molecular weight organic polymers used in coagulation and flocculation. These are normally synthetic chemicals, such as polyacrylamides or polyamines. Examples of polymers derived from natural substances include sodium alginate or starches. If synthetic, a polymer's composition and molecular size can be varied to suit operational requirements. Polyelectrolytes made from polyacrylamide, a linear resin, can be made anionic, cationic, or non-ionic, and the strength of the charge and the size of the molecule can be adjusted to suit particular needs. There are health concerns over the use of some polymers in water treatment. The 1998 EU Drinking Water Directive includes limits for some polymers and additionally the UK Government has regulations controlling dosage rates and, for some products, the composition of the polymer (UK Drinking Water Inspectorate, 2012).

The high cost of polymers is claimed to be offset by the very small dosage and the saving in pH adjustment chemicals. Dosages are typically between 0.01 mg/l, where used as a coagulant aid, and up to 0.5 mg/l, where used as the main coagulant. In the UK, regulators currently set limits of 0.25 mg/l average and 0.5 mg/l maximum for polyacrylamide polymers. A particular advantage of polymers in some instances is that they can be effective at relatively high pH values on waters containing iron and manganese. This can permit efficient single-stage removal by filtration of both metals. Polyelectrolytes are available as powders, beads, or liquids. An everyday example of a polymer is wallpaper paste. As with wallpaper paste, once the polyelectrolyte has been mixed with water, time is needed for the solution to thicken, a process known as ageing. Normally, polyelectrolytes are delivered as a powder. The polyelectrolyte is then put into an automatic dissolving system. The polymer powder is placed into a hopper. It is then fed and mixed with water often in an eductor or a mixer into which sprays of polymer powder and

Table 6.2 Comparison of various coagulants (Benfield *et al*., 1982)

Type of water	Alum	Ferric salts	Polymer
Type 1: high turbidity, high alkalinity (easiest to coagulate)	Effective over pH range 5–7 No need to add alkalinity or use coagulant aid	Effective over pH range 6–7 No need to add alkalinity or use coagulant aid	Cationic polymers usually very effective Anionic and non-ionic may also work
Type 2: high turbidity, low alkalinity	Effective over pH range 5–7 Many need to add alkalinity to control pH Coagulant aid not needed.	Effective over pH range 6–7 May need to add alkalinity to control pH Coagulant aid not needed	Cationic polymers usually very effective Anionic and non-ionic may also work
Type 3: low turbidity, high alkalinity	Relatively high dose needed to form sufficient floc pH near to 7 Coagulant aid may help	Relatively high dose needed to form sufficient floc Coagulant aid may help	Will not work well alone due to low turbidity Adding a clay to increase turbidity may be effective
Type 4: low turbidity, low alkalinity (most difficult to coagulate)	Relatively high dose needed to form sufficient floc pH near to 7 Alkalinity or clay need to be dosed to produce Type 2 or 3 water	Relatively high dose needed to form sufficient floc pH near to 7 Alkalinity or clay need to be dosed to produce Type 2 or 3 water	Will not work well alone due to low turbidity Adding a clay to increase turbidity may be effective

Low turbidity <10 NTU; high turbidity >100 NTU; low alkalinity <50 mg $CaCO_3$; high alkalinity >250 mg $CaCO_3$

water are mixed. The sophisticated mixing devices are needed to ensure proper wetting of the polyelectrolyte with water and to ensure a known strength of mixture. The mixture then passes to an ageing tank, where it is gently mixed to become homogeneous. It is then transferred to a stock or day tank, from where it is dosed. Solution strengths of polymers are low to ensure adequate dispersion, and are typically of the order of 0.5%, or 5 kg of polymer in 1 m^3 of water. Thus, relatively large volumes of solution may be required, particularly where polymer is used as the primary coagulant, and automatic polymer make-up equipment is normally used. Some polymers have a fairly short shelf life once made up as stock solution and are required to be used within a day or two.

However, the more usual use of polyelectrolyte is as a coagulant aid, to strengthen weak flocs formed in some waters. Where a metal salt is being used as the primary coagulant, polyelectrolytes often provide a powerful auxiliary bridging and linking action to promote more rapid settlement. It is normally advantageous to dose after the coagulant (i.e. after flocculation has commenced). Polymers may also be dosed prior to filtration as a filtration aid and are also important in sludge treatment.

6.3.1.2 Lime

Lime is widely used in water treatment for pH correction. The word lime is loosely used to cover both CaO (quicklime) and $Ca(OH)_2$ (hydrated, or slaked lime). Quicklime is slaked prior to use, by mixing controlled proportions of quicklime and water. This is done using continuous slaking equipment, possibly feeding to a slurry tank. Key parameters in slaking are temperature rise and slaking time. The equipment is relatively complex and the process has a reputation for being unreliable. For this reason it is uncommon to find water-treatment plants using quicklime; slaked lime is more usual. Lime is normally delivered in bulk as a powder. Quicklime is then slaked: slaked lime is made into a slurry.

Lime is dosed as a slurry typically containing around 5% CaO. The concentration of the slurry is important: it needs to be carefully controlled in order to avoid problems with blocked feed lines due to deposition. Lime dosing plants do not have a good reputation for reliability and standby equipment and dosing lines are always provided, together with easy access for clearing blockages. Because disinfection using chlorine is more effective at a pH below 8, whereas water with a pH of around 8 is often desired to minimise corrosion and to control lead dissolution, it is quite common to disinfect at a pH of around 7, and then dose lime to raise the pH to the value desired for the treated water. However, this can give rise to problems in meeting a turbidity standard at treatment works of 1 NTU (nephelometric turbidity units), due to the time needed for the lime slurry to fully dissolve. This problem can often be resolved by the use of proprietary slurry solutions, such as ‘Kalic liquid lime’. This is a calcium hydroxide suspension in which the lime particles are present as very fine particles, of around 1 micron. Such fine particles have a very high specific surface area and react very rapidly.

6.3.1.3 Weighters

Where enmeshment in a precipitate is the main coagulant process, theory suggests that the dose of coagulant required will tend to be inversely proportional to the concentration of solids that will be removed by coagulation. For waters having low concentrations of colloidal matter, it is sometimes found that large doses of coagulant are needed to ensure adequate removal of colloids. For such waters, it can be effective and economical to add fine material to increase the solids concentration in the coagulation/flocculation processes.

Even where double layer compression or charge neutralisation is the primary method of destabilisation, enmeshment will normally be important in the flocculation process and the addition of a weighter may assist.

Bentonite and Fuller's Earth can be used as weighters. They have been used on some soft, coloured waters, notably in the north of England and Scotland. However, it is unusual to find them being used, with water companies preferring to limit the number of chemicals used if at all possible.

6.4. Control of coagulation and flocculation

Given that coagulation and flocculation are now seen as important treatment processes in their own right, it is clearly important to control them properly. This is important not only to ensure optimal treated-water quality but also to minimise chemical dosing costs. All control methods depend on flow-proportioned dosing of chemicals, with the dosage determined by one of a number of alternative methods.

In the past, the dosages were defined by undertaking regular jar tests to confirm optimum coagulation pH, coagulant dose, and flocculation conditions. Although jar testing does not accurately reflect what is actually happening in the process, the coagulation/flocculation requirements predicted from jar testing are normally in practice a good indicator of the actual optimum. However, jar testing requires regular operator input and as a result other, automated, methods of determining chemical dosages are becoming more common.

Coagulation is sometimes controlled by an instrument called a streaming current detector (SCD). This is used where destabilisation is predominantly by double layer compression or charge neutralisation. The SCD takes a sample of coagulated water and forces it through a small cell containing a reciprocating piston. By measuring the current generated as the particles in the water pass the piston the charge on the particles can be deduced. It is then possible to control coagulant dose to ensure that the optimum charge for coagulation is maintained on the particles. Although the theory of SCDs appears attractive there are often practical difficulties and they are not widely used in the UK.

A more empirical approach is to control coagulation using historical data on raw-water quality, coagulant dose and pH value, and treated-water quality. This approach requires a large amount of data to enable the relationships between raw-water quality, coagulant dose and pH value, and treated-water quality to be established. Where this data exist, the coagulation control can be either manual or automatic, using suitable water-quality instrumentation.

Alternatively, continuously monitoring water-quality downstream of coagulation and using these readings for control can control coagulant dosages. Typically pH, turbidity, colour and residual coagulant would be measured downstream, possibly at more than one point in the process, and used to control coagulation.

6.5. Chemical storage and dosing

Apart from lime and polyelectrolyte, most chemicals in the UK are delivered as a liquid. This allows easy delivery and storage with uniform chemical strength, and avoids the process of making a solution. Polyelectrolyte for water treatment is normally delivered as a solid although some polymers are available as a liquid. The drawback to using liquid chemicals is that much of what is delivered is water. Where delivery distances are short this may be economical, but delivering water long distances by road to the water-treatment works clearly may not be economic where a solid chemical could be delivered. In particular, in many parts of the world alum is delivered as a solid, as a powder, in small lumps, or in large blocks of over 100 kg. It then has to be dissolved before being dosed, although one treatment works in Asia was seen to ‘dose’ alum by placing blocks in the hydraulic jump downstream of a standing wave flume.

Table 6.3 lists some of the chemicals normally used in coagulation and flocculation, either as coagulants or for pH control, and gives some details on storage and dosing. All of the chemicals listed need to be treated with care, and some are extremely dangerous. In designing or specifying systems for these chemicals, it is essential to consider health and safety aspects and how to safely cater for failures of plant. Liquid storage areas need to be bunded and drained to a storage/neutralisation tank. Different types of flanges may be used to ensure tanker deliveries cannot go to the incorrect tank. Emergency showers are needed at locations where most of the chemicals listed are stored and used. The risks associated with using chemicals need to be considered and addressed in designing and specifying chemical delivery, storage and dosing systems and the recommendations of suppliers relating to these issues need to be followed. In practice, polymers can be a particular risk, as although they are not as obviously hazardous, spillages can be extremely slippery and may not be immediately apparent.

Nowadays, most liquid chemicals are stored in above-ground glass reinforced plastic (GRP) tanks. However, not all GRP resins are suitable for all chemicals and specialist advice is needed in specifying chemical storage tanks. Polyvinyl chloride (PVC) tanks

Table 6.3 Storage requirements for various chemicals used for coagulation

Chemical	Delivery form	Storage requirements	Dosing requirements	Comments
Aluminium sulfate (alum)	Liquid	Normally stored in lined concrete tanks or glass-reinforced plastic (GRP) storage tanks	Diluted by carrier water after dosing pumps[a]	Strongly acid
Aluminium sulfate (alum)	Solid-small blocks (kibbled)	Normally delivered directly into concrete saturators	Diluted by carrier water after dosing pumps[a]	Saturated solution should be transferred to a day tank and diluted to a standard strength for ease of control Strongly acid
Aluminium sulfate (alum)	Solid-large blocks	Dry storage	As for liquid	Blocks placed in saturators as for kibbled material
Polymerised aluminium salts	Liquid	Normally stored in lined concrete tanks or GRP storage tanks	May be diluted with carrier water[a]	Acid
Ferric salts	Liquid	Normally stored in lined concrete tanks or GRP storage tanks	Diluted by carrier water after dosing pumps[a]	Strongly acid
Hydrated lime	Powder	Silo	Prone to blockages – carrier water and duplicate dosing lines normal	Batches of solution made up using precise quantities of limes and water Strongly alkaline
Sodium hydroxide	Liquid	Normally stored in lined concrete tanks or GRP storage tanks		Strongly alkaline
Polymers	Powder in bags	Dry storage or tanks	Carrier water used after dosing pumps	Batches of solution made up using precise quantities of polymer and water Solution requires ageing prior to use. Solution very slippery on floor
Acids	Liquid	GRP or lined steel tanks	Carrier water sometimes used after dosing pumps	Carefully designed storage essential

[a] An excess of carrier water will affect the coagulation process, particularly with the polymerised products for which the supplier’s instructions should be followed

may also be used for most of the above chemicals but, again, advice should be sought from manufacturers.

Chemical dosing has to be precisely controlled for trouble-free water treatment. This requires the use of positive displacement pumps that dose a constant quantity of chemical for each pump cycle. Until recently, chemical dosing has typically been done using variable stroke diaphragm pumps powered by variable speed motors. The quantity of chemical dosed per unit volume of water being dosed (in other words the dose in mg/l) was controlled by varying the stroke of the pump. This could either be set manually or by an instrumentation and control system. Varying the pump speed, using the signal from a flowmeter, catered for variations in flow. Systems of this sort have been used successfully for many years and continue to be used. The design of chemical dosing pumps takes into account the chemicals being dosed, and the nature of the water being dosed. The materials in the pump head are dependent on the chemical being dosed. Where chemicals are being dosed to a potable-water system then the pump has to be designed to prevent harmful substances from the pump entering the water.

The weakness with such systems is the chemical pump itself. While traditional dosing pumps are extremely reliable they commonly have two problems. If there is a blockage in a chemical line, they are unable to operate at the higher pressures needed to clear it, and they commonly use a diaphragm which requires regular replacement, or which would eventually fail. The advent of sophisticated computer-controlled chemical dosing has allowed the use of other forms of positive displacement pumps that are controlled only by varying pumping speed, with the control system adjusting pump speed to take account of both flow and dose. Small progressive cavity pumps are now increasingly used for dosing of chemicals. They are particularly useful for lime dosing where lines are prone to blocking and the high pressures generated by these pumps often clear minor blockages. The use of positive displacement pumps does not completely overcome problems of blocked chemical lines but it does reduce their impact on plant operation.

Examples of calculation of chemical dosages are given in Appendix 1.

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Chapter 7
Theory and principles of clarification

This chapter considers some of the principles that apply to clarification, covering settlement under quiescent conditions, shallow-depth settlement, upward-flow tanks, and dissolved air flotation (DAF). It also covers sludge-blanket (or upward-flow solids contact) clarifiers that combine flocculation, within a layer of flocs, with upward-flow clarification.

7.1. Theory of settlement

When silt-laden water is admitted to the still conditions of a sedimentation basin and its velocity falls to near zero, its capacity to transport solids disappears and solids will begin to settle or rise, depending on whether their density is greater or less than water. Assuming that no forces other than gravity is involved, a particle with a density greater than water will settle. The particle, assuming it to be spherical, will accelerate until it reaches its terminal velocity, given by:

$$v_s = [4g(\rho_1 - \rho)D/3C_D\rho]^{0.5} \tag{7.1}$$

where $D =$ diameter, $\rho_1 =$ density of sphere, $\rho =$ density of water, $g =$ gravitational acceleration, $v_s =$ settling velocity and $C_D =$ drag coefficient.

Equation 7.1 is derived from the definition of coefficient of drag, which is the ratio of the actual drag force to the dynamic drag force (Webber, 1971):

$$C_D = F/(0.5\rho v^2 A) \tag{7.2}$$

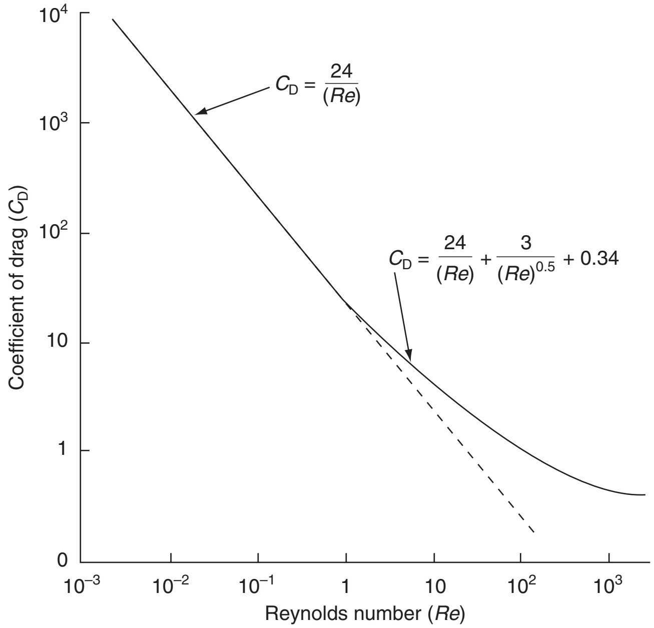
where $F =$ actual drag force, $v =$ relative velocity and $A =$ projected area of the moving body.

Where gravitational forces and viscous forces act on bodies, the relative size of the two forces is defined by the Reynolds number, where:

$$Re = \rho v D/\eta$$

where $Re =$ Reynolds number, $v =$ velocity of the body, $\eta =$ dynamic viscosity.

Figure 7.1 Relationship between C_D and *Re*



For Reynolds, numbers below 500, flow is predominantly laminar, whereas for Reynolds numbers greater than 2000 flow is predominantly turbulent.

In 1946, Camp collected and published the results of work into the relationship between the Reynolds number and C_D for spheres. The relationship is shown in simplified form in Figure 7.1.

For the straight part of the curve, which represents laminar flow:

$$C_D = 24/Re$$

Substituting for C_D in Equation 7.1 leads to:

$$v = g(\rho_1 - \rho)D^2/18\eta \qquad (7.3)$$

which is Stokes' law. For spheres with the same density, providing the Reynolds number is low, the unhindered settling velocity is proportional to the diameter of the sphere

Table 7.1 Variation of viscosity of water with temperature

Temperature: °C	Density of water: kg/m^3	Dynamic viscosity, η: 10^{-3} Pa s	Kinematic viscosity, v: 10^{-6} m^2/s
0	999.87	1.7921	1.7923
4	1000	1.5676	1.5676
10	999.37	1.3097	1.3101
20	998.23	1.0087	1.0105
30	995.68	0.8004	0.8039

squared. As the Reynolds number increases to more than approximately 2000, C_D becomes equal to approximately 0.4: thus the settling velocity is given by:

$$v_s = [3.3g(\rho_1 - \rho)D/\rho]^{0.5} \tag{7.4}$$

Using Equations 7.3 and 7.4 it is possible to calculate settling velocities for low and high Reynolds numbers. Some key properties of water needed to calculate these are given in Table 7.1. The kinematic viscosity of a fluid is equal to the dynamic viscosity divided by the density.

Settling velocity is inversely related to kinematic viscosity, which decreases with rising temperature (Table 7.1). It, therefore, follows that higher water temperatures decrease the drag coefficient and thus increase the rate of settlement. Similar particles could take twice as long to settle in the winter as in the summer and settlement basins need to be sized accordingly.

7.2. Settlement in horizontal-flow basins

For simple, unaided settlement consider a rectangular horizontal-flow basin as portrayed in Figure 7.2. It is clear that a particle settling at v_s m/s and being carried horizontally by water flowing at velocity v m/s would follow the inclined path AB, and by comparing similar triangles the particle would just reach the bottom when:

$$v_s/v = D/L$$

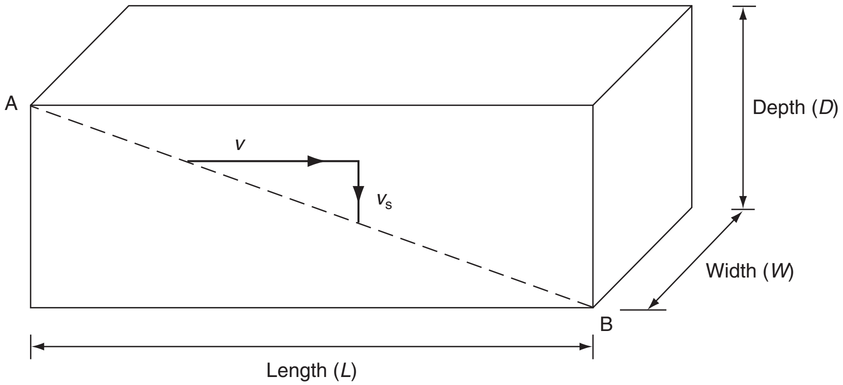
If Q is rate of flow is in m^3/s, W is the width of basin in m; A is area ($=WL$) in m^2, then:

$$Q = vWD$$

and so by substituting for v,

$$v_s = Q/A$$

Figure 7.2 Theoretical settlement in a horizontal-flow settlement tank



The quantity Q/A has the units of $m^3/s\ m^2$ or m/s, and is known as the overflow rate or surface-loading rate. It is equal to the theoretical settling velocity of a particle that would just settle out in the settling volume represented in Figure 7.2. It is generally stated in metres per hour or, sometimes, in metres per day.

The importance of surface area in the simple theory of settlement is clear and logically it would appear that the depth of the basin and therefore the liquid retention time is of little significance. This theoretical conclusion is the basis for tube and plate settlers. However, although the theory is of great importance, there are other factors involved and in practice simple theory does not apply in full.

7.2.1 Laboratory experiments

Much research has been done on the settling velocities of discrete particles and typical figures for the settling velocities of selected particles are given in Table 7.2. These are indicative of the general range of settling velocities of the particles to be removed by settlement; but the velocities are difficult to apply directly. Where a design is to be based on settling velocities, it is necessary to determine actual values by laboratory tests. However, the figures from laboratory testing should only be applied with caution because the particle sizes in any given suspension are very variable, and the particles rarely remain discrete. There is inevitably interaction between the particles of different sizes, particularly after coagulation, with larger particles falling faster than smaller ones and tending to collide and combine with smaller particles. The particle size distribution of a particular water will vary over the course of a year. The settling velocities in Table 7.2 are significantly less than the values obtained by calculation; the theory relates to spheres rather than the irregular-shaped objects found in real life.

Table 7.2 Settling velocities of particles in water

Diameter of particle: mm	Settling velocity: mm/s		
	Sand specific gravity 2.65		Alum floc, specific gravity 1.05 at temperature 10°C[c]
	Novotny *et al.*[a]	AWWA[b]	
1	140	100	0.7*
0.5	72	51[d]	
0.1	6.7	8	
0.05	1.7	2.9	
0.01	0.08	0.154	
0.005	0.01		

[a] Novotny *et al.*, 1989
[b] AWWA, 1969
[c] McLaughlin, 1959
[d] Value interpolated from data
* Calcium softening precipitates with a specific gravity of 1.2 settle approximately three times as fast as alum floc

Under typical water treatment works conditions, with flocculation and agglomeration taking place, the simplifying assumption of there being discrete particles does not apply, and it is easy to demonstrate that retention time is also of importance and, therefore, settlement is not independent of basin depth. One such experiment makes use of a cylinder, equal in depth to the proposed basin, with draw-off points at different levels, as shown in Figure 7.3. The turbidity of samples from each of the draw-off points A, B, C and D can be measured at increasing time intervals as the water clears slowly from the top. The practical importance of this test is that in a horizontal-flow basin the average turbidity at the outlet weir reflects that of the entire vertical cross-section of the basin and the effectiveness of the basin after any given time is represented by the average of the turbidity readings at all the outlets.

In a widely quoted experiment carried out by Camp (1946), the results in Table 7.3 were noted. The figures show the variation of concentration ratio, defined as the measured suspended solids concentration divided by the initial concentration, with time and depth under quiescent settling conditions. The figures are for discrete particles. The last two columns are the averages of the concentration ratios and, derived from these figures, the removal ratios. The figures show that removal is not independent of depth and is influenced by both overflow rate and retention time.

In this experiment, it can be seen that shallower basins would achieve better clarification in a given time. For instance, after 90 minutes retention the average concentration ratio of silt from points A and B would be 0.13, as opposed to 0.46 for a basin of full depth.

Figure 7.3 Settling rate experiment

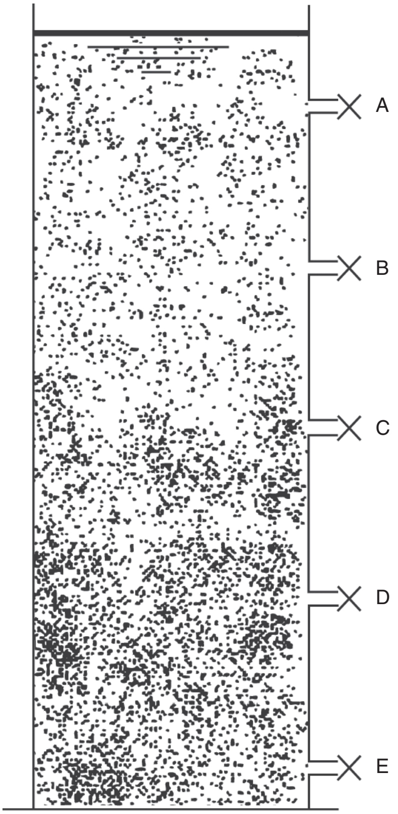


Table 7.3 Results of settlement experiment

Elapsed time: min	Concentration ratio sampling point						Removal ratio
	A	B	C	D	E	Average	
0	1.0	1.0	1.0	1.0	1.0	1.0	0.0
15	0.65	0.96	0.98	0.99	1.00	0.92	0.08
30	0.23	0.81	0.94	0.97	0.98	0.79	0.21
45	0.10	0.62	0.85	0.93	0.96	0.69	0.31
60	0.05	0.45	0.72	0.86	0.92	0.60	0.40
90	0.03	0.23	0.52	0.70	0.83	0.46	0.54
180	0.01	0.06	0.16	0.32	0.46	0.20	0.80

The removal ratio in the shallower basin would be 0.87 as compared with 0.54, and the logical conclusion is that for the conditions of this experiment a shallower basin would be better than a deep one.

7.3. Settlement in upward-flow tanks

Settlement in the upper regions of an upward-flow basin is controlled by making the area (A) of the tank sufficiently big so that $v < v_s$, where v is the upward velocity of the water ($=Q/A$) and v_s is the settling velocity of the particle that it is desired to remove. When this condition is achieved the particle will be settling through the rising water and clarification must result, with all particles with a settling velocity greater than v being removed.

In practice, the upward velocity of the water is kept down to about half the settling velocity of the floc particles. The normal velocity of settlement of well-formed floc is about 3 m/h (McLaughlin, 1959). If coagulant aids are used this may become 6–10 m/h, and where the floc is consolidated by pulsing or other devices it may be even higher. In a softening plant, the settling velocity of the particles of calcium carbonate is about 8 m/h. Applying the commonly used safety factor of 0.5, many upward-flow basins are provided with surface areas designed to limit the upward velocity of the water to the values shown in Table 7.4. Chapter 8 provides more details of typical loading rates for different clarification processes.

As in the case of the horizontal-flow basins, it would appear in theory that the depth of the tank is not particularly significant and that surface area is the only important factor. However, this conclusion is not correct. Depth is important for a number of reasons and the vast majority of upward-flow basins are fairly deep, typically 3–6 m.

7.4. Sludge-blanket clarifiers (upflow solids contact clarifiers)

These clarifiers were commonly known as sludge-blanket clarifiers but are also referred to as floc blanket or (upflow) solids contact clarifiers. They are upward-flow clarifiers and their efficiency depends on highly efficient flocculation resulting from passing the feed water through a layer of fluidised flocs. They have been developed from hopper-bottomed upward-flow clarifiers. These were desludged at regular intervals, leading to a build-up of settled sludge in the hopper prior to desludging. It was found that the

Table 7.4 Design upward water velocities for upward-flow tanks

Source of water	Velocity: m/h
River supply, normally coagulated	1.5
River supply, with coagulant aids	3–5
River supply, with coagulant aids and floc improvement by pulsing	6
Softening plants	4.2

tanks performed better than was expected and the reason was that as the water entered the tanks it flowed upwards through a layer of sludge. This led to flocculation, with collisions between small flocs in the feed water and the larger flocs in the blanket of sludge in the lower part of the clarifier.

The first sludge-blanket clarifiers were upward-flow clarifiers in which a layer of 'sludge' was maintained in the body of the tank, as opposed to operating the tank to minimise solids retention. However, such tanks are very deep and expensive to construct. Modern sludge-blanket clarifiers are flat bottomed and are much more complex. To work properly they depend on accurate distribution of feed water across the entire tank and precise control of the 'blanket'. The sludge blanket consists of a fluidised layer of flocs, which are settling at a velocity approximately equal to the upflow velocity.

The principles applied to sludge-blanket clarifiers are relatively straightforward. However, the practical application of the principles has led to a wide range of proprietary designs, some of which are extremely complicated. The Pulsator tank is a development of sludge-blanket clarifier whereby a pulse of pressure is regularly applied to the feed water in order to encourage flocculation and to maintain the blanket in a uniform suspension. This is done by the continuous sequential application and release of a small negative pressure to the feed water. Chapter 8 illustrates some of the possible layouts of sludge-blanket clarifiers.

7.5. The merit of adequate depth

Except for shallow high-rate clarifiers all basins have appreciable depth, typically 3 m or more. Erroneously or not, it is normal practice to relate basin capacity not only to surface loading but also to the hourly rate of throughput and to classify basins in terms of hours of nominal retention. A capacity of 3–6 h is typical of many horizontal-flow basins but vertical-flow basins are invariably much smaller. The reasons for providing adequate depth to basins are as follows.

- The theory of settlement is based on the concept of the discrete particle. A discrete particle by definition remains separate and unchanged in volume whereas, in practice particles grow in size by agglomeration with other particles. The bigger the particle becomes the quicker it settles; and the greater the distance through which it can fall the more of its smaller neighbours it adheres to and removes from suspension. Therefore depth (i.e. falling distance) has value.
- Additional depth may be required for the accumulation of sludge. If there is periodic removal of sludge some surplus tank capacity is required for sludge storage.
- The theory of settlement assumes gentle non-turbulent flow, but it is clear that, in very shallow horizontal-flow basins, velocities would tend to increase to a point where excessive velocity and turbulence could cause re-suspension of fine particles

and interfere with the whole settlement process. Because $v = Q/$(cross-sectional area of the basin) and the area is equal to the width times the depth, the depth plays a part in keeping v within reasonable limits (i.e. 0.3–0.9 m/min). Above velocities of this order more of the finer particles would remain in suspension.

- In upward-flow sludge-blanket clarifiers, water enters at the bottom and in its passage upwards has to flow through a zone of settling sludge. This action depends on the presence of sludge and is more efficient and easier to control in fairly deep tanks.

7.6. Shallow-depth (high-rate) sedimentation

It has taken many years to realise the undoubted advantages inherent in shallow-depth sedimentation and for tube or plate settlers to become generally accepted. Hazen first stated the principle of making settling basins as shallow as possible in 1904. In the 1950s, Camp was proposing settling basins with depths of only 150 mm and detention times of 10 min. Many early attempts actually to do this were unsuccessful because of the unstable hydraulic conditions in the shallow basins and the difficulty of removing sludge. Since the 1960s, the use of laminar-flow tilted tube or plate settlers has developed and their use is now commonplace.

The following deals with counter-current systems, these are systems where the flow of the water is upwards and the flow of the sludge is downwards. This is not the only arrangement possible and, although rarely encountered, both co-current and cross-current arrangements are possible. Similar principles to those set out below apply to these other layouts. Both plate and tube settlers use the principles and concepts of quiescent settling outlined above. They both require laminar flow to operate. This requires a Reynolds number of less than 800 (Yao, 1970).

7.7. Plates

Under laminar-flow conditions there will be a parabolic velocity profile for the velocity parallel to the plates, with the velocity being zero at the plates and a maximum midway between the plates as shown in Figure 7.4. The average velocity compared to the

Figure 7.4 Velocity distribution in a plate settler

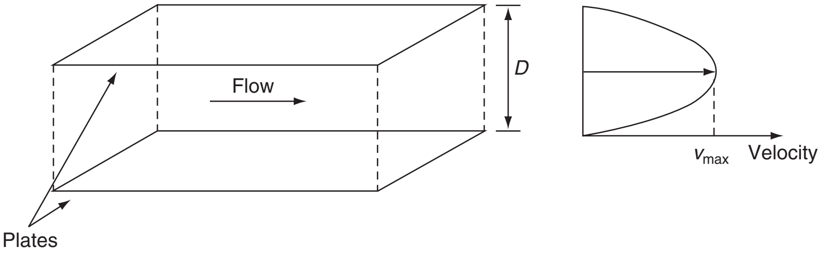
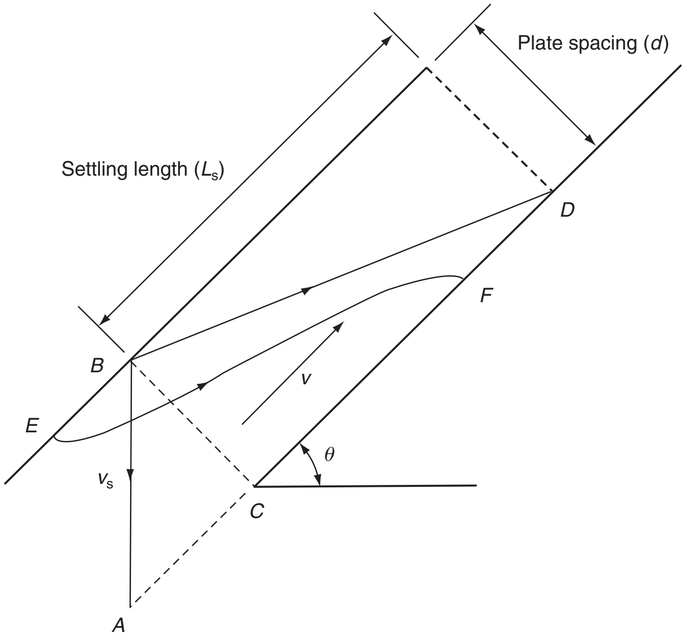


Figure 7.5 Theory of plate settlers



maximum velocity will depend on the geometry of the settler; for circular tubes the mean velocity is half the maximum velocity whereas for plates it will be two-thirds of the maximum velocity.

The settling velocity, v_s of the particle, theoretically removed in a plate settler is considered by Yao (1973). A simplified approach to arriving at his conclusions can be derived from a consideration of Figure 7.5. This indicates plates with a settling length of L_s, a plate spacing of d, and an angle to the horizontal of θ. The path of a particle settling at a uniform velocity will be complex because of the effect of the parabolic velocity gradient shown in Figure 7.4. The actual path will take the form of the curve represented by *EF*. However, this can be approximated to by the straight line *BD*, where *BD* represents the path (simplified) of the particle with the lowest settling velocity. It can be seen from inspection that:

$$v_s/v = BA/AD$$

$$v_s = v \times BA/AD$$

but, $BA = d/\cos\,\theta$ and $AD = L_s + d\tan\,\theta$.

Thus:

$$v_s = v \times d/\cos\theta \times 1/(L_s + d\tan\theta)$$

Thus:

$$v_s = vd/(d\sin\theta + L_s\cos\theta)$$

However, this assumes uniform laminar flow that is only found in plate settlers or shallow trays. Thus a correction factor k is introduced. The values of k for circular, square, and parallel plate settlers are respectively 4/3, 11/8, and 1. Thus:

$$v_s = kvd/(d\sin\theta + L_s\cos\theta)$$

It is apparent that the average velocity parallel to tubes or plates inclined at an angle of θ will be given by:

$$v = U/\sin\theta$$

where U is equal to the surface-loading rate, or vertical velocity in the tank and v is the average velocity along the tubes/plates.

Table 7.5 shows the effect of plate spacing and effective settling length on v_s. The figures in the body of the table compare the theoretical actual settling velocities to the average surface loading of a tank for plates at an angle of 60°, calculated as percentages. Thus, for a lamella plate installation with 80 mm between plates and a settling length of 2.5 m,

Table 7.5 Decrease in velocities as % of overall surface loading for differing lamella plate spacing and lengths

Effective length of plate: m	Distance between plates: mm			
	100	80	60	40
1	19.7	16.2	12.6	8.6
1.5	13.8	11.3	8.6	5.9
2	10.6	8.6	6.6	4.5
2.5	8.6	7.0	5.3	3.6
3	7.3	5.9	4.5	3.0

Notes:
1. Assumes plates at 60° to horizontal
2. No allowance has been made for correction coefficient
3. No allowance made for edge affects

angled at 60° to the horizontal, the theoretical settling of the slowest particle removed would be 7.0% of the overall surface loading on the tank, allowing a reduction in tank area of 93% to achieve the same degree of particle removal.

The figures in Table 7.5 indicate the importance of both length and plate spacing on the performance of lamella plate systems. However, they need to be treated with a degree of caution. Velocity through the clarifiers is an important factor in the design of highly loaded plate or tube settlers. This needs to be low enough to ensure laminar flow, as discussed above. However, laminar flow is easily achieved with the plate spacings and flow velocities normally encountered.

The figures in Table 7.5 make no allowance for edge effects and thus relate only to very large installations. In practice, the additional space required to accommodate the angle and length of the plates means that the figures overstate the theoretical benefits possible. In addition, a correction coefficient needs to be applied to allow for turbulence in the inlet and outlet zones, for the effects of sludge movement and removal, and for other factors that will lead to actual performance being worse than predicted. However, even if in practice the performance is only half of that indicated above it still represents a significant improvement over conventional clarifiers. In spite of the marked inclination of the tubes it is important to remember that the settling action is governed by horizontal-flow principles which are in no way different from those described in the early part of this chapter.

There are some practical difficulties associated with lamella clarifiers. Such clarifiers only have very short retention times, often of the order of 15 min within the lamella zone. Such short times are very unforgiving in the event of any upsets to the process or changes in raw-water quality. Thus, where such clarifiers are used, the process control will need to be more sophisticated than that required for a plant using conventional clarifiers.

The angle of tilt is critical: too little, and the sludge fails to slide down the plates; too much, and the tube acts on the upward-flow principle, with the benefits due to shallow-depth horizontal-flow action being lost. The material used to manufacture the plates or tubes is of great importance. It needs to be a low-friction material to ensure proper sludge removal. If plastic is used it needs to be resistant to sunlight, it also ideally needs to have good resistance to algal growth, which in practice means that it should be translucent. Counter-current settlers are normally at an angle of around 60°, co-current units can have lower angles (typically 30°) as water movement encourages the sludge to slip down. Co-current units have an arrangement whereby the clarified water is withdrawn at the bottom of the plates prior to the point where the sludge leaves the plates; this is necessary to prevent the sludge being mixed with the clarified water.

A particularly worthwhile benefit of high-rate settlers lies in the ease with which packs can often be inserted into existing basins of conventional design, thus increasing their output capacity without requiring additional land or civil structures. When used in this way the improvements in plant performance will often be very marked as the loading on the modified units will normally be low.

7.8. Dissolved air flotation (DAF)

It can be seen from Equations 7.1, 7.3, and 7.4 that the ‘settling’ velocity will be positive when $\rho_1 > \rho$ and will be negative when $\rho_1 < \rho$. In the first instance, the particles will settle, and in the second instance the particles will rise. Certain suspended solids like algae have a naturally low density and tend to float. If the average density of particles can be reduced to less than that of water by attaching air bubbles to them, then these particles will rise.

DAF involves mixing a flow of water supersaturated with air with a flocculated suspension and introducing the mixed flow into the bottom of a tank. The air comes out of solution, either using the flocs as nuclei onto which the air bubbles form or else becoming attached to flocs as a result of collisions. Once a particle has an average density less than that of water due to attached air, it starts to rise. As it rises the air bubble grows as the pressure decreases and the particle will, therefore, accelerate upwards. The ‘sludge’ accumulates as a foam on the top of the tank and is either scraped or flushed off the top. Clarified water is taken out of the tank at low level.

An important parameter in DAF is the ratio of air to solids. This is determined experimentally but it is useful to understand the background theory. Consider a solid particle with an air bubble attached. The density of the particle and air bubble combined is given by:

$$\rho = (V_a\rho_a + V_s\rho_s)/(V_a + V_s)$$

where ρ = average density of solid particle with air bubble attached, V_a = volume of air bubble, ρ_a = density of air, V_s = volume of solid particle and ρ_s = density of solid particle.

If the masses of air (a) and solid (s) are substituted in the above equation, it can be transformed to:

$$\rho = (1 + a/s)/[1/\rho_s + (a/s)/\rho_a]$$

and then to:

$$a/s = (1 - \rho/\rho_s)/(\rho/\rho_a - 1)$$

For a particle to rise, the density of the particle with air attached must be less than the density of the liquid and the above equation therefore represents the minimum air to solids ratio required. The air density used to calculate the air required should be based on the pressure at the bottom of the tank, and not on atmospheric pressure. Thus, the quantity of air required theoretically depends on the density and concentration of solids in the water and the depth of the tank. In practice, the required air quantity will be determined from on-site testing and will be an adjustable parameter in the full-size plant.

A typical DAF unit is normally rectangular with a water depth of 2–3 m. Loading rates are normally up to around 10 m^3/m^2 h, but can be higher than this. Retention time is around 20 min. The recycle rate is around 6–15% of the flow to be treated and the air requirement is around 6–10 g/m^3 of water treated (Edzwald *et al.*, 1992). The air is compressed to between 4 and 6 bar, depending on the air saturation system employed. The source of the water to be supersaturated is normally the clarified water; in potable-water treatment this is usually of acceptable quality. DAF is always preceded by coagulation and flocculation. A key factor in the performance of DAF units is the size of the flocs. Larger flocs have a lower specific area (the surface area per unit of mass) than smaller flocs. If the floc is too large there is insufficient area for sufficient air to attach, and the flocs may not be removed. Thus as well as careful control of the saturated water flow, which affects the number and size of the bubbles, it is important to control flocculation to optimise floc size. Flocculant aids are therefore not normally used. The development of DAF is one of the factors that have led to a much greater awareness of the importance of coagulation and flocculation in water treatment in the UK. It is normal to have two-stage flocculation with variable speed flocculators prior to DAF.

DAF is a relatively energy-intensive process, because of the need to dissolve air into the supersaturated water. Typical energy requirements are 40–80 Wh/m^3; a 10 Mld DAF plant would use 17–35 kW continuous power.

DAF is a high-rate clarification process with a short detention time in the flotation zone. Thus unless closely controlled the process is susceptible to upsets caused by changing raw-water quality or changes in chemical dosing. Nevertheless it is an increasingly common process. There are several reasons for this.

- The process is generally better at treating water containing algae and light flocs that settle poorly – indeed it requires smaller flocs than would be appropriate for clarifiers.
- The process starts up quickly and can be brought into use within one hour of starting-up. Thus, it is practical to use the process only when required.
- The sludge produced has a relatively low water content.
- Space requirements are less.

7.9. Other processes

There are other clarification processes that may occasionally be encountered in water treatment. Examples include the Sirofloc process, which uses adsorption onto fine magnetite; other processes using adsorption onto other materials, and even coarse membrane filtration processes. These are discussed in Chapter 4.

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Chapter 8
Types of clarifiers

8.1. Introduction

Following coagulation and flocculation, water with the fine suspended material in the raw water coagulated into flocs normally passes to a settlement/clarification phase. In the past, the only process used was some form of settlement basin in which the flocs settled out. Nowadays it is common to use either dissolved air flotation (DAF) or some form of sludge-blanket clarifier; neither of these can be described as settlement basins; both are better described as clarifiers. This chapter considers both traditional settlement basins and the more modern clarification processes. It considers some of the practical aspects of clarifier selection and design and some of the various types of clarifier in more detail. There are two basic types of settling basins: horizontal flow and vertical flow; and two other types of clarifiers: vertical-flow solids contact tanks; and DAF tanks. First horizontal- and vertical-flow tanks are considered, followed by vertical-flow solids contact tanks and then DAF.

Horizontal- and vertical-flow tanks predominantly remove previously flocculated particles; there is some additional flocculation but this is not a major factor. Horizontal-flow tanks can be either rectangular or circular. Simple vertical-flow tanks are normally square hopper-bottomed tanks, and should not be confused with vertical flow solids contact tanks.

Vertical-flow solids contact tanks achieve much higher efficiencies partly by passing flow through a layer of sludge/floc. Flocculation and entrapment of fine particles occurs as the flow passes through the sludge/floc layer and this means that these particles, which would otherwise rise and pass onwards, are retained within the floc layer. They have been developed from hopper-bottomed vertical-flow tanks, which are very deep, and have evolved into flat-bottomed upward-flow clarifiers that have a relatively shallow depth of construction. More sophisticated forms of solids contact tank are those involving the recirculation of sludge to mix with the incoming water, either using internal recirculation within a tank or an external system.

DAF tanks operate in a similar manner to horizontal-flow tanks, save that the particles rise rather than fall.

Pure theory is of limited use in designing a settling basin. For one thing, it is difficult to predict the worst conditions under which the basin will have to operate. Although laboratory tests on a series of samples will give an indication of: the most suitable types of basin and the required coagulant doses; of conditions for optimum floc formation; and of settling velocity, generous factors of safety have to be allowed before the results can safely be applied in practice. Also, although there may be extensive data on the performance of an existing plant which is being extended or modified, it is unusual to have anything other than relatively limited data for the design of new works: in some cases a new plant may be under construction at the same time as the reservoir which will supply the water to be treated. Where there is limited data for the design of a new plant, it may well be possible to use data from other plants in the region that treat water of similar quality.

8.2. Choice of clarifier

In the discussion of clarifiers that follows, the singular when used refers only to the type. No works of any size has less than two clarifiers and most large works have several, the number and size of the units depending in no small measure on the ease with which the chosen design can be scaled up.

The object is to choose a clarifier which will efficiently remove the floc, by settlement, or flotation, and from which sludge can be removed in an appropriate way. Some clarifiers are highly efficient but difficult to clean. Some are ideally suited to small works but do not benefit from scale effects and are unsuitable for larger works. Some work well when handled by experts but are unsuitable for unskilled operators. Some clarifiers will cope with higher silt peaks than others. Some are easy to build and some rather complicated. Some take up more space than others. Although there is a wide choice, there is often one type of clarifier that suits a particular job. Some of the more common types are shown in Table 8.1. This table should be used selectively, as some of the columns may not apply in any given case. In general, the columns on the left will interest engineers in more sophisticated countries and those on the right will be more appropriate in developing areas of the world. This table reflects developments over the past few years. In the UK, virtually all new water-treatment clarifiers use either flat-bottomed clarifiers with a sludge blanket, or DAF, preceded by a separate coagulation/flocculation stage. Elsewhere, other clarifiers are used, and there is of course a wide range of different types of clarifiers in existing plants.

8.3. Potential problems

There are several potential problems with clarifiers that designers should be aware of. These are discussed with respect to horizontal-flow tanks but many also apply to other forms of clarifier. Problems include

- ensuring equal hydraulic loading of tanks

Table 8.1 Factors in selection of settling basins

	Standard of performance in normal conditions	Advantageous use of land area	Effectiveness with algae	Effectiveness on small works	Effectiveness on large works	Ability to cope with sudden changes in water quality	Ability to handle wide range of raw water quality	Extent of use	Performance with unskilled operators	Low maintenance requirements
Conventional horizontal flow	+++	++	++	++	++++	++++	++++	++	++++	+++
Two level horizontal flow	+++	+++	++	+	++++	++++	++++	++	+++	++
Radial horizontal flow	++	+	++	++	+++	+++	+++	+	+++	+++
Plate or tube settlers	+++	++++	+	++++	++++	+	++	+++	+	+++
Hopper bottomed upward flow	++++	++	+++	++++	++++	+++	+++	+++	++	++++
Flat bottomed upward flow	++++	+++	+++	+++	++++	++	+++	++++	+++	+++
Solids recycling clarifier	++++	++++	+++	++++	++++	++	+++	+++	++	++
DAF	++++	++++	++++	+++	++++	+	++	+++	++	++

Scale of + to ++++, low to high benefit

- coincidence of peak output with peak turbidity
- low temperature
- excessive suspended solids
- proper pre-treatment
- liability to streaming
- persistent wind
- overturning of water in the basin.

These are discussed below. Times and figures mentioned normally refer to coagulated, well-flocculated water in which coagulant aids have not been used.

8.3.1 Ensuring equal-flow distribution between tanks

It is clearly foolish to design a set of settling basins to operate at a certain loading and then to provide a design that leads to different tanks within the set operating at different loadings, with some operating at higher loads than others. This is self-evident, but nevertheless it is common to find basins operating at different loadings due to poor hydraulic design. The best simple approach to splitting flows equally between tanks is to provide a proper flow-splitting chamber, or equivalent, using weirs with free discharge to divide flows to the tanks. Such an arrangement is simple and allows automatic redistribution of flows between remaining tanks if one tank is taken out of service. The drawback of such an arrangement is that it requires that a relatively high hydraulic head be available. This has to be provided for in the initial design of a treatment works; it is often the case that engineers are faced with an existing plant with poor flow distribution and insufficient hydraulic head available to introduce a flow-splitting chamber.

Where available hydraulic head is limited, designers may provide a symmetrical layout that controls flow splitting by similar inlet losses in the feed arrangements using the treated-water outlet weir as a control. This is often unsatisfactory and a difference in flow of as much as 40% has been observed for six parallel tanks (Kawamura, 1996). Ensuring that losses in the inlet channels or pipes are low compared to losses in the tank itself can minimise the problems with this method.

It is practical nowadays to accurately measure and control flows using magnetic flowmeters and actuated control valves. However, while this sounds attractive, in practice it is complex and would not normally be considered except for very large works.

8.3.2 Coincidence of peak output with peak turbidity

In some countries, notably those with cold winters and hot summers, the peak demand can rise to very high seasonal peaks, often 50% or more above average. If this seasonal peak coincides with poor raw water quality and high-suspended solids in a river it imposes very arduous conditions because the treatment plant is severely taxed at a time when maximum output has to be maintained. The designer has to pay particular attention to the sizing of clarifiers when such conditions occur. Whereas normally it may be possible to accept a degree of overloading to cope with short-term peak demands, this is clearly less acceptable if peak demands occur at the time of poorest water quality.

8.3.3 Low temperature

Settlement occurs in accordance with Stokes' law (Chapter 7): the downward velocity of the settling particles is inversely proportional to the viscosity of the water, which in turn is inversely proportional to the temperature. Thus, suspended particles sink more slowly in cold water. If poor raw water quality occurs in the winter or spring, when water temperature is low the designer needs to allow for this. Where settling tests are carried out on samples of raw water, allowance must be made for raw water temperature compared to the temperature of the water during the settlement test.

The temperature of surface water commonly falls to near 0°C in the winter in many areas of Europe and North America. Maximum water temperature in the summer in the tropics can commonly be over 30°C. The dynamic viscosity of water at 0°C is 1.79×10^{-3} Pa s and at 33°C is 0.76×10^{-3} Pa s. Thus, there can be a two-fold improvement in clarifier performance in the summer compared to the winter. As water demands are temperature dependent, the variation in water temperature means that clarifiers normally perform better in the summer, a useful factor in coping with peak demands. Freezing can also be a problem in cold climates and it is common to cover clarifiers where this can occur.

8.3.4 Streaming

The phenomenon known as streaming describes a condition in which some or all the incoming water does not mingle with the main bulk of water in the basin but passes rapidly through from inlet to outlet in a fairly well-defined stream. It occurs to some extent in all horizontal-flow basins but is particularly bad on radial-flow basins. Kawamura (1996) quotes an actual detention time of 30–40% of the theoretical in a rectangular sedimentation tank and tests have shown that, in the worst instances, some of the water entering a basin of 4 h capacity actually passes over the outlet weir within a few minutes. When this happens, the whole theory of settlement is clearly inapplicable. This short-circuiting is due to large-scale eddy currents established either by some form of jetting, by the mixing of waters of different densities, or by wind. Generally, at low loadings dead zones are larger and there is a greater degree of short-circuiting. Jetting is normally due to poor design of inlet arrangements; it is commonly associated with rectangular tanks and can be eliminated or minimised by proper design of the tank inlet and by using an appropriately proportioned tank. The design of horizontal tanks to minimise streaming is considered later.

Temperature differences of as little as 0.2°C and a turbidity in the raw water greater than 50 nephelometric turbidity units (NTU) can lead to the establishment of streaming

(Kawamura, 1996). Normally, a sedimentation tank is warmed by the sun and the most common form of streaming is for incoming water, which is denser than the water in the tank as it is colder and more turbid, to drop to the bottom of a tank before rising at the outlet. Judiciously placed baffle walls will minimise large-scale streaming, but provision of a baffle wall often causes problems, including precluding the installation of mechanical scrapers. A basin with two compartments in series is a more effective answer. Some benefit also results from constructing basins that are long in relation to their width.

Where basins are exposed or where there is a steady breeze there can be problems arising from the wind setting up surface currents and eddies. This problem is common in coastal areas where there are sea breezes for much of the day. Settling basins are normally the highest point in a works, as the flow is often pumped up and then allowed to gravitate through the plant. The remedy is to screen or cover the basins.

Streaming is often very marked in radial-flow basins because the ratio of length of flow to width (i.e. $r/2\pi r$) is less than one. Vertical-flow basins suffer from streaming only if badly constructed with the outlet weir not level, causing unequal hydraulic loadings along the weir.

8.3.5 Excessive suspended solids

Aside from causing streaming, excessive suspended solids may give operational problems including excessive sludge production. Excessive suspended solids is not normally a problem in the UK but may be encountered in catchments where vegetation cover has been reduced and soils are easily eroded. The effectiveness of a basin declines if the incoming water contains excessive suspended solids. The maximum suspended solids that an upward-flow basin can normally take in its stride is about 900 mg/l. Rectangular horizontal-flow basins can treat water with higher suspended solids loads than vertical-flow basins. It would be unwise to expose the latter to suspended solids frequently in excess of 1000 mg/l, whereas the former normally cope reasonably well, especially if the hydraulic loading can be decreased by reducing throughput. Circular-flow tanks with rotating scrapers and high-capacity sludge lines can handle very high levels of suspended solids, of up to the order of 20 000 mg/l. However, at such high solid-loadings, tanks would operate as pre-treatment, with high levels of solids in the settled water.

For simple tanks, a commonly used surface overflow rate of 18 $m^3/day/m^2$ together with a basin depth of 3–3.5 m gives a nominal retention time in a horizontal-flow basin of about 4 h. As a preliminary guide this might be varied up or down roughly in proportion to the ratio between the square root of the maximum suspended solids concentration to the square root of 900. For the typical fairly clear UK river (of about 500 mg/l maximum suspended solids) this would give $(500/900)^{1/2} \times 4$ h and so permit the use of a basin of 3 h nominal retention capacity, whereas a fairly turbid tropical water, with 2000 mg/l suspended solids, would require about $(2000/900)^{1/2} \times 4$ h or about 6 h capacity. This rule of thumb has no theoretical basis but has evolved from many practical examples; it is applicable to situations where there is likely to be a poor level of operating ability.

8.3.6 Proper pre-treatment

Where settlement basins are being operated at high loading rates, close to their maximum capacity, it is particularly important that the water has been properly coagulated and flocculated prior to entering the settling basin. This applies particularly to where highly loaded vertical-flow solids contact tanks are used. These tanks depend in part on interaction between flocs in the incoming water and the floc blanket and have short retention times. It is important to maintain uniform water quality and a steady pH value for the process to operate consistently and a higher degree of coagulant control is appropriate.

8.4. Horizontal-flow settlement basins

8.4.1 Introduction

At one time, the rectangular horizontal-flow basin was the most widely used form of clarifier in the world. However, now they are never used in the UK for new plants, and their use is unusual in new large plants in Europe. Rectangular horizontal-flow tanks are still used in North America because their advantages under typical American conditions outweigh their disadvantages. It is an extremely reliable basin and is very popular with plant operators because it rarely gives trouble and can always be relied on to outperform most other types when treating raw water with a high silt load and good performance is at a premium. One of the reasons for their continuing use in the northern USA is the ease with which they can be covered. Their proportions lend themselves to covering with reinforced concrete roof slabs to prevent freezing. In the US, it is normal to have sludge scrapers, as access for cleaning is more difficult for a covered tank. In less extreme climates, they are normally uncovered.

Clarifiers of this form are very easy to build and operate. The main advantages of horizontal-flow tanks are that they are less susceptible to shock loads or process upsets and they are easy to operate and maintain. If they do not perform well they can often be upgraded by adding plate settler modules. They are not ‘temperamental’ and will put up with a lot of inexpert handling. Their considerable size makes it less likely that sudden fluctuations in raw-water quality will affect clarified-water quality. They ‘scale up’ very favourably and are at their most economic on big works. They are also able to treat waters with exceptionally high silt loads: they settle silt quite well, have room to store it and are not too difficult to clean. Their cost per unit of volume is low, and although they are bulky in appearance they are normally very cheap in overall cost. They are, therefore, very good performers on big works on silty rivers and can be operated by relatively untrained staff.

8.4.2 Types

8.4.2.1 Rectangular basins

In its traditional form, a horizontal-flow basin resembles a large oblong box, filled almost to the top with water. The bottom is flat or has only a slight slope and the water is normally 3–4 m deep. Water enters at one end in the lower half of the tank and leaves at the other end over a surface weir. There may be baffles within the main box structure to inhibit short-circuiting. The basins typically have a retention time of at least 3 h, with longer retention times in colder climates or where there is no sludge removal equipment installed.

The rectangular basin is simple but has disadvantages: sludge collection mechanisms are not as simple as for circular tanks; and it requires more land than more sophisticated processes. Consequently, efforts have been made to retain the merits of horizontal-flow while overcoming these problems. Radial-flow tanks (circular in plan), multi-storey tanks and tanks using shallow-depth tube or plate settlers are all variations on the theme of horizontal-flow tanks.

8.4.2.2 Radial-flow basins

There is no fundamental difference in hydraulic design between the rectangular cross-flow tanks and circular-shaped radial-flow tanks. In a radial-flow basin the raw water enters through a central inlet and flows radially outwards towards a continuous peripheral outlet weir. The same values for surface overflow rates are applied and the tanks perform similarly to rectangular basins of the same loading, although, as noted below, short-circuiting is a common problem. Obviously, radial-flow velocities cannot be uniform because the cross-section increases with the radius, but this is not necessarily in itself a weakness, as maximum cross-section and therefore minimum velocity occurs where it is most needed, which is after the more rapidly settling particles have deposited.

Small circular tanks tend to be cheaper to build in concrete than square tanks of the same capacity, but they cover more ground per unit of area because of the wasted space caused by their shape. The outlet weir presents less of a problem because it can be placed right round the outer edge of the tank. Sludge collection uses rotating scrapers, which are relatively cheap and are very effective. However, streaming is a particular nuisance in circular basins because they are basically a poor hydraulic shape, length (i.e. radius) being less than breadth (i.e. perimeter). They are suitable in situations where there is a high silt load, which the rotating scrapers remove most efficiently. The Accelator type of circular upward-flow tank discussed below is superficially similar but has a far more complex design.

It is now unusual to find simple circular horizontal-flow settlement basins used for water treatment; where circular basins are used they tend to be complex proprietary designs incorporating flocculation and clarification enhanced by upflow solids contact in a floc layer.

8.4.3 Multi-storey tanks

It is quite possible to construct conventional horizontal-flow tanks in the form of a structure of two or more storeys. These often operate with the flow entering the lower level and flowing up to the top level, with the outlet above the inlet. Alternatively, they may simply operate as two separate tanks, one constructed above the other. Two level basins were fairly common in some areas of the USA and in South-East Asia. As area is such an important factor in settling, multi-storey tanks are often remarkably cheap and effective. Sludge removal is obviously more complex for a multi-storey tank, with a chain mounted flight scraper or some form of travelling suction device required. Where a tank operates in series, scraping is sometimes confined to the first chamber, because most of the deposits form at the inlet end of the first storey.

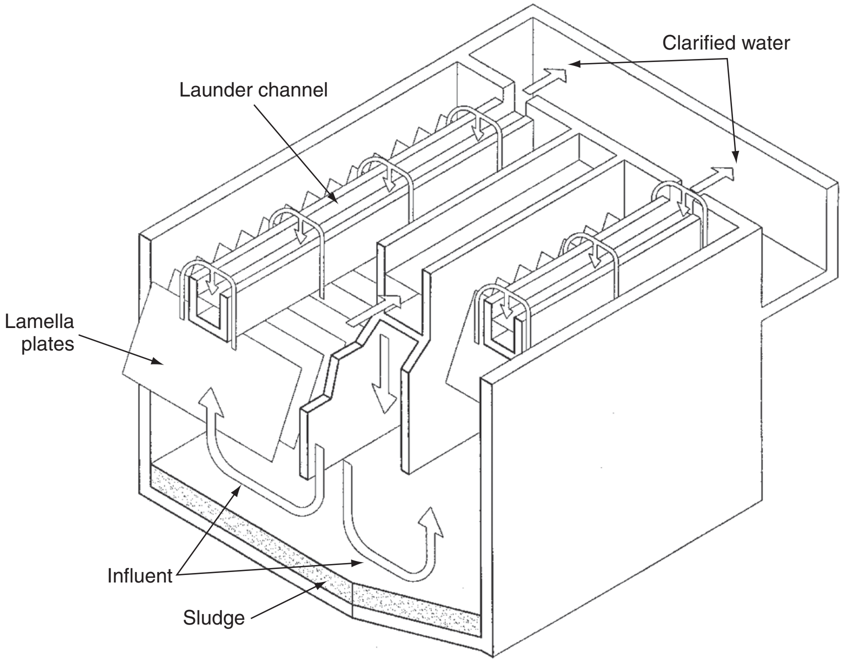
8.4.4 Tube and plate settlers

Hazen and Camp arrived at the logical conclusion (Chapter 7) that if area was the main consideration in settling tank design, the more extensive and shallower the tank the better. However, few, if any, very shallow tanks of great surface area have been built, but this principle has been adopted for the design of tube and plate settlers. Tube and plate settlers are increasingly common, both in new plants and installed to upgrade existing tanks. In principle there is no difference between tube and plate systems. Tube systems are normally hexagonal in cross-section, to permit them to be uniformly nested in blocks. They have the advantage of being more rigid than plates, which means that they deflect less under the weight of the settled sludge, and the rigidity is also useful for large units. They also prevent mass movement of water due to wind effects or temperature differences more effectively than plates. However, plates are easier to clean and are more common in water treatment. Plate systems are more flexible from the point of view of optimisation as it is quite straightforward to adjust the design of the plate spacing to the optimum.

Specialist suppliers normally undertake the detailed design of such units, and there are many factors that affect the details of the units. They are normally fabricated from a suitable plastic or stainless steel. Plastic is lighter and is a low-friction material, which assists in sludge removal. The angle of the plates is typically between 50° and 60° to the horizontal. The smaller the angle the better the settling performance, but the poorer the sludge removal. Occasionally flatter angles may be encountered, but such units will either be co-current settlers, or will require periodic draining down for sludge removal.

Tube settlers are always set below the top water level in the tank, typically around 500 mm, with clarified water collected in a channel through submerged orifices or over a weir. Plate settlers may also be located below top water level, but some designs have

Figure 8.1 Typical arrangement of a lamella separator (courtesy of Black & Veatch)



the plates extending above water level, with the clarified water being taken sideways into collection channels. Because they extend near to the surface of water, plate, and tube, settlers tend to suffer from attached algal growth. This can be reduced by fabricating them from an opaque material to minimise light penetration into the unit, but can only be eliminated by covering the tanks.

A typical diameter for a tube is 75 mm. Plate spacing is typically 50–75 mm. Typical retention times in a purpose built unit are of the order of 15–20 min, but will normally be higher where an existing settlement tank is being upgraded. Overall surface-loading rates of up to 40 m/h are possible with tube or plate settlers. However, care must be taken in evaluating proposals to understand the basis on which the loading rate is calculated. For small installations, tanks will be significantly larger than might be anticipated because a high proportion of the plan area will be effectively unused due to the slope of the settling units (Figure 8.1).

8.5. Design of rectangular horizontal-flow settlement basins

The principles which govern the design of horizontal-flow basins have been covered in Chapter 7.

8.5.1 General

Disadvantages of rectangular horizontal-flow tanks include that they require a relatively large area and are prone to streaming. Thus, the detail design of inlet and outlet arrangements is critical if a tank is to perform well. They do not benefit by the sludge accretion effect, and they need to be preceded by coagulation and flocculation if they are to work effectively. In cases where flocculation has been omitted, floc can often be seen forming part way along the basin. This is a sign of poor-process design and suggests that the performance of the clarifiers could be improved by adding flocculation.

As horizontal tanks do not depend on additional flocculation caused by particle collision, simple settlement theory is applied in their design. Of course, as seen earlier, flocculation does occur, even if it is not designed into the tanks. Tanks are designed to remove particles of the size necessary to provide a water of acceptable quality for the next stage of treatment, normally filtration. Acceptable filter loadings are discussed in Chapter 9, but a reasonable target water quality for clarified water is no more than 5 NTU, and values significantly lower than this are normal at most UK plants.

8.5.2 Loading rates

The first step is to determine the solids concentration in the raw water and the coagulant dose necessary to ensure formation of well-settling floc. This with the design flow rate will permit an estimate to be made of the quantity of solids to be removed by the tank and hence sludge volumes. Jar tests are essential to ascertain the optimum dosage of coagulant and the coagulation pH, the flocculation time, and the advisability of using coagulant aids. They will also indicate approximate surface-loading rates.

A well-formed floc typically settles at between 2 m/h at 0°C and 3.5 m/h at 20°C. Thus the simplest option is to design the tank on the basis of removal of such particles at the appropriate water temperature. Given that generally the highest water demands are in the hot conditions a value of 3 m/h is often used. This represents a surface loading of 3 $m^3/m^2/h$. In practice, a safety factor of at least 2 is generally applied, meaning a surface loading of up to 1.5 $m^3/m^2/h$. This represents a reasonably conservative design loading for a good floc; rates of up to 2.5 $m^3/m^2/h$ are quoted elsewhere but higher rates should be used with caution.

Table 8.2 shows the overflow velocities that are commonly used in designing conventional horizontal-flow tanks. Factors that favour settlement are coarse-grained sediment, high temperatures and low turbidity; factors that hinder it are colloids, cold water, high turbidity, and the coincidence of peak turbidity with peak water demand. It is necessary to look at the worst conditions in each case and decide how bad or easy the situation may be at maximum works output, or, alternatively, how much water has to be produced when the raw water quality is at its worst.

Table 8.2 Typical surface loadings for horizontal-flow tanks

Type	Q/A: $m^3/day/m^2$		
	Very bad conditions	Normal conditions	Easy conditions
Without coagulant aids	9	18	24
With coagulant aids	18	27	36

Alternatively, tests could be made to determine the period required for the water to settle and a suitable design surface-loading rate. In practice, full testing is rarely done but in the absence of plants treating similar water, some testing is clearly prudent if raw water samples are available.

8.5.3 Dimensions of rectangular tanks

All other things being equal, rectangular basins perform better than square basins because they are less prone to short-circuiting. Rectangular tanks with poor inlet arrangements are prone to streaming due to jetting of the incoming flow. This can be eliminated or minimised by proper design of the tank inlet and by using an appropriately proportioned tank. Kawamura (1981) suggests that tanks with a length to breadth ratio of less than two are generally ineffective, and Cox (1964) also suggests a length : breadth ratio of greater than two. Others (Culp *et al.*, 1986) suggest a ratio of three, and where possible this is recommended as a minimum.

The depth of the settling zone is normally about 3 m. It is apparent that for a given depth D the detention time is directly related to the surface overflow rate (Q/A). A depth of 3 m is common for basins up to 60 m in length. Above that a depth/length ratio of 1 : 20 is commonly noted. In either case, if scrapers are not installed the depth should be increased to provide a volume in which sludge can accumulate before removal.

8.5.4 Retention times

Where the climate is temperate, silt loads are low, and there is mechanised sludge removal, horizontal-flow basins generally have a nominal retention time of around 3–4 h. A tank with a water depth of 3.5 m and a surface overflow rate of 28 m/day would have a nominal retention of 3 h, and a tank with a loading rate of 18 m/day would have a retention of 4.7 h. For river-derived supplies under typical UK conditions, nominal retention periods of the order of 3 h would be appropriate.

In many tropical countries, where the rivers tend to be more turbid, basins are commonly of at least 4 h retention capacity. In really difficult cases where there is very high turbidity, a high proportion of colloids or low temperatures, the basins may require a still greater nominal retention capacity but this would be most unusual. All the above retention times assume that coagulant aids are not used. For more difficult tropical sources, pre-sedimentation tanks may also be necessary.

If in doubt settlement basins should be designed conservatively as the extra capacity can normally be provided at low cost at the design stage and it can be difficult and expensive to correct for too small a basin. However, despite this, modern designs of horizontal basins do not normally have the very long retention times sometimes used for older designs, it being preferable to improve settlement by improving coagulation and flocculation and by using a coagulant aid.

8.5.5 Other factors

The water coming into the basin carries fragile floc that must not be broken up as it does not easily re-form. To ensure that the floc is undamaged the velocity in inlet pipes and channels should not exceed 0.6 m/s and should preferably be less under normal operating conditions. An appropriate range is 0.15–0.6 m/s, with the minimum velocity set to minimise deposition of flocs.

The design of the inlet to a horizontal-flow tank is critical to the performance of the tank and it is essential to ensure a uniform and steady-flow distribution across the tank if the tank is to operate to its full potential. Various arrangements can be seen in different textbooks and there is no universally accepted arrangement. The best arrangement is to have an inlet channel running across the complete width of the basin. To ensure even-flow distribution into the tank, flow should enter, from the inlet channel, through a large number of ports distributed across the end wall of the tank. These should be 100–200 mm dia., with a maximum velocity of 0.3 m/s. The area of the inlet ports will typically be of the order of 5% of end wall area. Such an arrangement ensures a good distribution of flow at the inlet across the entire cross-sectional area of the tank. It normally removes any need for additional baffles, allowing easy installation of a sludge scraper. Where there is more than one basin there will need to be a separate flow distribution and isolation arrangement, to ensure equal-flow distribution between tanks and to allow one tank to be taken out of service.

Where the inlet arrangements are simpler, for example, using a weir inlet, a perforated baffle wall should stretch across the full width of the basin about 2 m from the inlet end. It should start just above surface and terminate about 1.5 m below. The velocity through any openings should not exceed 0.2 m/s. It should be possible for much of the water to enter the basin by passing down and under the wall. Kawamura (1981) discusses the location and requirements for baffle walls in detail.

The average horizontal velocity in the basin should not exceed 0.02 m/s but generally the velocity will be well below this.

One of the most important features of any basin is the outlet weir, which is situated at the surface and has a length at least equal to the width of the basin. It has been found that short weirs, with high loadings per unit length, induce currents leading to local high velocities, leading to poor settlement at the outlet end of the tank. In extreme cases at high flows there may be problems with scouring of deposited sludge from the floor of the basin. Making the weir a trough will combat this by increasing the weir length, with water entering over both sides and discharging sideways along the trough. In narrow basins two or more such troughs may be built. For a weir loading of 150 m^3/day/m, the maximum recommended, a single-lipped weir cannot discharge the water passing through a basin designed for a surface overflow rate of 18 m^3/day/m^2 if the basin exceeds 8 m in length, which in practice applies to virtually all horizontal basins. Multiple weirs of some sort, therefore, are normally desirable; they should be as near the outlet end of the tank as possible.

8.5.6 Sludge

As tanks have flat or gently sloping bottoms, removal of sludge cannot be done solely by hydraulic means but requires the installation of sludge scrapers to convey the sludge to hoppers. If sludge is voluminous, mechanical scrapers are necessary; in most countries they would be provided without a second thought. However, in older plants in parts of the world where labour and land are cheap, and where there are no high silt loads, horizontal-flow basins may be found without mechanical sludge removal installed. Such tanks require to be periodically drained down for sludge removal by hand. Where there is no mechanical sludge removal it is necessary to install additional treatment capacity to allow a tank to be taken out of service for cleaning while maintaining plant throughput; it is also appropriate to increase the retention time within the tanks to allow volume for sludge accumulation.

In long, narrow basins the sludge is scraped longitudinally into hoppers at the inlet end. Discharge is under hydraulic head and the sludge will normally have a solids content of 0.5–1.0% solids w/v, although it can be higher. To avoid settlement in the sludge pipes, velocities should exceed 1.4 m/s. Any new horizontal-flow tank should take into account the possibility of future upgrading by the addition of tube or plate settlers, and it would be prudent to provide sludge handling capacity additional to that initially necessary, to cater for this.

8.6. Design of plate and tube settlers

Practical design considerations are that the plate settler should be preceded by a coagulation/flocculation stage, most likely with addition of a coagulant aid. There should also be provision for removal of the coarser particles prior to their entering the settler unit. In practice, providing a suitably designed upward-flow entry to the settlers ensures this; where any heavy particles will settle out. The plates or tubes should be placed so that the base of each bank is at least 1 m above the floor. Where the retention times are short, it is essential that coagulation and flocculation be precisely matched to water quality. This reduces the probability of excessive carry-over of solids at times of changing water quality. If plate settlers are proposed for raw water sources prone to rapid and significant quality changes, it is necessary to pay great attention to coagulation control and flocculation, and lower-loading rates and longer retention times are advisable.

For smaller units sludge can be collected in a hopper and removed hydrostatically. Larger units may use a series of hoppers or a scraper.

Tube and plate settlers are also attractive for upgrading the throughput of existing basins. In such applications it is usually only necessary to increase the performance of basins by a relatively small amount, although in theory it will be possible to achieve at least a four-fold increase in throughput, based on plate surface-loading rates. In order to utilise the full potential of tube or plate settlers in existing tanks, it is often necessary to upgrade the feed and clarified-water pipes, and provide a large increase in sludge handling capacity.

Figure 8.1 shows a typical layout of a clarifier using a plate settler. An example of the design of a plate settler is given in Appendix 1.

8.7. Testing of water for the design of horizontal-flow clarifiers

The first question is whether coagulation is required. This can be assessed by carrying out jar tests or by settling a sample of water in a tall glass cylinder. The cylinder is filled with the water to be tested without the addition of coagulants or stirring. In many quite heavily turbid waters, the samples quickly show a clear dividing line between the upper clarified zone and the lower zone of settled silt. If this period is short and the line of demarcation is well-defined, flocculation is probably not necessary. If the period is lengthy and the zone of junction is blurred, colloids are probably present and flocculation is essential. In practice, coagulation is normally required and jar-testing is necessary to define the conditions under which a well-formed and easily settled floc is obtained. There is an argument for additional testing using a cylinder equal in depth to the proposed basin to take account of the interaction between settling particles. This would need to be a length of pipe of around 3 m long with sampling taps at different levels to permit samples to be withdrawn and analysed. However, in practice such testing is rarely done for several reasons.

- A large representative water sample is required and this may not be available – it is important to test the worst quality water likely to be treated.
- In nearly all cases, flocculation would be carried out prior to settlement, so the water being tested would be coagulated and flocculated prior to testing; in practice it is difficult to do this in a laboratory for large quantities of water.

- The physical requirements for carrying out such testing are not simple, involving a settling column several metres tall.
- Such testing would almost always indicate a higher-loading rate than would be acceptable or used in practice.

Simpler, if time consuming, settling tests in a tall glass cylinder can be used to assess the settling velocity of the particles needed to be removed. After adding water to the cylinder and allowing it to settle for, say, 15 min a sample can be taken from the cylinder at a depth of, say, 500 mm. This can be tested for turbidity or for suspended solids. If the solids content or turbidity were satisfactory then the above figures would mean that removal of particles with a settling velocity of 2 m/h or more would be acceptable from a process standpoint. The test can then be repeated using a shorter or longer settling time to better define the minimum settling velocity of the particles that need to be removed. An appropriate factor of safety, normally three, should be applied to the results from laboratory testing in designing the full-size tank.

8.8. Vertical-flow clarifiers

8.8.1 Introduction

Initially upward-flow clarifiers were square-shaped basins, with an inverted pyramidal-shaped hopper. Flow entered the tank in the hopper and flowed upwards to outlet weirs around the perimeter of the tank. In theory, any particle with a settling velocity greater than the upward-flow velocity would settle into the hopper and be removed. In practice, such tanks were often more efficient than expected. The reason for this is additional flocculation caused by inter-particle collisions leading to improved solids removal. Thus, it became usual to operate such tanks to maintain a blanket of fluidised solids in the tank to improve efficiency.

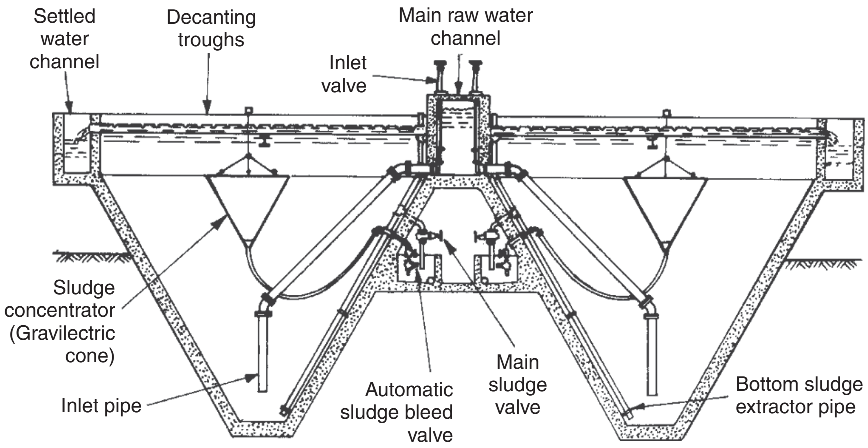
The principle of flocculation due to inter-particle collisions in a sludge blanket led to the development of more sophisticated upflow tanks of many patterns, which are often of proprietary design. Such tanks are often commonly referred to as sludge-blanket clarifiers but a more accurate description would be solids contact clarifiers, as their key feature is the flocculation that occurs due to particle collisions.

8.8.2 Types

8.8.2.1 Hopper-bottomed sludge-blanket basins

Figure 8.2 is a cross-section of a hopper-bottomed upflow basin. In these basins raw water (after addition of coagulant and flash-mixing) is admitted at the bottom of the inverted pyramidal base from where it passes upwards through a zone of fluidised settled floc (the ‘blanket’). The passage of water through the blanket acts to flocculate and entraps the small particles in the feed water and greatly improves clarification. Above the sludge blanket is a layer of clear water in the upper square-shaped portion of the basin; this makes it possible to observe the top of the sludge blanket. Under

Figure 8.2 Hopper-type settling tanks



typical UK conditions, such tanks work well. They are normally designed on the basis of the rate of rise permissible in the upper zone where the plan area is at a maximum. They were originally designed for, and are particularly effective on, water-softening plants. For such applications the permissible maximum upflow rate may be as much as 5 m/h.

In vertical-flow basins, it is usual to maintain an upward water velocity of half of the normal settling velocity of the flocs. As well-formed alum floc settles at about 3 m/h under normal conditions, a typical upward-flow tank (without coagulant aids) is often designed to run with upward-flow velocities of 1.5 m/h. Table 8.3 gives typical ranges of permissible loadings for different sorts of waters, with and without coagulant aids. Where coagulant aids are used lowland river waters can be treated successfully at upward velocities of up to 5 m/h, although the higher values quoted would not be used without confirmation that they would produce clarified water of acceptable quality.

The great merit of these basins is that, under conditions that suit them, they provide both flocculation and settlement, and they give a settled water of great clarity. Sludge can be removed easily without a scraper, using only the available hydraulic head.

Table 8.3 Typical surface loadings for hopper-bottomed vertical-flow clarifiers

Type	*Q/A*: m^3/day/m^2	
	Low-alkalinity upland waters	High-alkalinity lowland waters
Without coagulant aids	20–40	50–80
With coagulant aids	30–60	70–120

The drawback of these tanks is that, due to the slope of the hoppers, typically 60°, as the sidewall length increases they become very deep, imposing an effective upper limit on their size. Thus, large plants need a large number of relatively low-capacity tanks. The hopper-shaped bottom, which penetrates deep into the ground, imposes a limit on the economic dimensions of the square top (a limit which arrives more quickly when the ground is rocky or water-logged). A tank of 5 m by 5 m will have an overall construction depth of the order of 6.5 m, but this will increase to around 10 m for a 10 m by 10 m tank. In practice, bigger installations can be built only by adding an increasing number of fairly small units of standard size, each not exceeding 10 m square. While a reassuringly small total area of basin can still appear on the design drawings, it consists in most cases of a great amount of awkwardly shaped concrete, and the cost is commensurately high. It, therefore, follows that the hopper-bottomed upward-flow tanks were most suitable for smaller works, of less than 45 000 m^3/day.

The basins are normally considered unsuitable for treating waters with the high silt loads found so often in tropical rivers, because they are small and removal of large quantities of heavy sludge poses practical difficulties. They can be troublesome to operate when suspended solids exceed 1000 mg/l and if water more turbid than this occurs frequently horizontal-flow basins for preliminary treatment should also be used.

Another cause for concern regarding the use of hopper-bottomed upward-flow basins is that in hot countries in certain circumstances they have a tendency to ‘roll over’ in the afternoon. This is associated with a rapid increase in temperature of the incoming water and may be made worse by solar heating of the sludge blanket. The result is that the incoming water rises rapidly and the sludge blanket is also lifted. It is generally impossible to prevent changes in incoming water temperature, but shading the tanks will assist in some instances. The use of concentrator cones for sludge removal is also reported to alleviate the problem by permitting better control of the sludge blanket.

In older or unmodified tanks, sludge was drawn off from the lower level of the cone into a chamber beside each basin using valves operated either manually or by a timer. Nowadays, it is normal to use either sludge-blanket detectors to control desludging or sludge concentrator cones. Figure 8.2 shows a more sophisticated type of sludge draw-off, using a concentrator cone.

The concentrator cone is a cone made of a suitable material, often PVC-impregnated nylon. This is suspended within the clarifier with its rim just below the level required for the top of the sludge blanket. Excess sludge flows into the cone. The weight of sludge in the cone is measured, and when it reaches a preset value the drain valve on the cone is operated and sludge is allowed to flow from the cone. This system permits accurate control of the sludge blanket and ensures the maximum concentration of solids in the sludge.

8.8.2.2 Modified hopper-bottom designs

The earlier forms of hopper-bottomed basins, as described above, had inherent weaknesses. They were expensive to build and difficult to operate on silty rivers. However, they operated on a principle that was basically desirable, the upward passage of water through a zone of mature floc particles that captured and retained the fine particles in the incoming water. Subsequent developments have resulted in evolution of advanced designs using the principle of passing the incoming water upwards through a sludge blanket; these operate very effectively while being far easier to construct.

Arguably, the evolution of modern designs took place, although not in strict sequence, in four stages.

- The introduction of the multi-hopper tank.
- Development of the basic flat-bottomed tank upflow clarifier.
- Development of complex flocculator/clarifiers.
- Improvements and enhancements to the basic flat-bottomed tank.

The multi-hopper tank is a single tank with linked hoppers: it is equivalent to a battery of hopper-bottomed tanks with the upper common parts of the walls omitted. The advantage is a saving in civil construction costs. The disadvantage is that hydraulically it is a single tank, rather than a set of individual tanks. This means that it is most important to equally distribute flow between the various hoppers. This is more difficult than dividing flow between separate tanks.

8.8.2.3 Flat-bottomed sludge-blanket clarifiers

The next logical step was the development of flat-bottomed tanks. For such tanks to work two problems have to be overcome: firstly, incoming flow has to be introduced uniformly across the bottom of the tank, to ensure that there is a uniform upflow through the sludge blanket, keeping the particles fluidised; and secondly, there has to be a mechanism for sludge removal while maintaining the sludge blanket. Early flat-bottomed clarifiers were normally circular. This enabled rotating sludge scrapers/collectors to be installed easily, to collect sludge into the centre of the tank. Such tanks would typically have a conical bottom, with a shallow slope towards the centre, but were much less deep than hopper-bottomed tanks. Flow was distributed and collected by radial inlet and outlet channels, with a multiple feed to the bottom of the tank. An alternative design from the USA utilised two channels running round the perimeter of the tank. The outer channel contained the incoming flow, which entered the tank through a series of drop pipes. The inner channel collected clarified water that entered over a weir.

It was difficult for simple flat-bottomed clarifiers to work well, but modern tanks are very effective. Two examples of modern tanks are shown in Figures 8.3 and 8.4.

Figure 8.3 Black & Veatch flat-bottomed clarifier (sludge blanket) (courtesy of Black & Veatch)

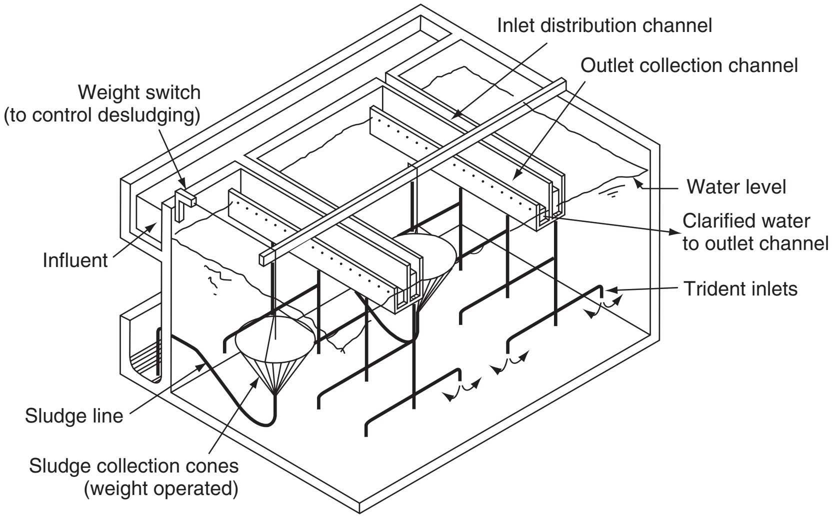


Figure 8.4 Pulsator clarifier (© Degremont). (1) Raw water inlet; (2) decanting troughs for clarified water; (3) sludge removal (controlled by valve); (4) stilling baffles; (5) upper level of sludge blanket; (6) vacuum chamber; (7) vacuum pump; (8) air release valve; (9) raw-water distribution system; (10) sludge thickening zones; (11) chemical addition

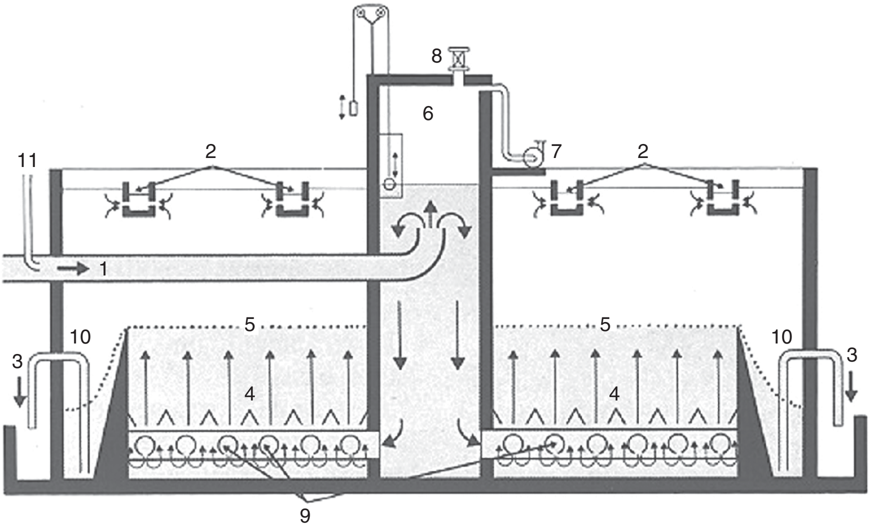


Figure 8.3 shows a Black & Veatch design of a rectangular flat-bottomed clarifier. This is a true flat-bottomed design. Two important aspects of the design are the system for distributing incoming flow to the tank, and the sludge concentrator cones for removing sludge and controlling the level of the sludge blanket. Black & Veatch use ‘Tridents’ to take flow from a series of inlet channels running across the tank and distribute it equally across the tank. Clearly the detailed hydraulic design of the flow-distribution system is critical for equal-flow distribution. The Black & Veatch ‘Gravilectric’ cones are used to collect, concentrate, and remove sludge and to control the level of the sludge blanket.

Figure 8.4 shows an example of a Pulsator clarifier, made by Degrémont. This is another type of upward-flow tank that depends on a sludge blanket for its effectiveness. It also has a flat bottom offering economic civil construction cost. The Pulsator derives its name from the way that the feed water is admitted at varying rates of inflow. Incoming water goes to an airtight chamber from where it is fed to distribution pipework on the bottom of the tank. The inlet chamber is connected to a low-pressure vacuum pump. Air is withdrawn from the chamber at a rate higher than the maximum inflow of water. The water in the inlet chamber rises as air is withdrawn. The vacuum is then released, which causes a surge of water into the distribution pipework before the vacuum is applied again. Thus, there is a period of low flow into the tank followed by a surge, followed again by a period of low flow. The cycle typically takes between 30 and 60 s. The sludge blanket expands during the period of maximum inflow and contracts when inflow diminishes. The movement induced keeps the sludge in suspension and encourages flocculation in the blanket. The tanks are generally rather deep, typically around 5 m. Sludge is decanted over a weir typically placed at about half tank height and can be withdrawn without significantly affecting the quantity of sludge in the blanket. To aid sludge withdrawal there are several draw-offs.

As in all upward-flow tanks, the water is collected by launders placed at regular intervals across the top to collect clarified water at a uniform rate across the tank. New tanks would normally be square or rectangular but the design can be retro-fitted in existing circular tanks.

The two designs shown in Figures 8.3 and 8.4 are typical of the clarifiers being widely used across Europe at present. Such clarifiers also can be used with plate or tube clarifiers above the sludge blanket to allow even higher surface loadings. Typical loading rates are given in Table 8.4. The higher-loading rates quoted apply to clarifiers with lamella or tube settlers installed.

One of the potential problems with flat-bottomed clarifiers is that they are not easy to clean because pipes obstruct the bottom of the tank. While most of the sludge is removed during normal operation there is inevitably a build-up of heavier material in the bottom of the tank. It is therefore necessary to shut the basin down occasionally

Table 8.4 Typical surface loadings for flat-bottomed sludge-blanket clarifiers

Type of water	*Q*/*A*: m^3/day/m^2	
	Low-alkalinity upland waters	High-alkalinity lowland waters
Without coagulant aids	30–60	70–120
With coagulant aids	50–100	100–200

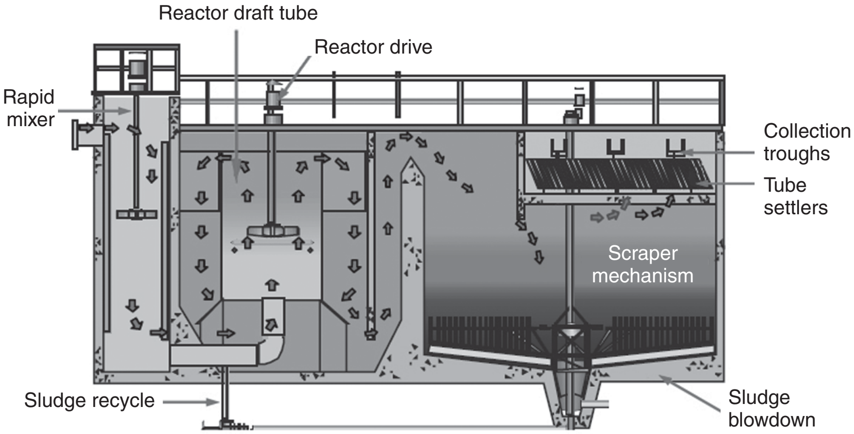
to permit manual cleaning. The frequency of this depends on the nature of the water being treated. Tanks treating soft low-turbidity upland water require cleaning less often than tanks clarifying lowland river water. Where there is likely to be a large volume of heavy sludge the use of other designs of clarifiers may be preferable, or pre-settlement basins could be used.

8.9. Other clarifiers

There are many proprietary designs of circular clarifiers that combine flocculation and upflow solids contact clarification and sludge recirculation. The various proprietary designs have a bewildering variety of names, the name generally intended to summarise the processes employed within the clarifier in a single word. Processes employed normally include some or all of: coagulation, flocculation, settlement, solids contact coagulation, sludge collection, and sludge concentration. The more complex are generally circular with an inner flocculation/coagulation zone and an outer clarification zone. Such clarifiers have been relatively unusual in the UK for potable-water treatment but are more common in North America and Europe. However, some recent UK treatment plants have used solids recirculation clarifiers, comprising several compartments in series, which include coagulation, flocculation and clarification, often with lamella clarification, together with recirculation to the flocculation zone.

The older types consist of a circular tank inside which a smaller circular inner compartment acts as a flocculation chamber. The tank acts as a hybrid between a horizontal-flow and an upward-flow tank, dependent on the depth and diameter of the tank. The flocculated water passes downwards through the central zone and escapes radially outwards underneath the dividing wall (which is suspended from above and nowhere touches the floor). The outer, annular, compartment is sized to keep upward velocities at the levels acceptable for upward-flow tanks and the water is decanted from the top by radial collecting troughs. The floor is generally constructed with a slight slope towards a central drain. A centrally driven rotary scraper, with blades set at angles, pushes the sludge towards the drain, from which it can be withdrawn as required. The scraper sweeps the entire floor area, its horizontal arms being free to pass under the suspended dividing wall. The ability of this sort of basin to handle heavy silt loads is one of its main features and they have been used to treat water from rivers prone to seasonally

Figure 8.5 A Densadeg clarifier (© Copyright of Degrémont)

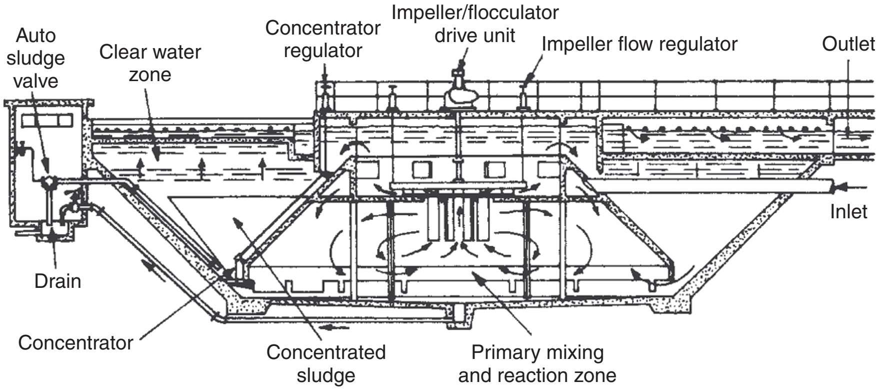


high silt loads. Degremont's Densadeg clarifier is an example of a more modern arrangement, comprising several compartments in series (Figure 8.5).

8.9.1 Accelator type solids contact clarifiers

The Accelator type of clarifier (Figure 8.6) is a more complex clarifier also incorporating sludge recirculation. It is circular in plan with a flat bottom and a diameter of up to over 30 m. It has a conical hood that divides the tank into two zones. In the inner zone, raw water and coagulants are mixed by a centrally mounted impeller before passing into the outer chamber, which acts as an upward-flow solids contact clarifier. The water enters the

Figure 8.6 Accentrifloc clarifier (courtesy of Black & Veatch)



outer hopper-shaped zone and flows upwards to radial launders spaced at regular intervals to ensure uniform upward velocity. The surface area of the outer area is sized to provide the surface loading required.

The key detail of the Accelator design is that sludge passes from the bottom of the outer hopper into the central mixing zone. This recycling of sludge is induced by the movement of the liquid in the central section, which draws sludge in through the openings at the bottom of the hopper. The mixing of the sludge in the central area ensures high-efficiency flocculation. Within the outer hopper there are pockets in which sludge is collected prior to being taken from the reactor.

Clearly such basins are complex, with several variable process parameters within the single tank. Factors include rotation speed, solids concentration, as well as chemical dosing rates and pH value. They require more operator skill than simpler basins, and arguably are better suited to water from reservoirs, which is less prone to rapid changes in water quality than river water.

8.9.2 Water softening

Water softening is considered in Chapter 11.

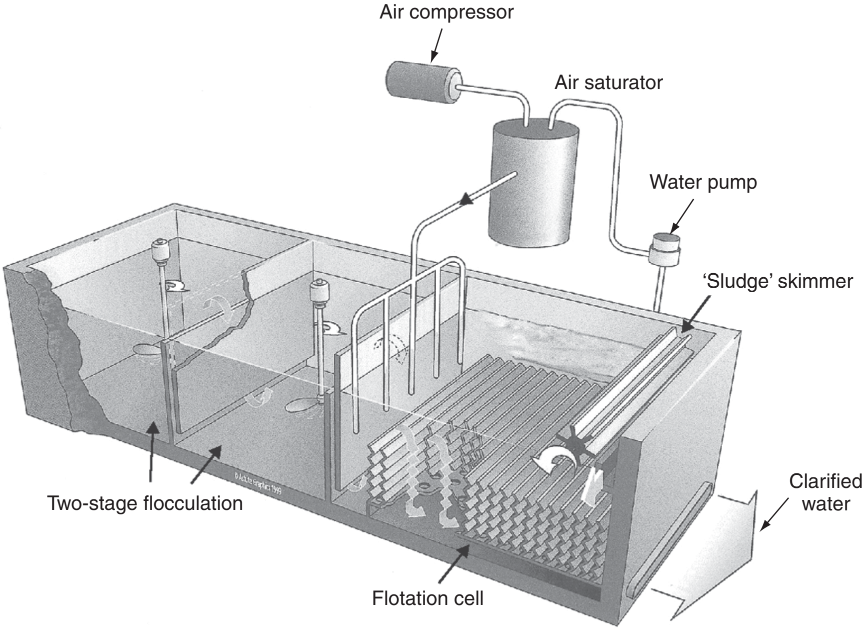
8.10. Dissolved air flotation (DAF)

DAF may either be provided as a separate treatment stage prior to filtration, or it may be installed above rapid gravity filters, combining both clarification and filtration in a single tank. Figures 8.7 and 8.8 show examples of DAF units. Figure 8.7 is a schematic of a DAF Rapide1 DAF unit. The layout shown uses two-stage flocculation followed by a flotation cell. Sludge (foam) is removed by a rotating skimmer that sweeps it into a channel. The flotation cell has plates to ensure the water flows vertically downwards, with clarified water removed at low level.

Figure 8.8 is a schematic of the Black & Veatch CoCo DAFF plant, combining flotation and filtration in a single vessel. The upper part of the unit is a dissolved air flotation unit with the clarified water flowing directly onto a granular filter. There are two waste streams from the unit. The floated sludge is flushed off periodically at high level. Water is introduced from the flushing channel and flows across the top of the unit to the scum channel. Backwash water from the filters is taken off at lower level into the washout channel by way of the washout bay.

Conventional DAF plants operate at high surface-loading rates, within the range 6–15, typically 10 $m^3/m^2/h$ although modern plants can operate in excess of 20 $m^3/m^2/h$ where water quality is good and is stable. Retention times are of the order of 20 min, less at high-loading rates, and tanks are typically 2–3.5 m deep. The basic process can be enhanced to operate at higher rates, for example, Degremont's Aquadaf^{TM} process

Figure 8.7 Purac Rapide1 DAF unit (courtesy of Purac)



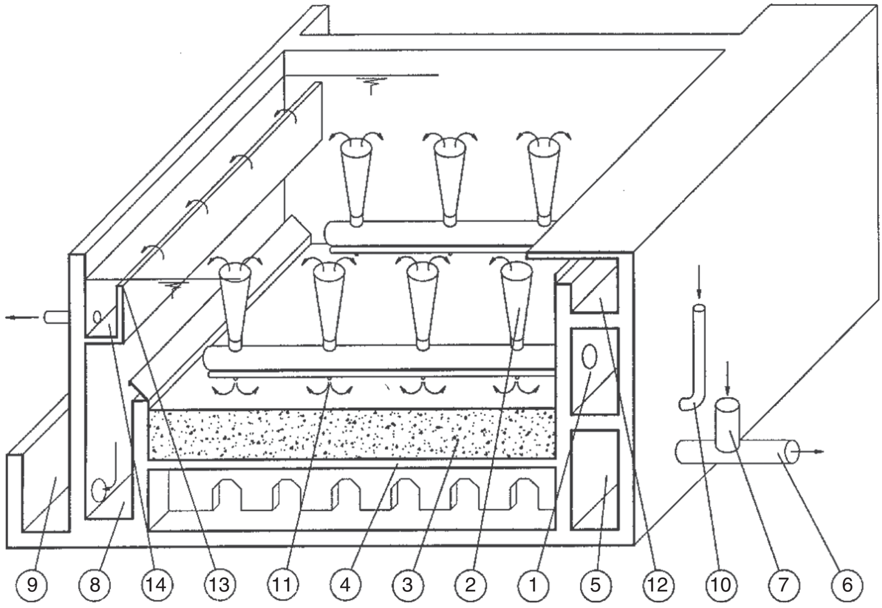
which uses a bubble blanket and improved removal of clarified water to permit significantly higher loadings than a conventional DAF plant.

8.11. Appropriateness of clarification processes

High design loadings are quoted for the many clarifiers of proprietary design. These can be achieved given good conditions, expert control and the use of polyelectrolyte. At the same time it should be appreciated that, even in plants operated by a highly skilled staff with automatic control and excellent laboratory facilities, often it is not possible to operate at the maximum loadings at all times, because of poor or changeable raw water conditions. The higher-loading rates quoted earlier should only be considered for new plants where there is certainty that they are appropriate. Where there is doubt it is better to size for a lower loading, possibly providing for increasing the loading when the plant is expanded and the process has been proved.

It can be argued that the best solution at present for some developing countries lies with simple but well-designed works that can easily be operated by competent but not technically qualified staff designed on the basis of the lower rise rates given in Tables 8.2 and 8.3.

Figure 8.8 Black & Veatch CoCo DAFF unit (courtesy of Black & Veatch). (1) Inlet duct; (2) inlet distribution cones; (3) filter media; (4) filter floor; (5) outlet and upwash duct; (6) outlet pipe; (7) upwash pipe; (8) washout bay; (9) washout channel; (10) air inlet; (11) air distribution nozzles; (12) flushing channel; (13) scum weir; (14) scum channel



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Chapter 9
Filtration

9.1. Introduction

In a conventional water-treatment plant, the filtration stage was often considered the core of the process. The incoming flow to the filters was clearly non-potable, but after disinfection the filtered water was virtually potable water. As water quality standards have developed, this perception is no longer true in many cases, but filtration is still arguably the most important process in most treatment plants, and the process has been developed over the years to produce water meeting higher water quality standards. In the past, filtration referred only to granular filtration, using sand or other granular material. Nowadays, where their use is appropriate, non-granular filters are becoming increasingly popular. This chapter mainly considers granular filtration in its various forms.

Basically, the process of filtration consists of passing water through a granular bed, of sand or other suitable medium, at low speed. The media retains most solid matter while permitting the water to pass, and the filtrate from a filter performing well will be crystal clear with a turbidity of around 0.1 nephelometric turbidity units (NTU). Except for treatment of high quality groundwaters, filtration almost always follows a clarification stage. Thus there is a trade-off between the performance of the clarification and the performance of the filters; the more effective the clarification the less the filters have to do and *vice versa*.

In order to achieve the required quality of filtered water, filtration has to remove particles far smaller than the sizes of the openings between the filtering media. Table 9.1 gives the relative sizes of sand grains and some of the particles present in water. A key requirement in water treatment is the removal of *Cryptosporidium* oocysts, which have a typical dimension of 5 μm, compared to a media particle size of 400–1500 μm (0.4–1.5 mm). Thus, the processes involved in granular filtration are more complex than simple straining. The processes involved are discussed in more detail later.

9.2. Types of granular filter

There are three basic types of granular filter. It is useful to understand the differences between them before discussing them in more detail. The types are

Table 9.1 Relative size of sand grains and suspended matter

Material	~Particle diameter: μm
Sand	800
Soil	1–100
Cryptosporidium oocysts	5
Bacteria	0.3–3
Viruses	0.005–0.01
Floc particles	100–2000

- slow sand filters
- rapid gravity filters
- pressure filters.

Slow sand filters are the oldest form of filters. They are ‘slow’ because they operate at low-loading rates. They use fine sand as a medium and treat the water using two processes: physical straining and biological action. They have a layer of biological growth on top of the sand and in the uppermost part of the sand bed. This layer is vital to their effective operation and, until it develops, slow sand filters provide only minimal treatment. Filters are cleaned at periods of between several weeks and several months, by scraping of the top layer of sand and biological growth.

Rapid gravity filters operate at far higher loading rates. To do this they use coarser media with a higher permeability. They treat the water by physical treatment alone, although the media can be granular activated carbon (GAC), which also adsorbs some chemicals dissolved in the water being filtered. Coagulation is normally required to ensure that smaller particles can be removed more effectively. Simple filters use a single medium, normally sand, but multimedia filters, which use two or more types of media, are common. Rapid gravity filters are cleaned by reversing the flow of water through them, assisted normally by the use of air, to wash out the dirt, a process known as backwashing.

Pressure filters are a form of rapid gravity filters, the only difference being that they operate under pressure in large closed vessels. Traditionally they have been used on groundwater sources where water was pumped directly from a borehole directly through the filter into distribution, without any need for re-pumping. In such situations, there was no clarification prior to filtration. They are sometimes used for surface water sources, normally on small plants.

The latter two types of filters depend solely on physical processes to remove solids, but slow sand filters also depend on biological action.

9.3. Granular filtration theory

The first important point about granular filtration is that simple straining is neither the most important method of solids removal nor is it desirable that it should be. If removal is predominantly by straining, the result is that the filter rapidly becomes blinded, with an impermeable layer of matter on the surface of the sand. This results in the filter requiring frequent cleaning. This, in practice, means that there should be few particles in the water to be filtered larger than 20% of the size of the media. It, therefore, follows that it is important that such particles are removed during clarification, or where there is coagulation but no preceding clarification stage, that flocculation should not aim to produce large flocs, as these would very rapidly blind the filters.

Removal other than by straining is not straightforward. There are a number of mechanisms for transporting and attaching the particles to be removed to the media. Consider the idealised situation where the filter media is single size spherical particles. The first question is whether the flow is laminar or turbulent. The Reynolds number is given by:

$$Re = vDp/\mu$$

where D is the particle diameter and v is the velocity. However, where the flow is between particles with a porosity of f, the equation is modified to:

$$Re = vDp/(1 - f)\mu$$

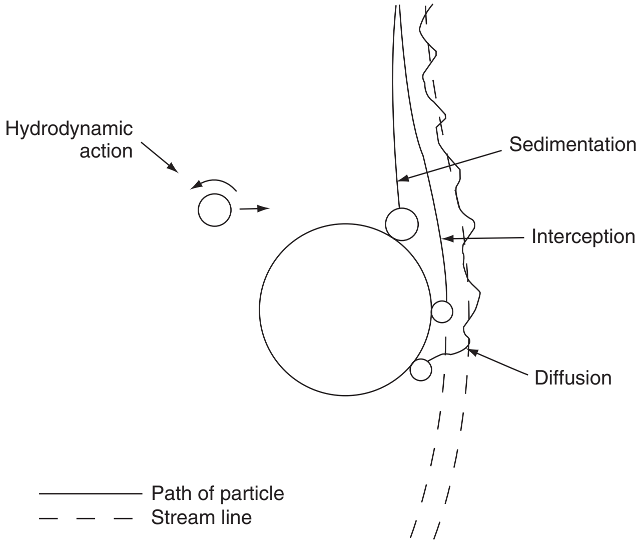
Typically, in water treatment D is of the order of 1 mm and v is of the order of 6 m/h. f is typically around 0.4, giving a Reynolds number of 2.8 at a temperature of 10°C. This is well inside the laminar range.

9.3.1 Transport mechanisms

Five different mechanisms have been identified (James, 1985) whereby particles in a flow of water through a filter may come into contact with a particle of filter media. Four of these are illustrated in Figure 9.1. These are the five mechanisms.

- *Interception*. Where a particle moving uniformly collides with a grain of filter media. The probability of this will be proportional to d_p/d_m, the ratio of the particle diameter to the media diameter. Thus the efficiency of interception can be increased by decreasing the size of the media, or increasing the size of the particle to be removed. For any given condition larger particles will be removed more efficiently.
- *Sedimentation*. This takes account of gravitational forces on the particles to be removed. For any particle the efficiency of removal will depend on the ratio v_s/v, where v_s is the settling velocity of the particle and v is the velocity of the fluid approaching the media. From Equation 7.3, it can be seen that v_s is proportional

Figure 9.1 Collision mechanisms in a granular filter



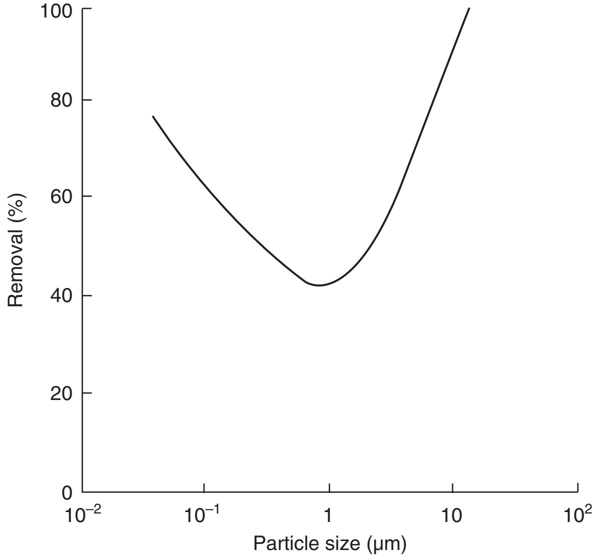
to the density difference and the diameter of the particle squared. Thus, dense particles will be removed more effectively, and larger particles will be removed very much more effectively.

- *Diffusion*. This is Brownian motion, which affects significantly only very small particles. The number of collisions is proportional to $(T/d_\mathrm{p}d_\mathrm{m}v)^{0.67}$. As would be expected, the importance of diffusion increases at higher temperatures, and for smaller particles and media.
- *Hydrodynamic action*. This arises from the velocity gradient in the vicinity of the grains of filter media. A small particle passing a grain of filter media will tend to be rotated by the velocity gradient. This will cause pressure differences across the particle, which will move it towards the filter media. This is believed not to be significant.
- *Inertia (impaction)*. The inertia of a particle heading for a collision with a grain of media will lead to a collision unless the hydrodynamic forces divert the particle to one side. At the low velocities and Reynolds numbers experienced in water filtration inertia is not considered a significant mechanism.

9.3.2 Attachment

Once a particle has made contact with the filtering media it needs to be retained. Provided there is no electrostatic repulsion this is effected by inter-particle attraction (Van der Waal's force), particularly for smaller particles. Where the particles are electrically

Figure 9.2 Particle removal efficiency in a granular filter – typical



charged there will be no attachment except to areas of opposite charge. It is important that charge neutralisation in any preceding coagulation has been effective.

9.4. Practical aspects

In practice, for a given set of conditions the efficiency of particle removal varies with particle size. Figure 9.2 shows a typical variation in particle removal efficiency with particle size. This shows that relatively large and relatively small particles are removed more efficiently than particles of around 1 μm. Very small particles are removed predominantly through diffusion and large particles are removed by straining. Particles around 1 μm are removed largely by interception and sedimentation, and these processes are less effective.

9.5. Coagulation prior to filtration

Filtration is used to remove the particles that are in the water. If the feed water is clarified water then the particles will normally be very fine, possibly with some flocs that have been carried over from the clarification stage. Sometimes, when the raw water has a low solid content, the clarification stage may be omitted and the water passes directly to filtration. In either case, it is common to dose a coagulant immediately prior to filtration. It is not sensible to dose a metal-based coagulant as this would itself impose a heavy solids loading on the filters, leading to rapid blinding and short filter runs. Thus, the coagulant normally used is a polymer dosed at a low rate.

Rapid mixing is required where the coagulant is dosed, but it is normally not necessary to flocculate prior to filtration as there is sufficient agitation between the dosing point and the filters for sufficient flocculation to take place, and large flocs are not desired. The coagulant used prior to filtration is commonly referred to as a filter aid. Filtration of water without prior clarification is often referred to as direct filtration. In the UK, where there is a risk of *Cryptosporidium* oocysts being present in the raw water, direct filtration is usually considered unacceptable as the sole main treatment process.

9.6. Rapid sand filters

Rapid gravity filters are the most common type of filter. They normally treat clarified water, although sometimes they are used for treatment of high quality raw water without prior clarification. Ideally they should be supplied with incoming water of less than 5 NTU of turbidity when they will produce excellent results. They may still perform reasonably well when the turbidity of the incoming water is 10–20 NTU but high turbidities will lead to much shorter run times and are not desirable. Operation at still higher turbidities may be possible but is unacceptable under anything but emergency conditions. A well-designed and operated rapid gravity filter should produce water with a turbidity of around 0.1 NTU. The feed water to rapid gravity filters is often dosed with a polymer coagulant to improve the removal of the fine particles carried over from clarification.

9.6.1 *Cryptosporidium*

In the late 1980s, there was increasing concern over a number of incidents of cryptosporidiosis associated with the presence of *Cryptosporidium* oocysts in public water supplies. As a result, the British government established a group of experts under Sir John Badenoch to investigate the problem and recommend how best to address it. The Badenoch group produced two reports (Badenoch, 1990, 1995) that made a number of recommendations relating to the operation of water-filtration plants. These are the key recommendations relating to filtration.

- Individual filters should be continuously monitored for filtered water turbidity and be backwashed in the event of the turbidity of the filtered water increasing.
- After bringing a filter back into service after backwashing there should be a slow start facility, and preferably an initial period of filtering to waste.
- Sudden changes in filtration rate should be avoided.

After an outbreak of cryptosporidiosis associated with groundwater in the late 1990s, the 'group of experts' was re-established, with Professor Ian Bouchier as chairman, to review developments since the Badenoch reports. The Bouchier report (1998) made several recommendations relating to water treatment, stressing the need to carefully monitor coagulation, clarification, and filtration, and to avoid bypassing treatment processes.

The recommendations arising from these three reports have greatly influenced water-treatment plant design and operation, particularly filtration processes, in the UK, and have meant that some filtration systems formerly used are no longer considered acceptable for new plants.

In 1999, the British government issued regulations requiring that any water-treatment plant considered at risk of having *Cryptosporidium* in treated water should either be continuously monitored for *Cryptosporidium* or should install a filtration system capable of removing particles down to 1 μm. This requirement was revoked in 2007, replaced by a more general requirement to undertake risk assessments and where necessary mitigate the risk by appropriate treatment.

9.6.2 Construction

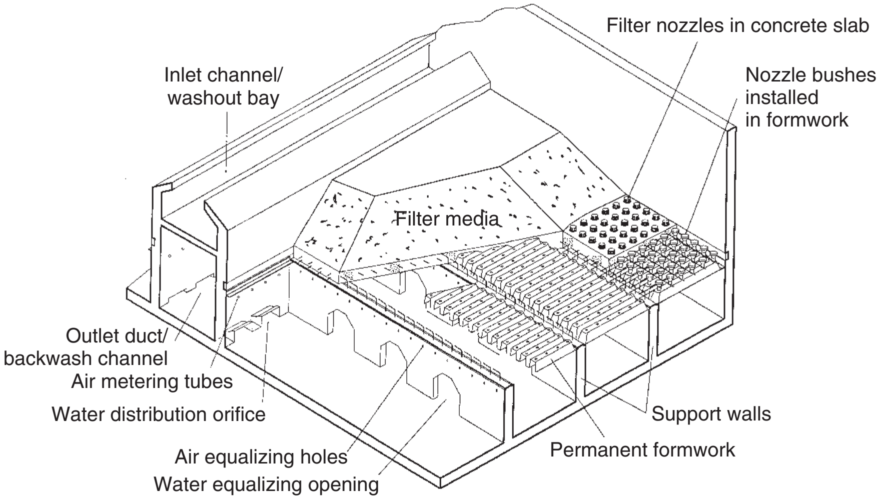
Rapid gravity filters are normally constructed in an open concrete structure, with a building and pipe gallery at one end. The filter is basically a bed of sand, or other media, through which water is passed. This is supported on a bed of graded gravel. Beneath the gravel there is a system of underdrains that collects the filtered water and is also used to introduce air and water for cleaning the filter. The underdrain system has to collect filtered water uniformly across the filter during filtration and distribute air and backwash water uniformly during washing. These systems have to be carefully designed and constructed to ensure they do this efficiently.

Filter bottoms (collectors, nozzles, porous plates, etc.) and the general underdrain system are normally supplied by specialist manufacturers and differ widely in detail. The basic requirement is that they should consist of a large number of orifices, uniformly arranged to collect filtrate from, and deliver air and washwater to, the underside of the filter bed. At the same time, they should not permit any filter media to pass. It is essential that the whole filter floor and nozzles be level. Although not essential, nozzle systems normally use a gravel layer to support the filter media. If the nozzles are designed with openings small enough to prevent media entering, it is necessary to have a very large number of nozzles to limit velocities and head losses through the nozzles; this is not normally practicable.

The graded gravel supports the media whilst also preventing media passing into the underdrain system. It also serves to equalise velocities across the layer, to ensure that there are no problems with high water velocities in the media adjacent to the nozzles. The nozzles typically have an opening of around 0.5 mm and are spaced across the floor of the filter to ensure a uniform-flow distribution across the filter. The detailed design of nozzles is quite a complex task and is normally done by specialist suppliers.

There are several different sorts of systems for the construction of rapid gravity filters. One common system uses a plenum underneath the filter floor for the collection of

Figure 9.3 Rapid gravity filter using a plenum floor (courtesy of Black & Veatch)

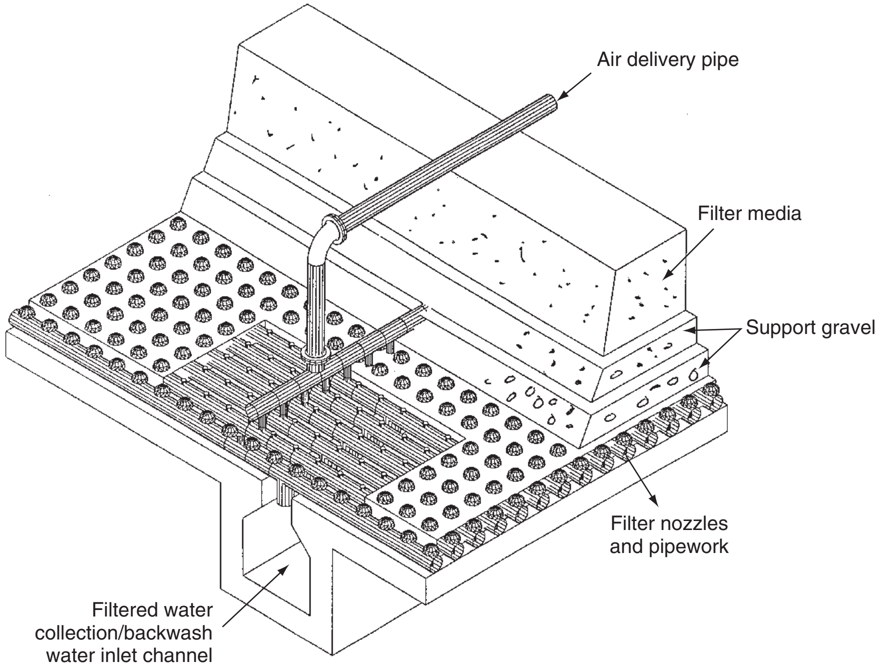


filtered water and the introduction of air and backwash water for cleaning. Figure 9.3 shows such a system, whereas Figure 9.4 shows a detail of a piped system. A plenum system provides an even distribution of air during cleaning and is less prone to operating difficulties than a piped system; it also allows access to the underneath of the filter. There are a variety of different systems used for rapid gravity filter underdrainage; earlier systems used filter tiles sitting on a solid concrete floor. Figure 9.5 shows a modern equivalent of this system manufactured by Johnson Screens. This uses semi-cylindrical channels fixed to the floor of the filter. The channels have a ‘vee’ section wire affixed, allowing an opening of down to 0.15 mm. The channels are installed at a close spacing. The combination of a small aperture and good coverage of the floor means that the gravel layer is not required and sand can be placed directly on the channels.

Normal practice now is to locate pipes in a pipe gallery at one end of the filter, with a building above to house control and instrumentation equipment. The gallery will normally house some or all of the following pipelines with associated valving

- unfiltered water feed (more usually located at the other end of the filters)
- filtered water, normally including flow measurement
- drain (connected to the filtered water outlets to allow filtration to waste)
- backwash water feed
- air feed
- backwash water outlet (connected to a drain)

Figure 9.4 Detail of rapid gravity filter showing piped system of under drains (courtesy of Black & Veatch)

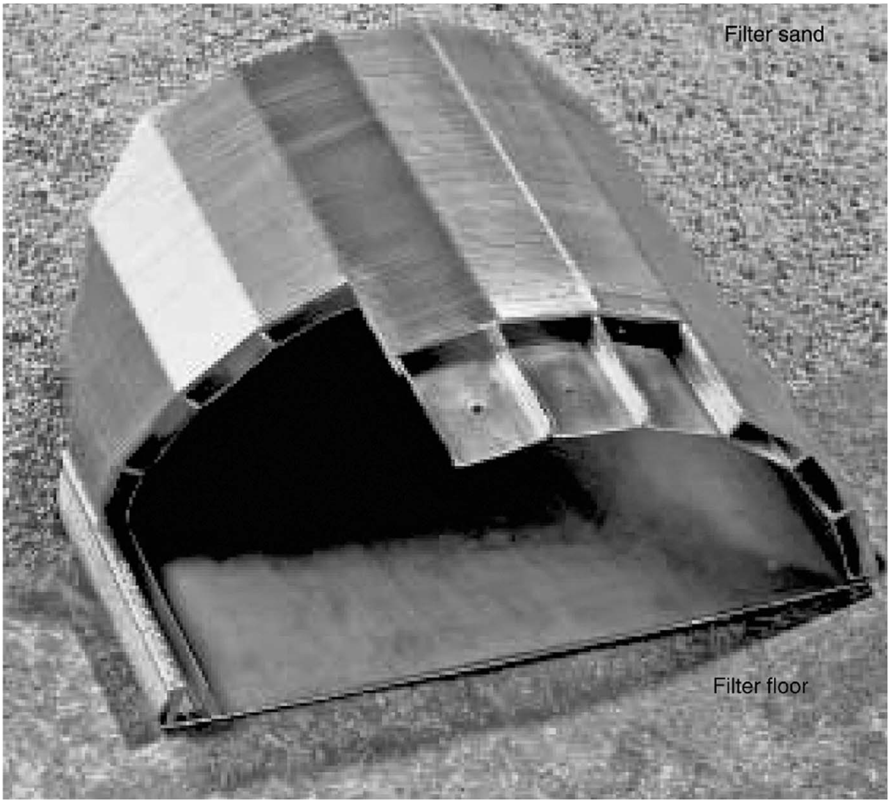


- sample lines (small diameter supplies for turbidity meters and particle counters).

The gallery will also contain flow measurement equipment and valves. As a consequence the pipework in the gallery is complex and crowded.

The backwashing process effectively limits the size of individual filters. Backwashing involves introducing a high upward flow of water through the media, normally in combination with air, to break up the surface crust. The backwash flow is normally pumped. Large filters require very large pumps and pipes to handle the high flows and there are limits on the distance water can travel to a backwash outlet while maintaining effective cleaning. For these reasons, filters rarely have an area greater than 100 m^2, with an upper limit of around 150 m^2. Depending on water temperature and media size, a filter of 150 m^2 can require a backwash flow of up to around 4000 m^3/h, implying a backwash pipe diameter of around 800 mm. The proportions of the filter are affected by the need to ensure good flow distribution and to minimise the length of the backwash water-collection troughs.

Figure 9.5 Detail of Johnson Triton™ filter underdrain system (courtesy of Johnson Screens)



9.6.3 Loading rates

The hydraulic loading rate on filters depends on the quality of the feed water, water temperature, and the media used. For many years, normal loading rates for sand filters were around 6 $m^3/m^2/h$. Nowadays, typical rates are 6–8 $m^3/m^2/h$. Higher rates are possible with dual media filters (up to around 12 $m^3/m^2/h$), for coarse filters for manganese removal (up to around 15 $m^3/m^2/h$), or if the feed water is of a consistent and high quality. The loading rate selected has to take account of one filter in a plant being out of service and another filter backwashing. This situation would represent a normal worst case for loading. It is prudent not to design to the higher loadings quoted above without being sure that the filters will perform adequately under all circumstances, particularly given concerns over *Cryptosporidium* removal.

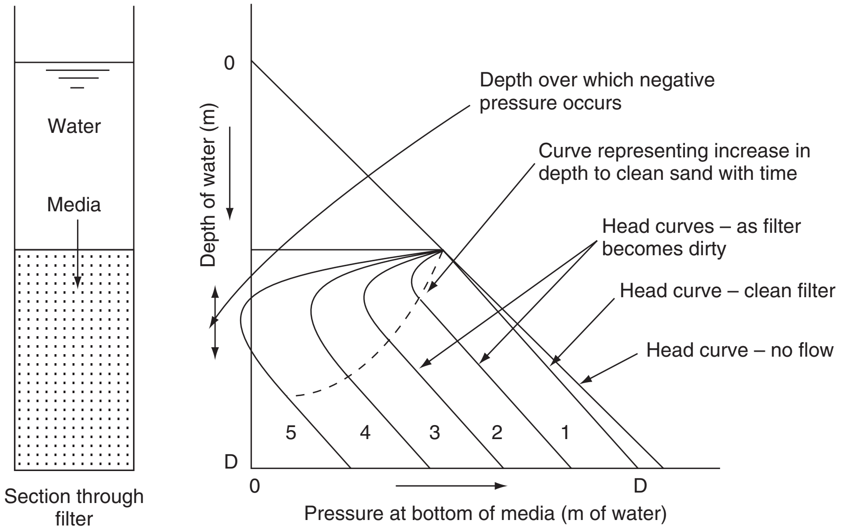
9.6.4 Filter hydraulics

A clean filter typically has a head loss of around 0.3 m under design conditions. Once the head loss reaches around 1.5–2 m, the filter will be cleaned by backwashing. The depth of water over the filter when backwashing is required is governed by the downstream hydraulics and the need to avoid negative pressures developing in the media, see below. However, typically the downstream hydraulic control will be set at around the top of the media, and thus the depth of water over the media will be a maximum of around 2 m.

9.6.5 Development of sub-atmospheric pressure within a gravity filter

In the early part of a filter run, when the media is clean, the particles being removed are intercepted high in the sand bed, but as the top sand gets dirtier the particles penetrate more deeply and the loss of head increases. If the head loss in the sand at any point exceeds the static head of water on the filter, sub-atmospheric pressure will be induced which may cause dissolved air to be given off from the water and result in air blinding of the filter. In poorly designed or operated filters, this tends to happen at around the time that the filter needs washing. The reason for this can be seen from Figure 9.6. The easiest way to ensure that this cannot happen is to have a level control on the filtered

Figure 9.6 Development of negative pressure in rapid gravity filter



Lines 1 to 5 represent the changes in pressure through the filter as the media becomes blinded. Line 5 results in the development of negative pressures within the media.

water outlet such that negative pressures cannot develop within the bed. The provision of an outlet weir with a cill level at the level of the top of the filter media achieves this.

9.6.6 Backwashing

Filters can often run for several days without washing. In modern plants, the washing operation is automated and is triggered either: at predetermined times; by loss of head; or by excessive filtered water turbidity. The washing of a filter takes about 20 min. Normally, filters are set to backwash at regular timed intervals, typically 24 h, but if there is a deterioration in feed water quality, backwashing may be triggered sooner by differential head loss, or by increased turbidity of the filtered water. At the start of the backwashing cycle, the filter inlet is closed and the water level in the filter drops. After a short period, a drain valve is opened to lower the level of water to the level of the backwash outlet weir. Once this level is reached, backwashing with air and/or water starts. Backwashing uses filtered water and is of key importance in maintaining filter performance. The particular backwashing regime adopted for a filter depends on the media used, filter layout, and the filter supplier's preferences. Backwashing may or may not fluidise the bed of media and may or may not use air as well as water.

Washing with water alone is not very effective in breaking up the crust that tends to form on the surface of the media. If washing is ineffective, the surface crust may crack and fragments can penetrate into the media to form mudballs. Once mudballs have become established in a filter they are difficult to clear. The use of air is particularly effective in preventing mudball formation and washing with water alone is now increasingly rare. In North America backwashing is sometimes combined with a surface wash system that directs water jets on to the filter surface during backwashing to break up the surface crust. Signs of concern in a filter are surface cracks, a tendency for the sand to shrink away from the concrete walls, and non-uniform surface turbulence during the washing process.

Initially, rapid gravity filters were backwashed solely by pumping water up through the filter. For this to be effective the media has to be fluidised, typically involving a bed expansion of 15–20%, although in some circumstances expansion of up to 30% or more may be used. A 500 mm depth of sand could be expanded by up to 150 mm. The amount of water required to do this is dependent on the media size and density and water temperature. Lesser expansions are often quoted but these will not give fully effective cleaning using water alone. The backwash rate required for a given bed expansion is inversely proportional to the viscosity of water; thus it can be deduced from the viscosity data in Table 7.1 that at 20°C for a given bed expansion, a backwash rate approximately 50% higher will be required than for a water temperature of 4°C. Thus it is essential to be able to adjust backwash rates to take account of water temperature. Following a period of intensive washing, the backwash flow may be reduced for a rinsing period to carry dirt removed from the media into the backwash drainage system.

Modern filters use air and water for backwashing. The most common sequence is first to pump air through the bed to break up the surface layer, which has normally become blinded with filtered material. After a short time, typically 2 or 3 min, water is also pumped through the filter at a low rate to provide a limited bed expansion (typically less than 5%) and to carry dirt out of the media. This continues for around 5 min. The air scour will then be stopped and the water flow increased to rinse the media and carry the dirt into the backwash drainage system. Other sequences may be employed. Where a low-density media is used, air and water may be used separately. Multimedia filters and GAC filters should normally be fully fluidised during backwashing; this is to ensure that, if any mixing takes place, the media is restored to its initial position in the bed after backwashing.

Sometimes an additional flushing stage is employed at the end of the backwash cycle. This involves allowing clarified water into the filter above the media. The water is introduced by way of the filter inlet and it passes across the filter to the backwash outlet. Its purpose is to remove the dirty water above the media more quickly and effectively than can be achieved by the normal backwashing process. For this to be effective, the backwash trough has to be located on the far side of the filter from the inlet.

When the filter has been washed, the filter is returned to service. As a result of the recommendations of the Badenoch reports (1990, 1995) on *Cryptosporidium*, in the UK it is now considered essential to have slow start after bringing a filter back into operation, and where possible initially to filter to waste for a short period. This is because the filter takes a while to settle down, and the first water through the filter carries out some of the material disturbed but not removed during backwashing. In order to minimise the magnitude of the period of relatively high turbidity in the filtered water, it is essential to have efficient backwashing, and this has led to a tendency to lengthen backwash periods.

Flow rates during backwashing are very dependent on the bed expansion required, temperature, and the type and size of media. Typical airflow rates are 50–80 $m^3/h/m^2$. Water-flow rates can vary between 10 and 30 $m^3/h/m^2$. The amount of washwater used is typically about 1.5–2.5% of the total daily throughput.

It is during backwashing that the maximum practical dimensions of the filter assume importance, as a large flow of water has to be transported to, up and across, and then away from a filter. During backwashing the bed expands, lifting the surface. The weir over which the washwater escapes has to be higher than the bed level when expanded or there will be loss of sand.

9.6.7 Media

Traditionally rapid gravity filters have used sand as the main filtering media. This is then supported on layers of larger material. The size of the media is defined using three key parameters.

- *Size range* – the maximum and minimum aperture sizes, the larger passing all the media and the smaller passing none of the media.
- *Effective size* – the aperture size that retains 90% by weight of the media.
- *Uniformity coefficient* – the size of the aperture through which 60% of the media will pass divided by the size of the aperture through which 10% of the media will pass. This is an indication of the distribution of particle size in the media.

The other key parameter is the specific gravity, and there are also other parameters used to define the shape and porosity of media.

For single media filters the most commonly used media is silica sand with a specific gravity of 2.65 and a size of 0.5–1.0 mm. This normally has an effective size of around 0.5–0.6 mm and a uniformity coefficient of around 1.5. The sand should be sharp, hard, clean, and siliceous. The size of the media used will depend partly on filter loading rate and partly on the quality of the water being filtered. At higher-loading rates the effective size may be increased to 0.7–0.8 mm. The sand layer normally has a thickness of 0.5 to 0.75 m. A rule of thumb for the depth of media in rapid gravity filters (Kawamura, 1996) is that the depth of the media divided by the effective size should be equal to or greater than 1000. Thus if media with an effective size of 0.6 mm were used the bed itself should be 600 mm thick. The supporting gravel bed typically has a total depth of about 450 mm, making the total thickness of the filter media around 1 m. The supporting gravel normally consists of a layer of coarse sand or fine gravel on top of two or three layers of gravel; a typical size range is 2–50 mm.

A problem with single media beds is that during backwashing the bed is agitated and then allowed to settle. During backwashing the smaller particles tend to rise more than the larger ones, and when backwashing stops the larger sand particles settle faster than the smaller ones. As a result after backwashing the sand tends to become graded, with larger particles at the bottom and finer particles on top. Because of this characteristic, the filter does not follow the natural order of water-treatment procedure. In all other processes, the bigger impurities are removed first. The single media filter, with its finest layer at the surface, tends to remove all impurities at one fell swoop. Clearly this is undesirable not only because the top layer of the media does all the work, but also, because the void spaces in the finer media are relatively small, there is limited capacity to store the entrapped impurities, and therefore the filter rapidly gets blinded. This is the opposite of what is required for long and effective filter runs.

There are several possible ways to overcome this. The problem would be reduced if the media were composed of identical grains, but in practice this is not possible. Another way is to have upward-flow filters, and there are examples found of such filters. The most common method of overcoming the problem is to have dual filter media; the combination of materials most commonly used is anthracite and sand. Anthracite has a specific gravity of approximately 1.6 and sand grains typically have a specific gravity of 2.65. This difference in specific gravities permits the use of anthracite particles larger than the sand used and means that provided the bed is fully fluidised during backwashing, the anthracite particles sink more slowly after washing and remain on top. The top layer of the filter is then composed of coarser media. There are no hard and fast rules for the thicknesses of sand and anthracite. Typical dual media filter would have 400–600 mm of anthracite of effective size 1.5 mm on top of 200–400 mm of sand with an effective size of 0.6 mm. The use of such dual media ensures a slower development of head loss with time as there is less blinding of the surface layer of media. Sometimes a third layer, of dense garnet, is included, but this is generally now accepted as having insufficient benefits to offset the additional complication and expense.

Apart from sand, it is quite common to find GAC used as a filter medium. This has the advantage of providing adsorption as well as filtration. The effectiveness of GAC as an adsorber is dependent on the contact time between the water and the GAC. In rapid gravity filters the contact time is relatively short, and this limits the effectiveness of GAC in filters. GAC adsorption is covered in Chapter 11.

While it is common practice to state the working rate of filters in terms of flow through a unit of surface area in a given time (e.g. $m^3/m^2/h$), it should be remembered that filter performance depends essentially on adherence of the impurities to the surface of the sand grains themselves. The smaller the grain size of sand, the greater is the surface area per volume of media. It, therefore, follows that the depth of the sand bed and fineness of media also contribute to filter efficiency.

9.7. Modes of operation

It is normal to provide a minimum of six filters. This provides sufficient flexibility to cater for one filter being out of service and another backwashing. A minimum of four filters should be provided. Filters can be operated in several modes.

9.7.1 Declining rate filtration

Under this method of operation, all filters have the same inlet water level and the only control on filtration rate is the hydraulics of the filter media and pipework. Thus, a filter that has just been back-washed will take a higher flow than the other filters. To prevent flow rates being too high there should be an adjustable valve on the outlet side of the filter to limit flow to a maximum rate, typically around 130% of the design average flow. When the filters are clean there will be a low head loss across the filter, and upstream water levels will be low. Over time the head loss across all the filters will increase and filtration rates fall slowly. Once the maximum possible head loss is achieved, filtration rates will decrease more rapidly. In order to allow for this, it is necessary to provide either an overflow weir for excess water to be diverted away from the filters, or throttling of raw water flow to limit flow to the capacity of the filters. The filters will be backwashed either at regular periods or when the minimum acceptable filtration rate is reached. There are drawbacks with this method of operation. When the filters are operating at design flow and, with a feed water of constant quality, the system works well. However, for example, if the system is operating at 50% of design flow, a newly cleaned filter will take a disproportionate proportion of the flow being treated. This is because, although the flow will be restricted to 130%, say, of design flow, where the flow is 50% of design a newly washed filter could be taking 260% of the average flow. Thus, when there are seasonal variations in flow, the hydraulic restriction on maximum flow should ideally be re-set. Also a deterioration in the quality of the feed water leads to an increase in the rate of head loss development, and this can cause problems if several filters then need to be washed to maintain throughput. Although used in the past, this method of operation is arguably no longer acceptable in the UK, not least because of concerns over the effects of high and uncontrolled filtration rates on removal efficiencies of *Cryptosporidium*. The use of declining rate filtration is discouraged in many US states (Kawamura, 1996).

9.7.2 Constant rate filtration

Constant rate filtration is now the conventional mode of filter operation. This can be achieved either by control of the influent flow, or by control of the outlet flow. In either case the head loss through the filters increases as the filters become clogged.

Where influent control is used, flow is divided between the filters in service by means of inlet weirs. This ensures that all filters receive equal flows. A weir on the outlet side controls minimum water level in the filters. Over time, the head loss across the filter increases and the water level in the filter increases. Filter backwashing is normally initiated on individual filters by either: the head loss across the filter reaching a pre-set level; filtered water turbidity reaching a pre-set value; or the filter having operated for a pre-set time.

Outlet control is used either to control the flows through filters, or to maintain a constant water level in the filters. Where it is used to maintain water level, flow is divided between filters by means of a weir inlet to each filter. The water level in each filter is then maintained at a more or less constant level by throttling a valve on the outlet. In the past, this was done using a float-controlled valve, which led to some variation in water level, dependent on the amount of throttling required. Now this is often done using a level detector to control a motorised valve.

However, the usual method for new filters is to provide flow control on the filter outlets to divide flow between operational filters. The flow into each operating filter is uncontrolled, with all filters having the same water level. Each filter outlet is equipped with a flowmeter and a flow control valve that is used to control flow through each filter. The bank of filters has a controller that calculates total flow and allocates it equally between operating filters.

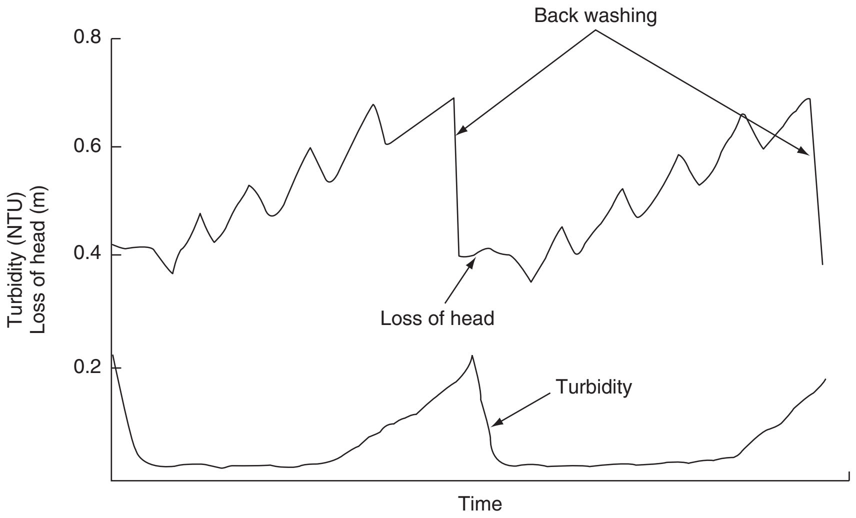
The controller also measures water level on the inlet side of the filters and maintains this at a constant level. Changes in flow rate, which would otherwise change the water level on the upstream side, are compensated for by increasing or decreasing flow through all the filters. Under this control philosophy, the increase in head loss through the filter with time is balanced by opening the flow control valve more to compensate. This method of control requires accurate flow measurement for each filter and a PLC controller. It is the method most favoured at present. Having accurate flow measurement and a PLC controller provides a great deal of flexibility in filter operation. In particular, it permits a controlled increase in filtration rate after a filter has been backwashed.

9.7.3 Performance

In practice a well-designed and operated rapid gravity filter will produce water of low turbidity for an extended period before there is increasing turbidity and the filter requires backwashing. After backwashing, a filter will initially produce filtered water with a relatively high turbidity, of up to 0.5–1 NTU. This should drop rapidly to around 0.2 NTU or less within around 30 min and to around 0.1 NTU after a further hour or so. Figure 9.7 shows the shape of a typical plot of turbidity with time for the filtered water from an individual filter, as well as the development of head loss. These are some points to note.

- When the filter is first returned to service after backwashing there is often high turbidity associated with the flushing out of material disturbed by backwashing that was not fully removed. The size of this peak partly depends on backwashing

Figure 9.7 Typical plots of filtered water turbidity and loss of head over time



efficiency. Where possible it is now normal to initially divert the first flow of filtered water to waste to ensure this first flush does not enter supply.

- The filter will then produce low-turbidity water until turbidity begins to rise as the filtering capacity of the media is approached. Once the turbidity reaches a pre-set value, or when the head loss across the filter reaches a pre-set value, the filter is backwashed.
- The intermittent loss of head peaks that can often be seen on a typical loss of head plot are associated with the back-washing of other filters. This results in a short term increased loading and may lead to increased turbidity in the filtered water, particularly when the filter is approaching backwashing.

The conventional rapid gravity filter consists of two basic systems: the filtering system and the backwash system. For modern plants there is also a sophisticated control system. In the UK, the costs and complexity of the current designs are accepted and acceptable. However, there have been attempts to reduce the cost and complexity of rapid gravity filters, particularly for poorer parts of the world.

A common way of saving costs on the backwash system is to construct an elevated tank to provide backwash water without the use of high capacity backwash pumps. The tank is filled either by a supply from the treated water main leaving the site if this is a pumped supply, or by a dedicated small pump if water flows to supply under gravity. This saves the cost and complexity of a large backwash pump. The pressure of the washwater required at the filter nozzles is about 0.6 bar. The top water level of the washwater tanks would normally require being a minimum of about 9 m above the filter bed level. The tank would be sized to be able to wash two or more filters. A drawback is that large filters require very large flow rates, involving large diameter pipes from the elevated tank to the filters. Nevertheless, for smaller plants providing backwash water from an elevated tank is economic and practical.

9.8. Pressure filters

Pressure filters have many of the characteristics of the rapid gravity type but are enclosed in steel pressure vessels and are normally used where hydraulic conditions in the system make their adoption desirable. They can be installed, for instance, at any point in a pressure pipeline without unduly interfering with the hydraulic gradient, and their use often eliminates the need for double pumping. They are most commonly used where the water being treated is groundwater of a relatively good quality that does not require clarification. Often, pressure filters on groundwater sources are used for removal of iron and manganese in addition to lowering turbidity. Where pressure filters are used on surface water sources they tend to treat good-quality soft upland waters from impounding reservoirs, which may also require iron or manganese removal. Such plants are common in the North of England. Pressure filters are also often used on smaller works or where simplicity of operation is advantageous.

There is no theoretical difference between the operation of a rapid gravity filter and a pressure filter. Rates of flow, criteria for washing and most other factors remain similar, although the maximum head loss through a pressure filter may be higher. Compared to rapid gravity filters, major differences are that it is not possible to see what is going on, flow splitting between filters is normally unregulated, and any chemical dosing and coagulation has to be done under pressure, in a baffled pressure vessel if needed. In a typical pressure filter installation, there is a noticeable absence of pre-treatment and flow controllers and, as these have considerable value, it follows that pressure filters must be working at a disadvantage.

The filter shells are commonly of 2.4 m dia. but can be up to 3.5 m dia., and can be installed vertically or horizontally. Horizontal units (Figure 9.8) are normally encountered in large installations where the smaller sand area in a vertical shell (5 m^2) would necessitate too many units. The length of a horizontal unit does not normally exceed about 15 m and thus the unit has $2.4 \times 15\ m^2 = 36\ m^2$ of media area.

Theoretically, the round shell of a horizontal filter constricts the disposition of the underdrains and detracts from the efficiency of the filter, but this is not significant in practice.

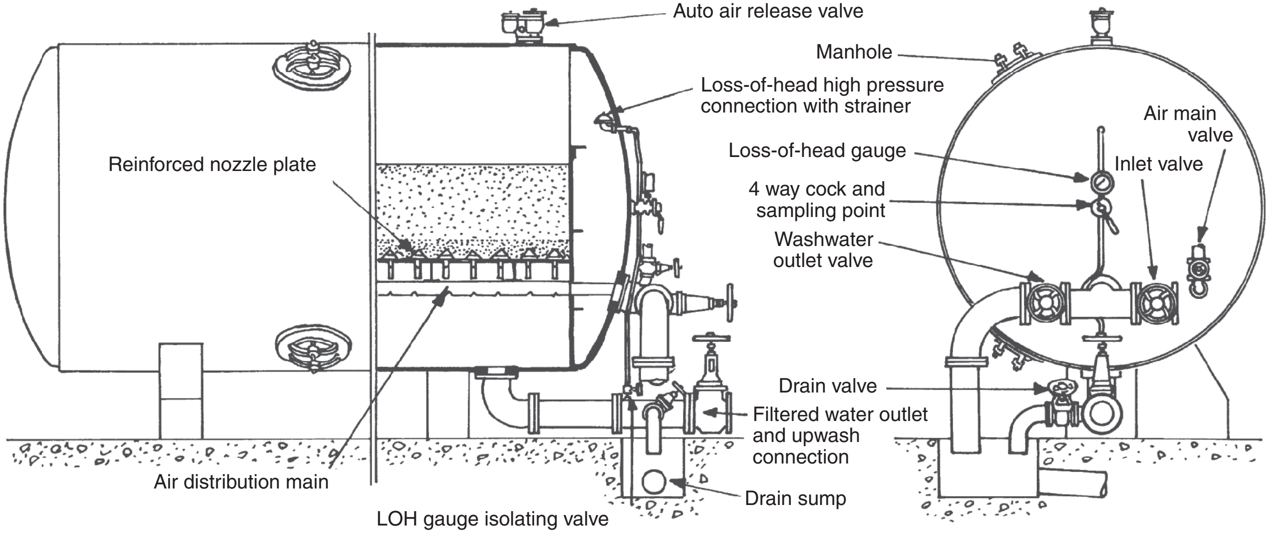
Pressure filters are suitable for the removal of iron up to around 10 mg/l, and for removal of low levels of manganese and are commonly used for this in small- or medium-sized plants. This is covered in Chapter 11.

9.9. Slow sand filters

Slow sand filters are the original type of sand filters and have a successful history going back to the early years of the nineteenth century. Slow sand filters were first used in London in 1820 and there are still many big cities (including London and Bristol) where they still treat much of the water supplied. They work without coagulants and are often found on reservoir- or lake-derived supplies. They can produce excellent quality water, but use is declining because the process takes up a lot of space and can be labour-intensive to operate. They can work for short periods or at low-loading rates with incoming water of up to 30 NTU turbidity but ideally require an average turbidity of not more than 10 NTU. The slow sand filtration process has been extensively researched recently and a notable improvement has been the inclusion of a layer of GAC within the media. Slow sand filters have considerable merit, and in countries where land and labour are cheap, and chemicals are relatively expensive; the possibility of using them should not be overlooked.

The principle of operation of slow sand filters is different from that of rapid filters, the main difference being that the filtering action mostly takes place at or near the surface of the sand. On the sand surface, a biologically active mat comprising: debris from the water being treated, algae, protozoa, bacteria, and other organisms, is established.

Figure 9.8 Horizontal air-scoured pressure sand filter (courtesy of Black & Veatch)



This is known as the ‘schmutzdecke’ (sounding more impressive in German than its English translation of ‘layer of dirt’). It is within this mat that much of the treatment takes place, with suspended and dissolved matter including micro-organisms being removed by both physical and biological action. However, treatment extends 300 mm or so into the filter, in a heterotrophic zone, where organisms use the nutrients in the water as a source of energy. Thus, slow sand filters are emphatically not mere filters and they do not provide acceptable treatment until the schmutzdecke has become established. A mature slow sand filter also has attached algae growing in the water above the sand and there is some treatment here, before water passes through the filter.

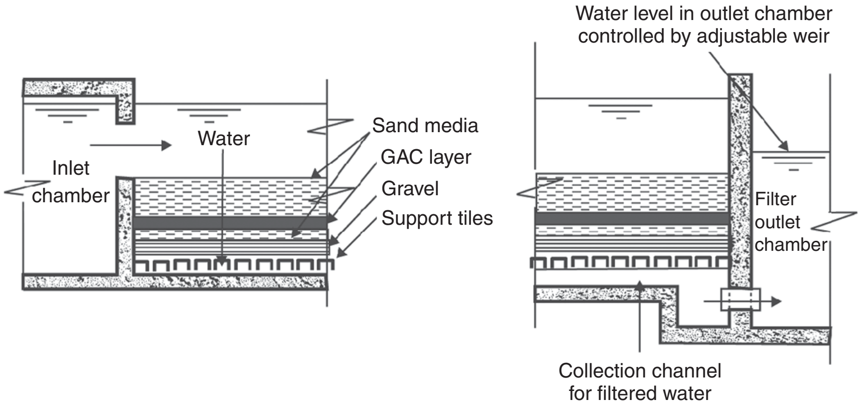
A slow sand filter is brought into service by filling it with water. This is done by filling upwards, introducing water beneath the bed in order to reduce the risk of air being trapped in the fine sand. It is then operated at a low rate for a period of between 3 and 10 days, with the filtrate being either discharged to waste or recycled within the works. During this period the schmutzdecke forms and ‘ripens’. Once this has happened the filter can be brought into normal service. In the summer, when there is more light and the water is warmer the filter ripens more quickly.

The filter typically will then operate for a period of between two weeks and up to three months, the length of the run depending on the quality of the water being treated, the time of year, and the filter loading rate. In the summer, runs are much shorter than in the winter due to there being more light and a warmer water temperature, both of which encourage algal growth. Covering a filter to reduce the intensity of the light reaching the filter dramatically increases run times. Cleaning is carried out by skimming a depth of around 20 mm of sand off the top of the filter. This is done by taking a filter out of service, draining it down, and then using plant or hand labour to skim off the sand. After cleaning, the filter is then restarted as described above.

Once the thickness of the sand layer has been reduced to 400–500 mm by repeated cleaning, the filter has to be re-sanded. Removing more sand than in a normal cleaning does this, around 150 mm of sand, and then replacing sand up to the design maximum thickness. Sand removed from the filters can be washed and re-used if economic.

Slow sand filters are of simple construction (Figure 9.9). A deep bed of sand rests on graded gravel, within which underdrains of open-jointed tiles are buried. The sand is carried to the full depth of the bed near the outer walls and no pipes should be laid within 0.6 m of the walls, to avoid water short-circuiting down the walls and through the gravel to the underdrains. The sand is normally finer than that in a rapid filter, and its quality and grading are less exacting (Table 9.2), so it is more likely to be found locally. It is vital to have sufficient filters to ensure that when a filter is out of service the increased loading on the remaining filters is acceptable. Except on very small works there should never be fewer than four filters, and six is a reasonable minimum number.

Figure 9.9 Slow sand filter



Each filter requires a method of measuring and controlling the flow through that filter. From a control point of view it is easiest to measure and control flow on the outlet side, with the inlets of the filters all at the same water level. If inlet valves were to be used for controlling flow through the filters there would be problems caused by the long response time; this is because both the water level above the filters and as a consequence the flow rate through the filter would change slowly in response to changes in incoming flow. Manual control is quite practicable because of the slow development of head loss. If flow measurement is not provided then the alternative is weir inlets to each filter, which under normal operating conditions would provide an equal loading to each filter. During ripening of a filter, throttling back the outlet valve would cause the water level in the filter to back up and drown the inlet weir. On the outlet side, there needs to be provision for diverting flow to waste during the ripening period and for draining down the filter. There also needs to be provision for filling the filters with filtered water via the underdrains.

Table 9.2 Typical slow sand filter construction

	Depth: m	Grading: mm
Water	1.2–1.8	–
Sand	0.8–1.2	0.2–0.4
Fine gravel	0.05	5–10
Medium gravel	0.05	10–25
Coarse gravel	0.15	10–80
Underdrains	–	–

As cleaning the filter can mean taking a filter out of service for up to 2 weeks, if manual cleaning is used, it is important to monitor filter status carefully, and plan cleaning so that throughput can be maintained, particularly in the summer when demand is normally highest and filter runs are shortest. Taking a filter out of service immediately increases the loading on the remaining filters and the danger is that this will then mean that another filter will then require cleaning. Because of the significant time lag between taking a filter out of service and it being back in operation, there is a danger of filters cascading rapidly out of service, causing major production problems.

Traditionally, slow sand filters have been loaded at a rate of around 2–5 m^3/m^2/day (0.08–0.2 m/h). However, if the raw water is of a high quality and provided they are managed properly they can operate at average rates of up to around 12 m^3/m^2/day. The slow sand filter works serving London have been upgraded by using rapid gravity filters as roughing filters for pre-treatment, and the slow sand filters now operate at this high rate at times of peak demand. To achieve such high loadings a turbidity of less than 5 NTU is required for the water being treated.

The head loss through a clean filter is small, as little as 50 mm. The maximum head loss before cleaning is required is normally taken as around 1 m. Adjusting the outlets to increase the head available over the sand provides the increased head required as the filters become clogged.

The merits of slow sand filters are that the quality of the filtrate is high, well below 1 NTU, and essentially all harmful bacteria are removed; no chemicals are required or even desirable; the filters can be built with local materials often using local sand; and operation is easy. The need for pre-treatment depends on the quality of the raw water and the loading rate applied to the filters. Where filters are to be operated at high loading rates or will treat waters with high turbidity or suspended solids, pre-treatment is required. This is normally microstrainers or rapid gravity filtration to remove turbidity but where directly abstracted river water is used a settlement stage may be necessary to remove silt. Water with a turbidity of up to 30 NTU can be put directly on to the filters at rates up to 5 m/day. (Such values are higher than for most reservoir-derived waters, for which slow sand filters are especially suited.)

The disadvantages of slow sand filters are that they cover a large area and land costs may be high; they do not deal effectively with highly turbid water, nor will they always remove colour; algal growths cause operational trouble; and, unless sand skimming is mechanised, the filters are labour intensive.

A significant improvement to slow sand filters has been to include a layer of GAC within the sand layer to adsorb organic chemicals. This would typically be around 150 mm thick, providing a contact time of 30 min at a loading rate of 7.2 m/day. This is normally sufficient for lowering pesticide and colour to acceptable values. The GAC has to be replaced at regular intervals and this would normally be done when the bed is re-sanded. A further possible improvement is to cover the filters. This greatly extends filter runs.

Slow sand filters have undergone a dramatic reassessment over recent years. They have long been recognised as very suitable for use on reservoir- or lake-derived supplies for small communities where technical supervision is lacking. However, they tended to be regarded as old-fashioned technology, requiring large areas of valuable land. Recently this impression has changed. By upgrading pre-treatment, they can be operated at higher rates and by including GAC in the media water suppliers can utilise them for pesticide removal, avoiding the cost of constructing dedicated GAC adsorbers. In the Netherlands, slow sand filtration is used as a polishing stage in plants designed to produce biologically stable water with very low biologically assimilable organic carbon content.

9.10. Other filters

Traditionally, water-treatment plants have used the filters considered above. It is likely that rapid gravity filters will continue to be the filtration process used in most treatment works. Such plants are flexible, and well understood and trusted. There are, however, a large number of other forms of filter that may be encountered. Not all of these are approved for potable-water treatment in the UK. The ‘Dynasand’ filter is a sand filter employing a moving bed of sand. This is constructed as a steel cylinder. Clean sand is introduced at the top of the filter. Water to be filtered enters at the bottom of the filter and flows upwards. At the same time the bed of sand is slowly moving down, with dirty sand being removed from the base of the filter by an air lift pump. This sand is cleaned and returned to the top of the filter. These filters are suitable for pre-treatment of water but are not acceptable in the UK as the main filtration process because of concerns that they are more likely to permit the passage of oocysts. They can operate at rates up to around 15 $m^3/m^2/h$.

A system that offered significantly lower capital costs used small cells and a travelling bridge to backwash individual cells. The filter was constructed from cells approximately 500 mm wide and typically 10–15 m long. The number of cells was set by the filtration capacity required. Backwashing used a travelling bridge on which was mounted the backwashing pump and a hood the same size as a single cell. To backwash the filter, the hood was positioned over the first cell and the pump started. The pump drew water upwards through the cell and discharged into a backwash channel at one end of the filter. After several minutes of backwashing the pump was stopped, and the bridge moved on to position the hood over the next cell. The pump was then re-started and that cell washed. During backwashing the rest of the cells continued to filter as normal. The system has several problems: there is no control over filtration rates of individual cells; it is not possible to filter to waste or provide a slow start after backwashing; and there is concern that at the end of backwashing raw water from above the filter may pass directly through the filter as the bed settles. For these reasons, although the system is used successfully at a number of drinking water plants, in the UK it is generally no longer considered acceptable for new plants, because of concerns over the opportunity for *Cryptosporidium* oocysts to pass through the filter cells during and immediately after backwashing.

Other filters use mats or fibres to remove fine solids. One design uses a large number of fibres that are twisted around a central cylinder. Flow passes through the mat of fibres into the cylinder. The efficiency of such systems improves as material is deposited on the filter. When the head loss reaches a preset value filtering is stopped and the filters are untwisted, allowing dirt to be flushed from the loose fibres. A filter of this sort can remove particles down to below 5 μm, achieving better than a 99% removal of *Cryptosporidium* oocysts.

Diatomite filters are compact, high-efficiency filters that are suitable for armies in the field, swimming pools, and for meeting short-term emergencies. They are small and portable, and depend on the deposition of filter powders of diatomaceous earth (Kieselguhr) on porous filter 'candles' for their filtering action. They cannot deal with highly turbid water and because of extremely high head losses in the filter their running costs are high.

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Chapter 10
Membrane processes

10.1. Introduction

This chapter gives an overview of membrane processes. It defines the various membrane processes, introduces some of the key concepts involved and discusses the reasons for the increasing use of membrane processes.

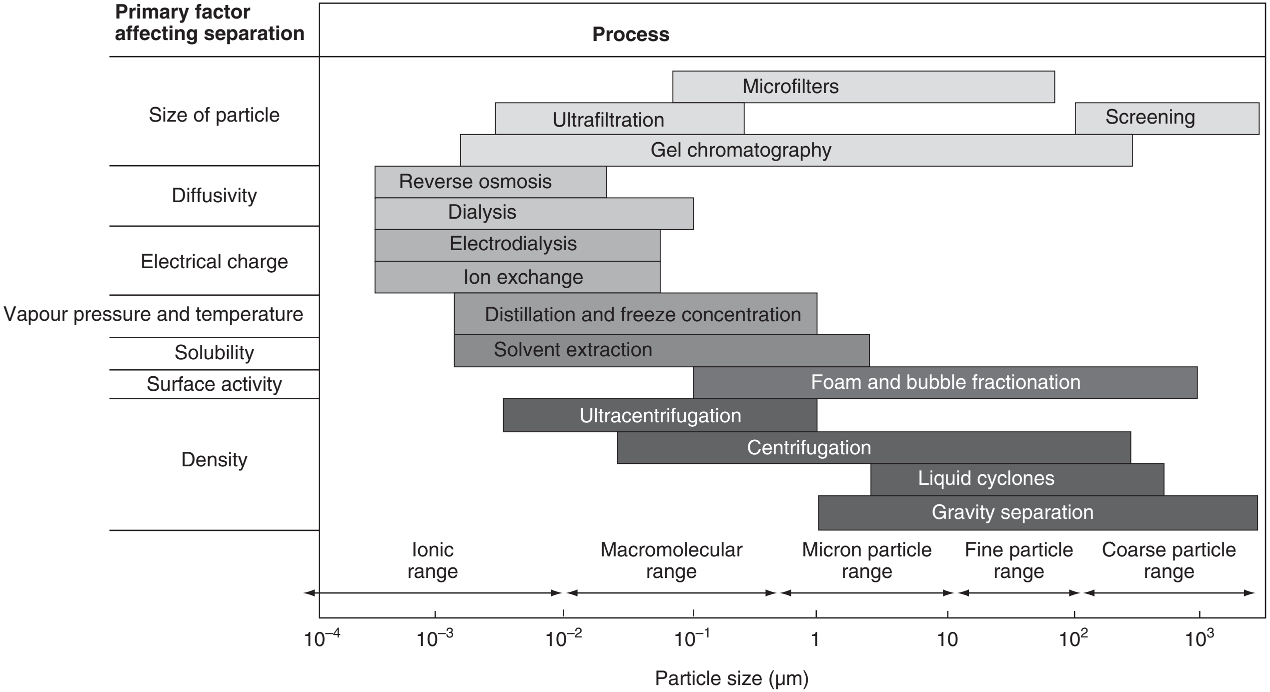
The term membrane process applies to processes that use membranes to remove either very small particles or molecules and ions from water. Figure 10.1 shows the range of particle sizes associated with membrane treatment and the membrane treatment processes used for different sizes of particles (Cheryan, 1986). It should be noted, however, that there are no hard and fast definitions of the particle sizes removed by the different processes and different definitions to those given below are common. It is also common to specify the approximate molecular weight of the molecules retained by the coarser membranes as this better reflects their performance.

Reverse osmosis (RO) is the process that removes essentially all particles and dissolved chemicals from water. However, a proportion of small dissolved undissociated molecules and dissolved gases do pass through the membranes. The mechanism of separation is a mix of physical straining and diffusion. Although it is arguably misleading, RO is often considered to remove particles below 1 nm (0.001 μm). Nanofiltration is RO but refers to the use of leakier membranes that typically allow through a high proportion of monovalent ions and a small proportion of bivalent ions. With the wide range of membranes available, RO membranes can be optimised to remove particular sizes of dissolved molecules and ions and the use of the term nanofiltration implies only that the smallest and most mobile ions will pass through.

Ultrafiltration is largely a straining process, essentially a very fine sieve. It removes suspended matter, colloidal material, and, where the pore sizes are at the lower end of the range, some large organic molecules. It is considered to remove particles in the range of 0.001 to around 0.1 μm.

Microfiltration is a pure straining process; removing particles in the size range 0.1–100 μm without affecting the soluble materials in the water being treated.

Figure 10.1 Particle sizes with applicable separation processes



Another membrane process is electrodialysis. The key difference of electrodialysis is that it is an electrical-based process that only removes ions and charged particles, although there may be some secondary straining.

10.2. Reverse osmosis

Perversely it is easier to define osmosis before defining RO. Osmosis is the term for the phenomenon whereby, if two salt solutions of different concentration are separated by a semipermeable membrane, water will migrate from the weaker solution through the membrane to the stronger solution, until the solutions are of the same concentration. The salt will be retained by the membrane and will not cross from one side to the other. Reverse osmosis involves applying a differential pressure to reverse this natural flow, forcing water to move from the more concentrated solution to the weaker.

The pressure required to do this has two components. The first is the pressure needed to prevent water moving to the more concentrated solution. This is the osmotic pressure and is a function of the difference in ionic strengths either side of the membrane. Assuming a temperature of 27°C and pure water on one side of the membrane this pressure varies linearly from approximately 0.4 bar for water with a sodium chloride concentration of 1000 mg/l to approximately 14.8 bar for water with a sodium chloride concentration of 35 000 mg/l (approximately equivalent to seawater). This pressure is the minimum pressure required to prevent osmosis, but to attain RO the pressure needs to be increased to force water into the stronger solution with an additional head loss. As would be expected the head loss through a membrane depends on the membrane used and the flow rate per unit of area through the membrane. For a given membrane material it is inversely proportional to the thickness of the membrane. In practice, it is typically of the order of 15–60 bar, with more saline waters normally requiring higher pressures. Combining the two components it can be seen that the flow of water through a membrane is given by:

$$Q_W = K_W \times (A/t) \times (\Delta P - \Delta\pi) \quad (10.1)$$

where Q_W = water-flow rate, K_W = membrane permeability coefficient for water, A = area of membrane, t = thickness of membrane, ΔP = pressure across membrane and $\Delta\pi$ = osmotic pressure.

Membranes used in RO are not completely efficient semi-permeable membranes; they also allow some diffusion of salt. This depends on diffusion and the flow rate of salt is given by:

$$Q_S = K_S \times (A/t) \times \Delta C_S \quad (10.2)$$

where Q_S = salt-flow rate, K_S = membrane permeability coefficient for the salt, A = area of membrane, t = thickness of membrane and ΔC_S = difference in salt concentration across the membrane.

The two important points that follow from Equations 10.1 and 10.2 are that

- increasing the differential pressure increases water flow across the membrane
- the rate of salt movement across the membrane is independent of pressure.

Thus, from the point of view of minimising treated-water salt concentration, it is better to operate at high differential pressures (which maximise water flow) and low differential salt concentrations (which minimise salt flow across the membrane). However, energy and pre-treatment costs also need to be considered, as will be discussed later.

10.2.1 Membranes

There are a variety of membrane materials and development of new improved materials is continuously happening. Initially, membranes were based on cellulose acetate. This was developed into a range of cellulose-based polymers. A notable advantage of these membranes is that most are tolerant of chlorine at low levels, meaning that bio-fouling can be easily avoided by chlorination of the feed water; this is most significant as the membranes can be biologically degraded. Other materials have subsequently been developed including polysulfone, polyurea and polyamide-based materials. There is now a wide range of polyamide materials and the properties of the material can be optimised for particular applications. Of particular note are thin film polyamide-based materials which being thinner, consequently involve lower head losses across the membrane and therefore operate at significantly lower pressures. The polyamide-based materials are resistant to biological degradation, but are chemically attacked by chlorine, although chloramine can often be used to control biological fouling without significantly affecting membrane life.

The two most common forms of membranes used are spiral wound modules and hollow fibre modules. In spiral wound modules, the material used comprises the membrane itself, integral with a more porous supporting material that provides the necessary strength to allow the module to be manufactured. Spiral wound modules are of a Swiss-roll construction, consisting of three layers

- a high-pressure layer which contains the feed water which is to be treated
- the membrane layer
- a low-pressure layer which contains the treated water, the permeate.

The high- and low-pressure layers contain spacers that permit flow to pass through them. Normally, the permeate flows to a collection tube in the centre of the module. Feed water is introduced at the outside of the module and the concentrated feed water, the concentrate, is collected at one end of the module.

Hollow fibre modules have the membrane in the form of a hollow fibre. Hollow fibres are manufactured by extruding the membrane material into very fine hollow fibres, with thousands of fibres formed into bundles. The fibres are sealed into pressure bulkheads, normally forming a 'U' tube within the module. The feed water and the concentrate are on the outside of the fibres and permeate is collected from where the fibres are sealed into the pressure bulkhead.

10.2.2 Pre-treatment

Given the very small pore sizes involved, it is vital that water is adequately pre-treated prior to the RO process. There are three potential problem areas: colloidal fouling; biological fouling; and scaling caused by chemical deposits. The nature of the problems to be addressed depends upon the source and characteristics of the water being treated. Water from sewage treatment plants contains high concentrations of organic material and bacteria (Franks *et al.*, 2007). Biological fouling is normally minimised by adequate pre-treatment and maintaining a disinfectant in the feed water. The disinfectant used is chlorine which depending upon the resistance of the membrane material may either be free or as chloramine.

Scaling caused by chemical deposits arises when the concentration of dissolved salts in the feed water exceeds the solubility of the salt, and it precipitates onto the membrane. Problems are usually associated with calcium carbonate, calcium sulfate, silica, and sometimes calcium phosphate, if phosphate is present in the feed water. Thus, hardness is an indicator of potential problems. Scaling is controlled by pH adjustment, the use of anti-scalants, and limiting the proportion of feed water that passes through the RO membranes.

Colloidal material needs to be received by pre-treatment. Pre-treatment involves producing a low-turbidity water using coagulation, settlement, dual media filtration, and either cartridge filtration or ultrafiltration to remove all particles greater than 5 μm or less. The use of ultrafiltration to remove particles of less than 0.1 μm is a more reliable way of pre-treatment than traditional cartridge filtration and is one of the factors that has led to a greater use of thin film composite membranes which operate at lower pressures (Jacalangelo and Trussell, 1997). After pre-treatment the water may then have its pH value lowered, and possibly be chemically dosed to prevent scaling. Depending on the membrane material it may need to be chlorinated. The treated water is then pumped using high-pressure pumps through the RO plant.

10.2.3 Membrane cleaning

No matter how effective the pre-treatment is, RO membranes require regular cleaning, with the cleaning regime dependant on the nature of the water being treated and the membrane itself. Cleaning involves taking modules out of service and flushing appropriate chemicals through the process to remove accumulated scale, colloidal material and growth. This might involve using acid cleaning first to remove mineral scale, followed by an alkaline cleaning to remove organic material. However, sometimes this is reversed. The cleaning may involve the use of detergent or a chelating agent to improve performance.

10.2.4 Post-treatment

The water produced by RO is usually acidic with no alkalinity and is very corrosive. If there has been acid dosing it will contain high levels of carbon dioxide. It has, therefore, to be treated to make it both palatable and safe, so that it does not attack piping and plumbing. It is necessary to add alkalinity and hardness and adjust the pH to ensure a stable water is produced. Typically, post-treatment involves passing the water through a bed of limestone or dosing lime and carbon dioxide. Where brackish water is being treated it may be that only a proportion of the water need be treated and the RO product can be blended with untreated water. However, some post-treatment will still be required.

10.2.5 Some practical aspects of RO treatment

With operating pressures up to around 70 bar it is clear that energy costs will be a very significant proportion of the operating costs of a RO plant. Older plants using seawater feed had an energy use of up to 10 kWh/m^3 of product water. Modern plants use of the order of 3 to 5 kWh/m^3, with the reduction associated with thin film membranes and energy recovery from the reject water. Power use is a function of the following factors.

- Temperature affects water viscosity and diffusion rates. Higher temperatures significantly reduce energy requirements.
- Operating pressure for an RO plant requiring feed water at 60 bar the energy input required to each cubic metre of feed water is approximately 1.6 kWh. Assuming a pump/electric motor efficiency of 80%, this requires electrical power of approximately 2 kWh/m^3 of feed water.
- Typically it takes around 2.5 m^3 of feed seawater to produce 1 m^3 of product water. Thus the energy required per cubic metre of product water would be 5 kWh before energy recovery. It can be seen that the proportion of product water compared to feed water is critical.
- At a power cost of £0.1/kWh this would be a power cost of £0.5/m^3 of product water, excluding any energy recovery. Energy recovery would typically reduce the power usage by around 40% for a seawater plant, less for a plant treating brackish water.
- In addition there will be energy used in pre-treatment, site facilities, and product water handling and delivery. Total energy use for treating seawater is unlikely to be less than 3.5 kWh/m^3, even for the most modern plants

Although these figures are typical of a seawater plant, they will vary dependent on the design of particular plants. The reject water, which in the example above represents 60% of the water pumped to the plant, will have a high specific energy on leaving the RO plant. It is therefore usual to install an energy recovery plant to recover as much of this energy as is economic.

A key parameter in the design of a RO plant is the conversion; this is the permeate flow as a percentage of the feed flow. Spiral wound modules only achieve around 10%, whereas hollow fibre modules achieve considerably higher rates. In the light of energy use as discussed above it is clearly absurd to work with a low conversion rate. Thus, it is normal to use the reject water from the first set of modules as the feed to a second set, and to use the reject flow from the second set as the feed to a third set. Thus, by employing a series of treatment stages, a higher conversion rate can be achieved. A limit on conversion is the acceptable salt concentration in the product water and the solubility of salts in the reject water. It can be seen from equation (10.2) above that the salt flow across the membrane is proportional to the difference in concentration across the membrane. The salt concentration in the feed water increases as the water moves down the series of modules, and the salt concentration in the product water similarly increases. Also as the concentration of salt increases in the feed water there is an increasing risk of precipitation of dissolved salts. The overall conversion rate for an RO plant typically varies between around 25% for older seawater RO plants and up to 80% or more for a plant treating brackish water.

Conversion rates impact not only power cost but also pre-treatment cost, which becomes significant for seawater plants with a low conversion rate.

The cost of water produced from RO plants is dependent on several variables including the following.

- Water source – seawater plants cost more to build and operate than plants using brackish water.
- The size of the plant affects both the capital cost per unit of capacity and the operational cost. The size of a RO plant is increased largely by adding more modules and thus above a certain size the capital cost is approximately proportional to capacity. However, the cost of ancillary works, such as the seawater intake is often significant and are less dependent on plant size than the RO plant itself. Similarly, some elements of operational costs such as power, membrane replacement costs, and chemical costs, are proportional to throughput or capacity, but labour costs will be proportionally higher for a small plant.
- Electricity costs are clearly a major component of the direct operating costs.
- Financing costs – the costs of financing the plant are typically of the order of £0.2/m^3 for a seawater plant with high utilisation. As utilisation drops the cost per

> cubic metre of water produced would increase. For brackish water the financing costs will be lower. Despite this a RO plant can be very effective at economically meeting peak demands if the costs of providing other water resources are high; for example, in tourist areas with dry summers which experience high peak demands in the tourist season.

Clearly, the major factors in the cost of desalinated water are the cost of power, which is the major direct operating cost, and the utilisation, affecting the financing cost. The cost of power varies depending on the fuel; power generated from natural gas is normally far cheaper than power generated from renewable sources. At a power cost of £0.1/kWh, the total cost of desalinated water from a high utilisation plant will be of the order of £0.6/m^3. Where a utility opts to power a plant using renewable energy the power cost will normally be significantly higher.

It is quite clear that the quantity of water produced by RO will increase rapidly over the first decades of the twenty-first century. The cost of water will drop as membranes improve, becoming cheaper and more efficient. However, seawater RO will always remain relatively expensive and energy intensive. The other main processes used for desalination, multi-stage flash (MSF) and multi-effect distillation (MED), are thermal processes which are widely used in the Middle East associated with thermal power stations. However, at the time of writing they appear unlikely to be significant outside of this region, with virtually all new capacity using RO (Anon, 2012).

10.2.6 Nanofiltration

Nanofiltration is a form of RO. It has developed as improvements in membrane materials have allowed membranes to be optimised for removal of larger molecules and ions. It operates at lower pressures, partly because the membranes let through most monovalent ions, leading to lower osmotic pressure, and mainly because the membranes themselves are more permeable, with a much lower head loss. Nanofiltration is not used for the production of drinking water from brackish water or seawater but is used to soften water and for nitrate removal. It can also be used to treat waters to remove organic matter that would react with chlorine to form trihalomethanes (THMs) and for colour removal (Irvine *et al*., 2000).

10.3. Microfiltration and ultrafiltration

Ultrafiltration is a filtration process using very fine pores such that as well as all suspended matter, large organic molecules are removed, with the cut-off point dependent on the characteristics of the membrane. The process also removes all micro-organisms save possibly for smaller viruses, which will only be removed by membranes with a small pore size. Microfiltration has a larger pore size than ultrafiltration.

Historically, most ultrafiltration membranes used hollow fibres. They normally comprise a thin skin of membrane on a supporting structure. They may have the membrane on either the outside of the fibre, in which case they will filter from the outside to the inside of the tube, or on the inside, in which case they will filter from inside the fibre to the outside. The membranes operate at far lower pressures than RO membranes, typically between 2 and 5 bar.

Ultrafiltration is often a cross-flow filtration process. In a conventional filter, flow is perpendicular to the filtering medium and all flow passes through the medium. In a cross-flow process, the main flow is parallel to the filtering medium, with only a proportion of the flow passing through the medium. Thus, rejected material tends to be carried with the reject water, being continuously removed from the filter module. This process is easier to visualise where the flow of feed water is in the centre of a hollow fibre. However, some ultrafiltration plants operate at 100% recovery rates, with all the feed water being recovered as permeate. Where this occurs, the cross-flow effect is lost at the end of the process and filtered material accumulates. The need for flushing and cleaning is greater where this is the case.

As the membranes operate at lower pressures a wider choice of material is available for the membrane, and the use of more resistant materials assists in cleaning. As well as flushing and backwashing, ultrafiltration membranes are cleaned using chemicals to dissolve material attached to the membrane.

In the UK, ultrafiltration is now commonly used to provide treatment of high-quality groundwaters for *Cryptosporidium* removal, often as the only treatment apart from disinfection. There are also indications that ultrafiltration is now being used to replace conventional clarification and filtration processes, notably in Guernsey where an existing conventional plant has been modified to a single stage of ultrafiltration, with a microstrainer at the intake (Redhead, 2008).

Microfiltration may use either hollow fibres or, where coarser filtration is required, some form of filtration mat.

10.3.1 Ceramic membranes

A relatively recent development in public water supply is the use of ceramic membranes for ultrafiltration. Ceramic membranes have been under development for some time, and have been used in Japan for public water supplies since the late 1990s. They are used more widely for industrial purposes and in particular in the oil industry, but they have been little used in potable water treatment to date outside of Japan. The great advantage of ceramic membranes is that they are far more robust than traditional polymeric membranes, with a far longer life expectancy. They can be cleaned using stronger cleaning agents or steam and can withstand high temperatures without being damaged. They are resistant to chlorine and other oxidants, offering more process flexibility. As the cost of ceramic membranes drops the additional capital cost will be offset by their long life and such membranes are likely to be increasingly common in potable water treatment. As of early 2013, outside of Japan, there are only a handful of municipal water treatment plants planned or using ceramic membranes, none of which are in the UK, but it seems certain that increasing numbers of such plants will be constructed.

The capital cost of ceramic membranes is higher than polymeric membranes but the operating costs are lower, and membrane life is far longer. An evaluation of a 9500 m^3/day plant carried out in 2010 for a small town in the USA, to replace a slow sand filtration plant that could not be operated at times of high raw water turbidity, had a capital cost for the ceramic plant 80% higher than a polymeric membrane, but an operating cost of 50%, giving a similar whole life cost (Hernandez, 2012, personal communication).

The membranes themselves comprise a support structure and the membrane layer, typically a metal oxide, nitride or carbide. Before its use in the UK in public water supply a membrane would have to be approved by the quality regulators, which in England and Wales is the Drinking Water Inspectorate.

10.4. The future

There seems no doubt that membrane processes will increasingly be used in the future. The reasons for the increasing popularity of these processes include

- an increasing need to provide potable water from saline or brackish water (using RO)
- their ability to soften water and remove nitrates (RO and nanofiltration)
- their use for single-stage treatment of soft upland waters containing colour, iron, manganese, and micro-organisms (nanofiltration and ultrafiltration)
- their ability to effectively remove protozoan oocysts (ultrafiltration).

In all instances, the membrane process is one of the options available and the decision as to which process to use will depend on local circumstances and, at least in part, on personal preferences.

However, RO is an energy-intensive process with correspondingly high carbon dioxide emissions, and this may become an important factor as carbon taxes become significant. This is being addressed by some suppliers by purchasing power for RO plants from renewable power suppliers, particularly wind farms, although this has cost implications as noted above.

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Chapter 11
Other processes

11.1. Introduction

This chapter considers briefly a number of processes, most of which have been developed to comply with the prescriptive water-quality standards that have come into force over the past 20 years. Several of the processes notably ozonation and granular activated carbon (GAC) adsorption are still being developed. One thing that the processes considered have in common is that it would be possible to write entire books on them, and this chapter only gives a broad overview of the processes and introduces some of the key concepts. These are the processes considered.

- Adsorption using activated carbon – initially used in the 1970s to treat for taste and odour but adopted on a large scale in the 1980s in particular for pesticide treatment.
- Ozonation – used for many years in continental Europe for disinfection but developed in the 1980s for treatment of organic chemicals, notably pesticides, and for disinfection in treatment works where chlorination would lead to failures of the trihalomethanes (THM) standards.
- Ion exchange – used for softening and nitrate removal, and also more recently for lowering of total organic carbon.
- Removal of iron and manganese.
- Nitrate removal – to meet the nitrate standard.
- Air stripping – to remove volatile organic chemicals.
- Water softening – the odd man out in that it has been used for many years and is now little used for new plants.
- Arsenic removal.
- Chemical dosing for lead treatment.

11.2. Activated carbon adsorption

Activated carbon is manufactured by heating carbonaceous material, normally coal, wood or coconut shells, under controlled conditions. Initially, the material is heated in the absence of air to drive off all volatile substances. It is then activated at high temperatures by further heating it to a high temperature and allowing it to react either with steam, air, and carbon dioxide or with phosphoric acid. This removes the volatile matter within the pores of the carbon and produces a material that is largely carbon with an open porous structure with a high surface area per unit weight.

11.2.1 Properties of activated carbon

There are two broad classes of activated carbon: gas adsorbing carbon, which has predominantly micropores which allow the entry of gases but permit only limited entry of liquids; and liquid-phase carbons which have a wide distribution of pore sizes and which allow liquid to penetrate the carbon.

Activated carbon comes in two main forms for water treatment: powdered (PAC), which is a finely ground material, and granular (GAC) which comes in a range of grain sizes, typically 0.5–1.5 mm. In water treatment, activated carbon is used to remove dissolved organic chemicals, either by dosing PAC to the water or by passing the water through a bed of GAC.

The properties of activated carbon are defined by several parameters. Table 11.1 lists some of the key properties of activated carbon and summarizes their significance (Benfield *et al*., 1982).

There is, however, no way to predict from the general data on carbons how an activated carbon will perform with a particular chemical or mix of chemicals without carrying out trials. These trials would normally cover several different types of carbon. However,

Table 11.1 Parameters used to characterise activated carbons

Property	Why important
Particle size	Based on standard sieve sizes The finer the material the higher the rate of adsorption In fixed beds finer material will have a higher head loss
Specific surface area: m^2/g	The higher the specific area the greater the adsorptive capacity of a given weight of carbon Sometimes referred to as the 'BET surface area' after the method used to calculate it
Pore volume: ml/g	The total volume of pores in the carbon
Apparent density: kg/m^3	The apparent (or bulk) density of the material
Specific gravity	The specific gravity of the material
Iodine number	A measure of the quantity of iodine adsorbed following a standard test. Relates to the volume of the material in the pore size range of 10–28 Å and indicative of the suitability of the carbon to adsorb low-molecular-weight organic compounds
Molasses number	Similar to iodine number but measures pores greater than 28 Å and indicative of ability to adsorb high-molecular-weight substances

manufacturers now have extensive data on the performance of their carbons and in practice they will often be able to advise on the treatment required. Factors affecting adsorption are given below and are discussed at some length by Benfield *et al.* (1982).

- Characteristics of the carbon – the particle size affects the rate of adsorption, but the capacity to adsorb a particular chemical is a function of area and pore size.
- Adsorbate solubility – in general the more soluble a chemical is the more difficult it is to remove, but there are exceptions.
- pH – which has a strong influence through its affect on ionization of chemicals. Organic acids are better adsorbed at a low pH, organic bases at high pH.
- Size of molecule – generally larger molecules are adsorbed better.
- Temperature – adsorption occurs at a higher rate at high temperatures but the capacity of a carbon is reduced as temperature increases.
- The concentration of the chemical to be removed – there will be an equilibrium established between the chemical in solution and the chemical adsorbed onto the carbon.

11.2.2 Adsorption isotherms

The equilibrium relationship between the amount of substance adsorbed and that remaining in solution is defined for a given set of conditions by an equation known as an adsorption isotherm. There are several forms of isotherm, depending on the theory used to model the adsorption. Two of the more common forms are the Langmuir and Freundlich isotherms. The Langmuir isotherm is based on an equation for adsorption:

$$x/m = abc/(1 + ac) \tag{11.1}$$

where x = weight of material adsorbed, m = weight of adsorbent, c = concentration of material remaining in solution once equilibrium has been reached, and a and b are constants.

Equation 11.1 can be rewritten as:

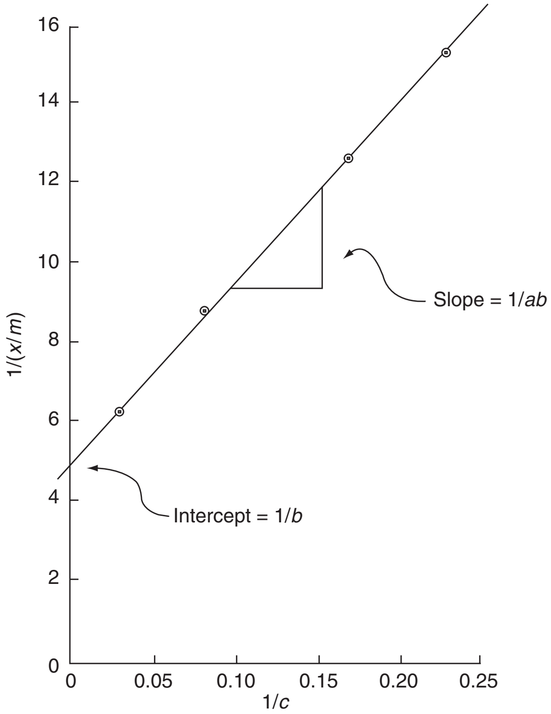
$$1/(x/m) = 1/b + 1/abc$$

Thus for a system which obeys the Langmuir theory, $1/(x/m)$ plotted against $1/c$ will give a straight line with a slope of $1/ab$ and an intercept of $1/b$ when $1/c$ is zero. The values to be plotted are derived by batch testing different concentrations of the chemical of interest. Figure 11.1 is a plot of a Langmuir equation.

The Freundlich isotherm assumes that the equation for adsorption is:

$$x/m = Kc^{1/n} \tag{11.2}$$

Figure 11.1 Langmuir isotherm



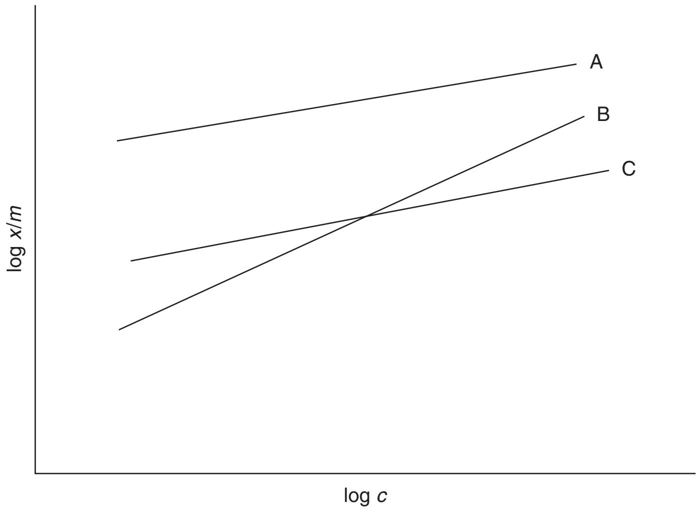
where K and n are constants. Plotting x/m against c on log graph paper will give a straight line with a slope of $1/n$ and an intercept of K where $c = 1$.

11.2.3 The uses of isotherms

Isotherms enable the maximum ability of a carbon to adsorb a chemical at a given equilibrium concentration to be established. This is used to calculate the theoretical quantity of substance that could be removed by a carbon while ensuring that the concentration in the treated water did not exceed a given value. However, in practice such plots cannot be directly used in water treatment for several reasons.

- There may be interference between the substance of concern and other substances present in the water which will compete for adsorption sites within the carbon.
- The isotherm represents the equilibrium situation. In water treatment, this may be valid where PAC is dosed but where a fixed bed is used due to the kinetics of the adsorption process it would not be practicable to have sufficiently long contact time for equilibrium to be attained.

Figure 11.2 Typical isotherm plots for different carbons



- The data are derived from batch tests but a continuous-flow system will react differently – initially all the substance will be removed, and then the concentration in the treated water will increase steadily with time until there is no removal.

However, isotherms do permit the comparison of different carbons. Consider Figure 11.2 which shows idealised isotherm plots for three carbons. It can be seen that carbon A has the highest adsorptive capacity at all concentrations over the range considered. This suggests it to be the best for removal of the substance considered. However, if the choice had to be between carbons B and C, the decision would depend on the concentration of interest. Carbon C would be preferred at lower concentrations, and carbon B at higher concentrations.

11.2.4 Design of activated carbon adsorption systems

These are the three possible systems that it would be possible to use in water treatment.

- Dosing of PAC followed by subsequent removal in a clarification stage.
- A continuous process passing water through a fixed bed.
- A batch process using contact vessels containing a fixed bed on a fill and draw basis.

In practice, the first two are used and the third is not.

PAC dosing is used where there is an intermittent problem of taste or odour or organic pollution, or as a temporary measure until a fixed-bed reactor can be installed.

Drawbacks of PAC dosing include that: PAC is a very messy substance to handle; dosing has to be higher than might be needed in order to cater for variations in concentration of the substances being removed; the PAC cannot be recovered for regeneration and re-use; and there is an increase in the quantity of waste solids produced. It is fairly common for plants to have a PAC dosing facility for intermittent seasonal or emergency use but there are very few plants that dose PAC continuously. Where there is a continuing need for adsorption GAC is either placed as a media in rapid gravity or slow sand filters, or else is used in purpose-built contactors.

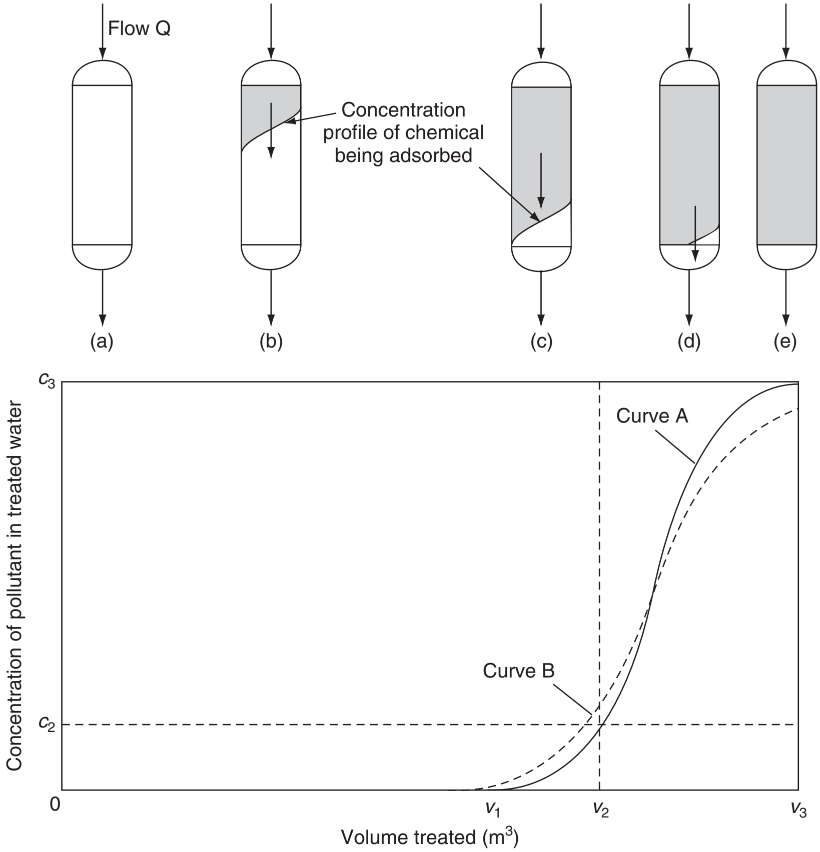
A key design parameter in GAC absorbers is the empty-bed contact time (EBCT). This is equal to the volume occupied by the GAC bed divided by the flow rate through the bed; it is normally expressed in minutes. For taste and odour removal, an EBCT of around 5 min will normally be effective but for removal of pesticides and other organic chemicals a time of 15 to 20 min is normally required.

A rapid gravity filter typically has a maximum hydraulic loading of at least 6 m^3/m^2 h. Thus for an EBCT of 15 min a bed depth of 1.5 m is required. This is impracticably high and generally rapid gravity filters using GAC media are normally limited to use for taste and odour removal, with dedicated contactors if removal of other chemicals is required. A GAC layer 200 mm thick in a slow sand filter with a loading rate of 0.6 m^3/m^2 h would have a contact time of 20 min, which is high enough for most purposes.

Figure 11.3 illustrates the theoretical performance of fixed-bed reactors in removing substances by adsorption. Figure 11.3 (a) shows the reactor filled with new adsorbent, with a flow, Q, passing down through the reactor. The substance being treated is removed in the adsorption zone. The size of this zone depends on the velocity of the water being treated through the reactor and the rate of removal: at a low velocity or a high rate of removal this zone will be shallow. Figures 11.3 (b)–11.3 (d) show the concentration profile of the water as it passes through the absorber. Figure 11.3 (c) shows a volume of V_1 having passed through the bed and the contaminant having just reached the outlet to the absorber with breakthrough beginning. If $C2$ is the maximum allowable concentration of the contaminant in the treated water, then V_2 represents the maximum volume of water that can be treated before the carbon requires to be replaced. Figures 11.3 (d) and (e) show the GAC becoming exhausted; 11.3 (e) shows the bed saturated and the treated water concentration of the pollutant the same as in the untreated water.

An important point from Figure 11.3 is that it is assumed that the fixed bed is in fact fixed, with the pollutant concentration given by curve A. In reality, it is necessary to regularly backwash rapid gravity filters and it is normal to clean most contactors periodically by backwashing them using water or air and water. This, however, can give problems if it then results in the GAC within the filter becoming mixed. Mixing of the media will move

Figure 11.3 Showing the breakthrough of pollutant in an adsorber column



some GAC from the top of the bed to a lower position. If the carbon that is moved was in equilibrium with the concentration in the feed water of the substance being removed, moving it to a position where the concentration is lower due to adsorption higher in the bed will result in release of the adsorbed chemical. This will lead to the concentration of contaminant following a curve similar to curve B, with the shape dependent on the degree of mixing. The quantity of contaminant retained within the GAC when fully exhausted will be the same in both cases, but in the mixed case the GAC would normally require to be replaced sooner. This problem can be overcome by ensuring that backwashing of filters/absorbers is carefully controlled to ensure full fluidisation of the bed. This ensures that, after backwashing, the larger particles remain at the bottom of the bed and the lighter particles remain near the top of the filter. An alternative approach would be to provide two or more absorbers in series.

11.2.5 Practical aspects

Where adsorption is required in a plant using rapid gravity filters, it is normal now to provide GAC absorbers as a dedicated adsorption treatment stage. Where this is affordable and practical, separate adsorption is the best solution from a process point of view as it allows the adsorption process to be optimised, without the constraints imposed by the need for solids removal. Indeed, some works which had rapid gravity filters modified to use GAC media have subsequently been converted back to sand filters, with separate GAC absorbers constructed. Absorbers are normally large closed vessels. They require facilities for periodic cleaning to remove solids trapped within the bed.

Chemically, GAC is a strong reducing agent; it therefore reacts with chlorine and other disinfectants. As a consequence there can be problems with controlling biological growth within the absorbers. Where the GAC is present in rapid gravity filters, regular backwashing tends to limit problems. Where the GAC is present in dedicated absorbers it may be necessary to backwash to remove biological matter growing within the filter. An alternative approach is to accept the biological growth within the absorber and provide effective disinfection afterwards, however there can be problems associated with poor disinfection of lumps of biological material shed from the absorbers. There are some advantages to having biological growth within GAC absorbers in that the organisms may use some of the adsorbed chemicals as a substrate, prolonging the life of the GAC. This is referred to as biological activated carbon.

Where slow sand filters are used the GAC can be placed in a layer in the lower part of the sand bed. This has proved a very cost-effective way of upgrading sand filtration plants, avoiding the high capital cost of providing a dedicated adsorption stage.

Once the GAC has reached the end of its effective life it is removed and replaced with new or regenerated carbon. Used carbon can be regenerated, with some loss of material, and re-used. Provision is made in adsorbers for placing new GAC and removing used material using an automatic system, normally as a slurry.

11.3. Ozonation

Ozone is an unstable form of oxygen with the chemical formula O_3. In solution, it decays to O_2 in the process producing free hydroxyl radicals. Ozone and hydroxyl radicals are the two most powerful oxidants used in water treatment. Ozone is therefore a strong disinfectant; it is also used for oxidising many of the synthetic organic chemicals found in water from lowland rivers and the natural organic colour found in soft waters from upland rivers. Ozone is a colourless gas with a characteristic pungent smell often associated with electrical sparking. It is sparingly soluble in water; at 20°C the solubility at 1 bar partial pressure is 570 mg/l.

Ozone is believed to react in water as an oxidant both by direct oxidation of chemicals by molecular ozone and by indirect oxidation by hydroxyl free radicals produced during decomposition of ozone (USEPA, 1999). Oxidation by hydroxyl free radicals is the stronger process. At low pH values, direct oxidation by ozone is the more important process. At high pH values, or where the formation of hydroxyl free radicals is encouraged by UV light or by dosing hydrogen peroxide, hydroxyl oxidation may predominate.

Up until the early 1990s, ozone was little used in the UK although it was widely employed in France and some other European countries. However, the European standard of 0.1 μg/l for pesticides introduced in the 1980 Drinking Water Directive, and the widespread use of pesticides and their presence in most lowland rivers, meant that an effective and economic method of removal was required. The method generally adopted was ozonation, to oxidise many of the pesticides, combined with GAC adsorption to remove those pesticides resistant to ozonation and also the by-products of ozonation.

Ozone is manufactured by passing air or oxygen through a high-voltage alternating current discharge; this is a carefully controlled corona rather than an uncontrolled spark. After passing through the electrical discharge a proportion of the oxygen is converted to ozone. The electrodes can either be plates or, more commonly, concentric tubes.

The efficiency of ozone production compared to power input is a function of several factors in particular the oxygen content of the gas and its ozone concentration after passing through the ozone generator. Systems using pure oxygen are most efficient and systems using air are least efficient; the higher the ozone content of the gas after the ozone generator the lower the efficiency of power use. Other factors include temperature, the voltage and frequency used, the pressure at which the system operates, and the gap between the electrodes. Ozone production decreases at higher temperatures, and as there is a large heat production due to the electrical discharge, efficient cooling is essential.

Energy requirements for ozone generation, excluding any power used in air preparation, are in the range of 10–25 kWh/kg O_3 with the lower figure relating to low concentrations of ozone in a pure oxygen feed and the higher to high concentrations of ozone in an air feed.

It is essential that the feed supply to ozone generators is free of oil and dust and has very low moisture content. The presence of oil can lead to explosions and water vapour leads to uncontrolled sparking and the production of nitric acid. Preparation of the feed gas, commonly referred to as ‘air preparation’, is as important as the actual production of ozone in the generator, and requires significantly more space.

Older systems used either air or liquid oxygen and there was little to choose between them in terms of cost, with the higher efficiency of the liquid oxygen systems balanced by the higher cost of the liquid oxygen required. Currently, the most common systems use an oxygen-enriched feed produced by treating air using pressure swing absorbers (PSA). These preferentially remove nitrogen from air, producing a gas with an oxygen content of up to 80–95% oxygen, compared to ~20% in the atmosphere. Energy requirements for air preparation are in the range of 10–15 kWh/kg O_3. The overall power requirement for an ozone system using air as the oxygen source is of the order of 20–40 kWh/kg O_3.

After preparation, the ozone-rich gas is then dissolved in the water. This is normally done in the UK using contact tanks where ozone is introduced via diffusers in the bottom of the tanks. Alternatives are to use a venturi eductor system or to introduce ozone into a turbine or static mixer. The dissolved ozone reacts with the various inorganic and organic chemicals in the water with any residual breaking down into oxygen. The following factors are considered in the design of ozone contact tanks.

- Ozone concentration – the equilibrium ozone concentration in water with all other things being equal will vary in proportion to the ozone concentration in the feed gas.
- Bubble size – smaller bubbles have a larger surface area per unit volume of gas and thus speed up the transfer of ozone to water.
- Pressure – the gas transfer rate is dependent on pressure, thus deeper tanks are more efficient.
- The ozone demand of the water – where reduced iron or manganese is present in the water ozone is removed at a high rate by the oxidation of these substances, leading to higher transfer rates than where reactions are slow.
- pH – at elevated pH values ozone rapidly decays.

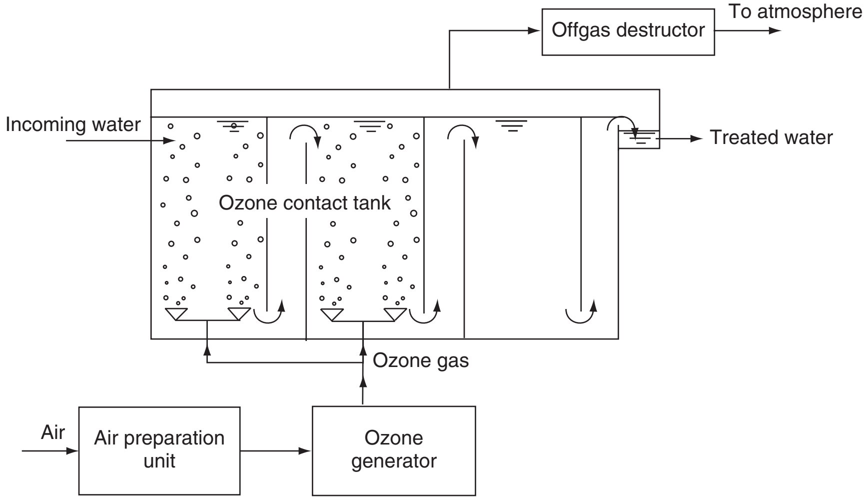
A well-designed and operated system will transfer around 90% of the ozone to the water. The balance is vented to the atmosphere by way of an ozone thermal destruction unit. Figure 11.4 is a schematic diagram of an ozone system.

In potable-water treatment ozonation has several important uses

- oxidation of organic micropollutants, notably pesticides prior to GAC adsorption
- removal of taste and odour by oxidation of the organic matter causing the problem
- as a disinfectant
- oxidation of soluble iron and manganese.

A potential problem with ozonation is the production of bromate from water containing bromide. The 1998 Drinking Water Directive sets a bromate standard of 10 μg/l. Bromate formation can be minimised by operating at a low pH.

Figure 11.4 Diagram of ozonation system



In practice, the two most common locations for ozonation in water treatment are as a pre-treatment, prior to clarification, and as a polishing treatment after the main filtration process. The former is loosely referred to as pre-ozonation, and the latter as post-ozonation. For larger plants treating water from lowland rivers it is common for both processes to be used. Pre-ozonation aims to pre-treat water: it oxidises organic pollutants, and any iron and manganese present kills algae, and has a beneficial effect on subsequent coagulation. It also disinfects the water. Post-ozonation oxidises any remaining organic pollutants and disinfects the water.

When used for disinfection, ozone is extremely effective against pathogenic bacteria and viruses. The effectiveness of ozone is temperature dependent, but a *Ct* of 1.6 (mg min/l) will essentially kill all viruses and pathogenic bacteria. This would typically be achieved by ensuring an ozone concentration of 0.4 mg/l for a contact time of 4 min. In practice, where ozone is used for disinfection, a typical arrangement would be to have two or three cells in series, each of around 4 min detention time. Sufficient ozone would be dosed in the first cell to give the required residual, and this would be maintained in the latter two cells. The dose required will depend on the nature and quality of the raw water. The need to limit bromate formation may impose limitations on the acceptable dose of ozone and on pH value.

When used for treatment of organic contaminants, the required *Ct* may be higher than that required for disinfection. While the actual design criteria vary, treatment of lowland rivers have the following typical criteria.

- Pre-ozonation – ozone dose of around 2 mg/l with the capacity to dose up to 5 mg/l and a contact time of 10 min.
- Post-ozonation – ozone dose of around 1 mg/l with the capacity to dose up to 3 mg/l and a contact time of 10 min.

The effectiveness of ozone to oxidise organic chemicals can be enhanced by combining ozonation with dosing hydrogen peroxide or the use of UV light. As noted earlier, this leads to increased formation of hydroxyl free radicals. In practice, it has generally not been found necessary to use these enhanced forms of ozonation but they may well be required in the future to meet future standards for chemicals resistant to simple ozonation. Enhanced ozonation will permit lower ozone doses and may reduce the formation of bromate. However, it does lead to control problems due to difficulties of measuring ozone concentration when free radicals are present.

11.4. Ion exchange

Ion exchange involves passing water through a bed of ion-exchange resin, changing the composition of the water by removing unwanted ions from water and replacing them with other ions. Traditionally, this has been used to remove inorganic ions and involved passing water through a bed of insoluble material made of a synthetic resin; when the bed was exhausted it was regenerated. However, recently continuous processes have been developed which have expanded the range or water quality problems addressed by ion-exchange, and these recent developments are considered at the end of this section.

The resins used are designed such that they will remove either cations or anions from the water passing through the bed, replacing the ions removed from the water with ions of the same charge from within the resin. The main uses of ion exchange in potable-water treatment are to remove nitrate or to soften water. This section will cover four aspects of ion exchange.

- Classification of resin used.
- The factors affecting ion removal.
- Uses of ion exchange in water treatment.
- Modern developments.

11.4.1 Classification of resins used

The basic classification of resins is into cation-exchange resins (which contain exchangeable cations) and anion-exchange resins (which contain exchangeable anions). These can then be sub-divided further depending on their affinity for cations or anions. In water treatment, the resins of most interest are strong-acid exchange resins, which are the cation-exchange resins used in softening, and selective strong-base resins, which are the anion-exchange resins used in nitrate removal. In water treatment both of these are regenerated with sodium chloride solution.

11.4.1.1 Regeneration

In softening, sodium ions within the resin are released into the water and replaced by cations from the water being treated. Regeneration involves introducing sodium ions into the resin, displacing the ions removed from the water. In nitrate removal, chloride ions within the resin are released into the water and replaced by anions (predominantly nitrate) from the water being treated. Regeneration involves introducing chloride ions into the resin, displacing the ions removed from the water. Thus, in both cases, the regenerant used is a concentrated solution of sodium chloride. Regeneration involves introducing the regenerant solution into the bed and ensuring a contact time adequate for the required degree of regeneration. After regeneration rinsing is required. Regeneration is considered further below.

11.4.2 Factors affecting ion removal

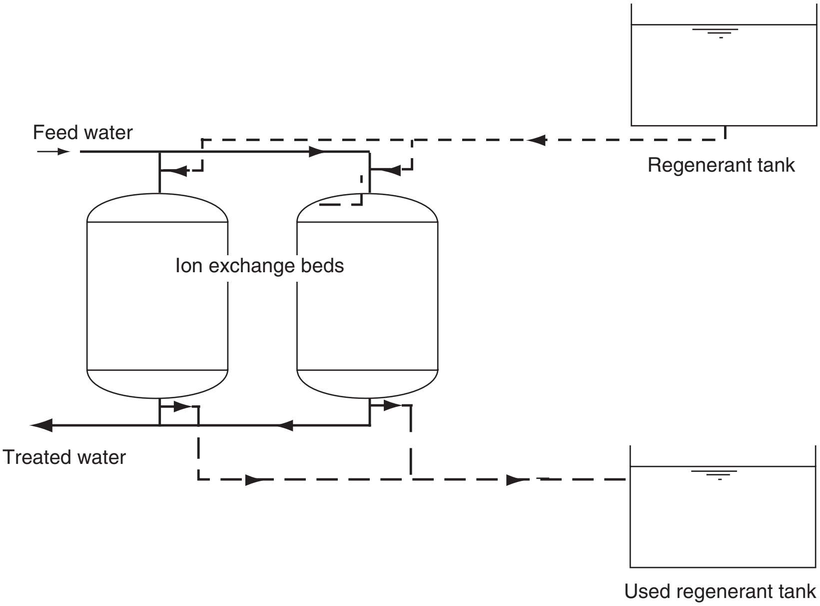
A resin does not remove all anions or cations equally. Rather it will preferentially remove some ions as opposed to others, and, as a resin becomes exhausted, there may be release of ions already removed and their replacement by other preferred ions. In weak solutions, the main factor is valency, with ions of higher valency being preferred. However, other factors also apply, notably the size of the ion. Size is important because ion exchange is not simply a surface phenomenon, but involves diffusion into the resin itself. Thus, it is possible to optimise resins for the removal of specific anions or cations. Important physical factors include contact time, temperature, and the cleanliness and stability of the water being treated. As the ion-exchange resin is in a bed it is important that there is little or no deposition of material within the bed.

A unit weight of a particular resin will have a fixed capacity for a particular ion. This is normally expressed either as the total capacity or the operating capacity. The total capacity represents the theoretical capacity under equilibrium conditions of a resin to remove a specific ion. In practice, this is of little interest and the operating capacity is far more important. This represents the capacity of the resin to remove a specific ion under the actual conditions under which the resin will be operating. Additional factors affecting this are: the ionic composition of the water being treated, as there will be competition between the ion of interest and other ions in the water; the acceptable concentration of the ion of interest in the treated water, which will define the point at which the resin will need to be regenerated; and the effectiveness of regeneration, which may not be 100% effective.

11.4.3 Uses of ion exchange in potable-water treatment

As noted above, the main uses of ion exchange in water treatment are for softening and for nitrate removal. Softening involves removing bivalent metal cations, and nitrate removal involves removing nitrate anions. There is an important difference between the two processes. In softening it is desired to remove bivalent metal ions. Such ions will typically comprise over 80% of the cations in water. In such a water, cation

Figure 11.5 Elements of an ion exchange system



replacement would largely be removing the ions it was required to remove. (Actually, sodium need not be considered as no sodium is removed.) Thus, ion exchange is very efficient for softening.

However, nitrate removal is different. The proportion of nitrate anions present is likely to be less than 50% of the bivalent anions. Thus, a non-selective resin would remove mainly ions that were not nitrate, and this would represent inefficient treatment. This has led to the development of resins that remove nitrate preferentially, greatly improving efficiency.

The key components of a fixed bed ion-exchange system are listed below and shown schematically in Figure 11.5.

- The ion-exchange beds. These are normally pressure vessels, similar to pressure filters, containing the resin. There will normally be several vessels to allow treatment to proceed uninterrupted while one bed is being regenerated.
- A regenerant holding tank which contains the strong salt solution used for regenerating the resin.
- A used regenerant holding tank, containing used regenerant and rinse water prior to disposal off-site or to a sewer.
- The appropriate valving and transfer pumps.

Ion-exchange plants are often referred to as co-current or counter-current. This refers to the directions of normal operation and regeneration. A plant where water flows downwards during normal operation is a counter-current plant if regeneration is carried out with an upward flow of regenerant; if regeneration involved downward flow the plant would be a co-current plant.

The significance of this is that regeneration will be most complete and effective in those parts of the bed that are treated with the freshest regenerant solution. Thus, in a counter-current system in which normal flow is downwards and regeneration is upwards, the part of the bed that will be most effectively regenerated is the bottom of the bed. This means that there should be no breakthrough of the ion being removed until the bed is fully exhausted. If regeneration were co-current, the bottom of the bed would be the area least effectively regenerated. There will then tend to be some leakage of ions from this area of the bed during normal operation. In practice this is of little significance in potable-water treatment, although it is important in demineralisation plants where counter-current regeneration is preferable.

11.4.4 Modern developments of the ion exchange process

Over recent years, there have been significant developments in ion-exchange technology aimed at widening the use of the process. Continuous processes have been developed which by continuously removing and regenerating a small proportion of the resin avoid the need for periodic regeneration required for traditional fixed bed reactors. The use of such a process to remove organic carbon as pre-treatment to membrane processes appears to be of particular interest. The processes only work with that portion of dissolved organic carbon that has a suitable ionic charge. One process which has been used to a limited extent in the UK and more widely in the USA is the MIEX® process. The process has been used both as pre-treatment to membrane treatment, and to reduce the formation of trihalomethanes by removal of the precursor chemicals. These are relatively new processes and as such they may well be further developed, and areas of application may expand.

11.5. Water softening

11.5.1 Introduction

Water chemistry is a very complex and specialised subject. This short section touches on some aspects of water softening and water stabilisation, and is no more than a very simplified introduction to water softening. For more detail, reference should be made to an appropriate specialised textbook. In this section, only calcium and magnesium are considered which is all that is required for most waters.

Hard water is water that requires the use of a large amount of soap to produce lather. Soap is normally the sodium salt of a fatty acid. When soap is added to water containing polyvalent metal ions, the salt formed by the polyvalent metal ions and the fatty acid precipitates, forming a scum. Only when all the polyvalent metal ions have precipitated will a lather form. Thus, hardness is caused by dissolved polyvalent metal ions. In practice, it is only bivalent metal ions that occur to a significant extent in normal water, notably calcium and magnesium. Waters that are high in bivalent ions can also cause scaling problems in hot-water systems.

Hardness is defined by the total concentration of bivalent metal ions present – but expressed as mg $CaCO_3$/l. This is the equivalent calcium carbonate concentration that would have the same effect as the bivalent metal ions actually present. Calcium carbonate has an equivalent weight of 50, and thus the equivalent hardness due to a concentration M of a bivalent metal ion is:

$$\text{Hardness as } CaCO_3 = M \text{ (in mg/l)} \times 50/\text{Equivalent Wt of } M$$

The total hardness is calculated by summing the results of the above calculation for all bivalent metal ions present. It can also be measured directly by titration.

Hardness is conveniently classified into carbonate and non-carbonate hardness. Where the metal ions are associated with bicarbonate, boiling will result in the bicarbonate being converted to carbonate and precipitating as calcium or magnesium carbonate. (It should be noted that magnesium carbonate is soluble in cold water but is less soluble in boiling water and thus is largely removed by boiling.) Where the metal ions are associated with chloride, sulfate, or other anions, with no bicarbonate present, boiling will not precipitate the bivalent metal. For this reason, carbonate hardness is also referred to as temporary hardness, and non-carbonate hardness as permanent hardness. The most common cause of carbonate hardness is water passing through chalk or limestone aquifers, with non-carbonate hardness often associated with water passing through clay soils containing sulfates. There are no hard and fast standards for hardness but water with more than 200 mg $CaCO_3$/l is normally considered to be hard.

Water softening involves the removal of bivalent metal ions. It is popular with consumers because it reduces scum formation and scaling of kettles, and it is needed for hot-water heaters and boilers to reduce or eliminate scale formation. For industrial-scale boilers and for power stations it is essential to soften the water used. There are two main ways to soften water: removal of the bivalent ions by precipitation; or replacement of the bivalent ions by sodium ions using an ion-exchange process. Other less common options are electro-dialysis reversal (EDR) or reverse osmosis optimised for softening. Chemical precipitation is the process normally used in potable-water treatment. The only circumstances where ion exchange or other process might be used are where water contains a high proportion of non-carbonate hardness, or where the hardness continually varies. In industrial water treatment ion exchange is more common on boiler feed waters.

11.5.2 Chemical precipitation of hardness

If hydroxide, as either lime or sodium hydroxide, is added to water containing carbonate hardness, the bivalent metals will precipitate as either calcium carbonate or magnesium hydroxide. Where non-carbonate hardness is present, adding lime and carbonate to the water also results in precipitation of calcium carbonate or magnesium hydroxide. It is normal to use calcium hydroxide (lime) as the source of hydroxide, and sodium carbonate (soda or soda ash) as the source of carbonate. Thus, the process used to remove both carbonate and non-carbonate hardness is commonly referred to as the lime–soda method. The proportions of lime and soda required depend on the composition of the hardness.

11.5.3 Lime–soda method

11.5.3.1 Removal of carbonate hardness

The first part of the lime–soda method considered is the addition of lime to precipitate carbonate hardness. Carbonate hardness is associated with bicarbonate and calcium and magnesium ions present in water.

The initial reaction on adding lime is a reaction between dissolved carbon dioxide and the lime. Assuming that the carbon dioxide is dissolved as carbonic acid, this is represented as:

$$Ca(OH)_2 + H_2CO_3 \rightarrow CaCO_3 + 2H_2O \qquad (11.3)$$

This reaction does not affect the hardness but does represent a lime demand that needs to be satisfied before there is any softening. Adding additional lime results in the precipitation of calcium and magnesium. Calcium is precipitated in a single-stage reaction:

$$Ca(OH)_2 + Ca^{2+} + 2H_2CO_3^- \rightarrow CaCO_3 + 2H_2O \qquad (11.4)$$

Thus, one mole of lime removes one mole of calcium carbonate. Magnesium is precipitated in a two-stage reaction:

$$Ca(OH)_2 + Mg^{2+} + 2H_2CO_3^- \rightarrow CaCO_3 + MgCO_3 + 2H_2O \qquad (11.5)$$

$$Ca(OH)_2 + MgCO_3 \rightarrow CaCO_3 + Mg(OH)_2 \qquad (11.6)$$

In the first stage, the bicarbonate is essentially neutralised by the lime, with calcium carbonate precipitating. Magnesium carbonate is soluble in cold water (but is less soluble in hot water and, thus, is largely removed by boiling). In the second stage, addition of additional lime results in the formation of insoluble magnesium hydroxide. Thus, two moles of lime are required to remove one mole of magnesium carbonate.

11.5.3.2 Removal of non-carbonate hardness

The second part of the lime–soda method considered is the addition of lime and sodium carbonate to precipitate non-carbonate hardness.

Calcium non-carbonate hardness is removed by adding sodium carbonate:

$$\mathrm{Ca^{2+} + 2Cl^{-} + 2Na^{+} + CO_3{}^{2-} \rightarrow CaCO_3 + 2Na^{+} + 2Cl^{-}} \tag{11.7}$$

The calcium is precipitated as calcium carbonate, with one mole of sodium carbonate removing one mole of calcium non-carbonate hardness.

Magnesium non-carbonate hardness is removed by adding sodium carbonate and lime:

$$\mathrm{Mg^{2+} + 2C1^{-} + 2Na^{+} + CO_3{}^{2-} + Ca(OH)_2 \rightarrow CaCO_3 + 2Na^{+} + 2Cl^{-} + Mg(OH)_2} \tag{11.8}$$

The magnesium is precipitated as a hydroxide: the hydroxide is supplied by lime, and the sodium carbonate is required to remove the calcium added as the lime. Thus, one mole of lime and one mole of sodium carbonate are required to remove one mole of magnesium non-carbonate hardness.

11.5.3.3 Other points to note

It is not essential to use the lime–soda method for precipitation of hardness. Sodium hydroxide can be used to precipitate carbonate hardness and magnesium non-carbonate hardness. It will also precipitate some or all of the calcium non-carbonate hardness. Depending on the chemical composition of the water, it may be necessary to add some sodium carbonate as well.

For potable-water treatment, it is normally acceptable to design to soften by calcium removal alone. This avoids the dosing of sodium carbonate and means that only calcium carbonate is precipitated. This precipitates more easily than magnesium hydroxide and is easier to dewater. For industrial water treatment, magnesium hardness will normally also have to be removed.

Chemical precipitation does not completely remove all calcium and magnesium hardness as both calcium carbonate and magnesium hydroxide are not wholly insoluble. Water softened by chemical precipitation is unlikely to have a hardness significantly below 50 mg $\mathrm{CaCO_3}$/l.

In practice, where a potable water is to be softened the flow will be divided into two, with one stream being softened and the other bypassing the softening process; with the flows combined downstream of softening. The proportions of softened and unsoftened flows depend on the hardness of the raw water and the desired degree of softening required.

Dosing rates are usually higher than those required in order to speed up the reaction rate. Where part of the flow is softened this can lead to further precipitation of hardness after the softened flow is recombined with unsoftened flow, and additional chemical dosing may be required to control this.

An example of a calculation of chemical use in softening is given in Appendix 1.

11.5.4 Application of processes

Traditionally, softening was carried out in large hopper-bottomed clarifiers designed by companies specializing in the design and construction of water-softening plants. Where chalk or limestone borehole waters were being softened the only purpose of the clarifiers was for the removal of precipitated hardness. Such clarifiers removing calcium hardness were operated at a loading rate of around 4 m/h. The sludge produced from softeners consists of fine calcium carbonate and can be difficult to thicken, dewater and dispose of.

Where clarification is required to remove suspended solids it can be combined with softening. However, softening generates a high pH value, of around 10.5, and this places constraints on the coagulant that can be used. It is normal to use a ferric coagulant that works well at high pH.

Nowadays, where softening is required, pellet reactors are normally used. These consist of vertical cylindrical cylinders with a conical bottom. The water to be softened and lime are injected into the bottom of the conical section. The reactor contains a bed of single-size sand. The flow of water liquidises the bed. As the softening reactions take place, the calcium carbonate and magnesium hydroxide precipitate onto the grains of sand, which act as seeds. The sand particles become coated with precipitated material and grow in size. Once the reactor has stabilised it is necessary to regularly remove the heavier particles and replace them with sand.

The reactors operate at high-loading rates, above 50 m/h. The great advantages of using pellet reactors are the high-loading rates and the nature of the waste material; this consists of hard pellets of around 2 mm in diameter. These require no dewatering and are easy to handle and dispose of. Pellet reactors work excellently when the hardness is predominantly calcium hardness but are not to be recommended where magnesium is present to any extent (>25 mg/l) because magnesium does not react and precipitate as quickly as calcium to form pellets. The design of these reactors is a specialised task and is normally done by the manufacturer/supplier of the plant.

11.5.5 Post-softening treatment

The water leaving a softening plant will be supersaturated with calcium carbonate and will have a high pH. It will, therefore, tend to continue to precipitate calcium carbonate unless the pH value is lowered. The easiest way of doing this is to dose a strong acid, such as sulfuric or hydrochloric acid. This, however, may produce a corrosive water and it is often better to lower the pH value by injecting carbon dioxide, a process often referred to as re-carbonation. An alternative is to dose a proprietary chemical such as Calgon.

11.5.6 Softening by ion exchange

Ion-exchange treatment has been discussed above. Where ion exchange is used in softening the resin is a cation-exchange resin. The water to be softened is passed through the bed of resin. Sodium is released from the resin and replaced by calcium, magnesium, and other metals present in the water. Thus, the treated water will contain no bivalent ions and will have effectively zero hardness. When the resin is exhausted it is regenerated by a strong solution of sodium chloride.

The water produced by ion-exchange softening will have a high sodium content. However, for potable treatment, only a proportion of the water would be softened, and the softened water would then be mixed with the unsoftened water, lowering sodium levels.

The wastewater produced by ion-exchange softening plants is a strong saline solution, also containing the metals removed in the softening process. This can normally only be disposed of to a sewer, where it is diluted with sewage to lower salinity to an acceptable value. Ion exchange has been used in the UK for potable-water softening but it seems unlikely that many new plants will be constructed in the future because of problems of waste disposal.

11.6. Removal of iron and manganese

Both manganese and iron can exist in different oxidation states. Iron exists as either +2 or +3. Within the normal pH range of interest in water treatment, Fe^{2+} is soluble and Fe^{3+} is insoluble. Manganese exists in a range of oxidation states but effectively two states are of interest, Mn^{2+} is soluble, and Mn^{4+} is insoluble.

Well-aerated surface waters from rivers or reservoirs will normally only contain low levels of iron and manganese. This is because the iron and manganese will be oxidised to insoluble forms and will tend to settle out of the water. Where they are present they are easily removed during conventional treatment.

Groundwater from confined aquifers and water from the lower anoxic zones of reservoirs often contains the reduced soluble forms of iron and manganese. Water from reservoirs may also contain iron or manganese complexed with organic material. This section considers some aspects of the removal of dissolved iron and manganese. Benfield *et al.* (1982) and Morris and Siviter (2001), cover in more detail the theoretical and practical aspects of iron and manganese removal.

Where inorganic iron and manganese are present, the treatment required is oxidation of the iron and manganese, which will cause them to precipitate, followed by removal of the precipitated material. Oxidising agents commonly used are oxygen and chlorine; other possible oxidants are ozone, chlorine dioxide and potassium permanganate.

Iron is relatively easy to oxidise. The requirements are a pH of 7.0 or above and the presence of dissolved oxygen. The process proceeds rapidly but can be further hastened by chlorination. The rate of oxidation is affected by both temperature and pH, proceeding faster at high pH values and temperatures. Figure 11.6 (Benfield *et al.*, 1982) illustrates the effect of pH on reaction rate. In practice, the pH value is adjusted either by chemical dosing or, if there is significant dissolved carbon dioxide, by aeration.

Figure 11.6 Effect of pH on rate of iron oxidation (Benfield *et al.*, 1982)

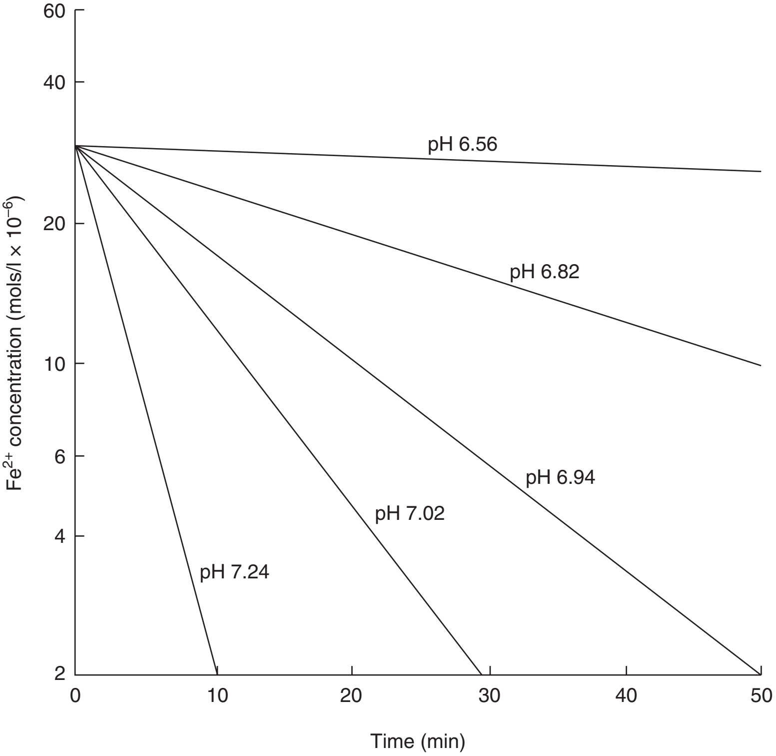
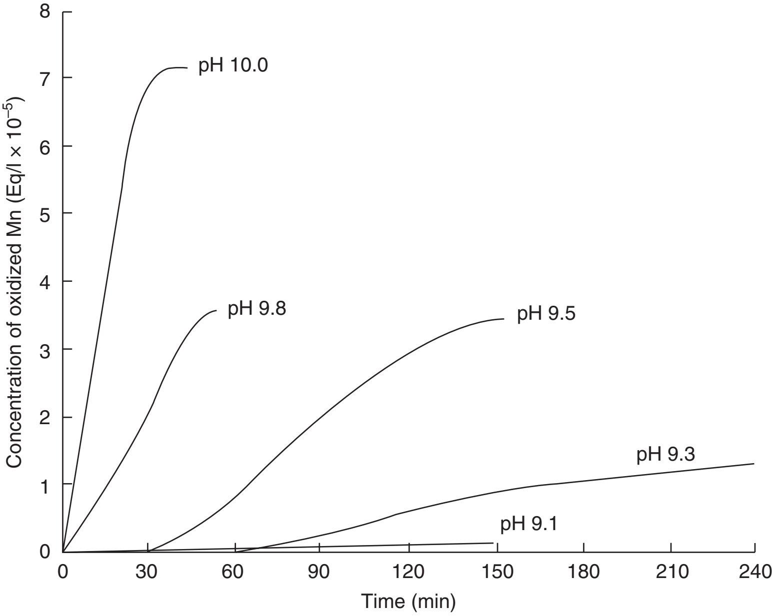


Figure 11.7 Effect of pH on rate of manganese oxidation (Benfield *et al.*, 1982)



Aeration may itself need to be followed by dosing to stabilise the water if it leads to precipitation of calcium carbonate.

Manganese has a reputation for being more difficult to remove. This is because it reacts more slowly than iron and may pass through treatment only to precipitate out later. Manganese oxidation by oxygen requires a pH value of 7.5–10, and proceeds only slowly below a pH value of 9.5. Figure 11.7 (Benfield *et al.*, 1982) illustrates the effect of pH on reaction rate. Where manganese is present it is important to dose a chemical oxidant, normally chlorine, to speed oxidation. However, catalytic effects complicate the oxidation of manganese. If water containing dissolved manganese is passed over manganese dioxide there is normally a strong catalytic effect and the manganese oxidises and precipitates onto the manganese dioxide more rapidly than would otherwise be expected.

Where there are significant concentrations of both iron and manganese it is fairly common to provide two-stage treatment, with iron oxidation and filtration at a pH value of around 7.0, followed by manganese oxidation and filtration at a higher pH value. The iron filters will normally be loaded at a rate of 6–9 m/h and the manganese filters at 10 m/h or higher. This is a reliable way of treatment but is expensive and arguably is not normally necessary as in most cases single-stage removal should be possible. However, the need to guarantee compliance with water-quality standards with minimum operator input, makes two-stage removal attractive to water companies.

Where iron or manganese are present, complexed with organic matter, as they can be in water drawn from the lower anoxic zones of reservoirs, oxidation and precipitation are not possible except at elevated pH. Where this is the case, coagulation followed by clarification and filtration is normally effective.

11.6.1 Other options

Ion exchange can be used for iron and manganese removal; in this case it is important not to aerate the water prior to treatment in order to avoid iron and manganese precipitating out of solution within the ion-exchange bed. Iron and manganese can both be removed biologically, but the optimum conditions required for each are different, precluding single-stage biological removal of both metals. In practice, biological removal of manganese is unusual whereas there are many plants that treat groundwater to remove iron (Morris and Siviter, 2001).

Biological iron removal involves passing water through filters, normally pressure filters, with careful control over dissolved oxygen. The key requirement is a low dissolved oxygen level, of around 1 mg/l, in the feed water, which should not be chlorinated. High filter loadings can be used, and because the iron is present in a dense form, the filters can treat waters with a higher iron concentration than could be treated by a conventional oxidation/filtration process. After treatment, the water needs to be aerated and disinfected. The process is not suitable for all groundwaters.

11.7. Nitrate removal

There has been a steady increase in nitrate levels in many groundwater sources. This, together with the nitrate standard of 10 mg/l as nitrogen, introduced in the first EU Drinking Water Directive, has led to a number of sources being unsuitable for water supply without either blending with lower nitrate water, or removing nitrate. Where a source does have high nitrate levels the first preference is blending. Where this is impractical or expensive then nitrate removal will be required. There are two main options for nitrate removal: ion-exchange or biological treatment. It is also possible to use reverse osmosis or electrodialysis.

The simplest treatment option is ion exchange. Where this is used, a proportion of the flow passes through an ion-exchange unit using resin optimised for nitrate removal. The advantages of ion exchange are: simple operation, the process is independent of temperature, can be automated, and is essentially unaffected by varying nitrate concentrations. The drawbacks are the production from regeneration of a saline waste, which may cause disposal problems, and an increase in the corrosiveness of the treated water. The use of ion exchange for nitrate removal normally requires the use of a special resin to be practicable. A conventional resin removes sulfate ions in preference to nitrate ions. Thus, where both sulfate and nitrate are present, both are removed, reducing the effective capacity of the resin for nitrate, and increasing the regeneration water per unit of nitrate removed. There is a further potential problem associated with the use of conventional resins where both sulfate and nitrate are present. If regeneration is delayed until the capacity of the resin has been exceeded there will be a higher nitrate concentration in the treated water than in the influent. This is associated with the displacement of nitrate previously removed by sulfate. (However, in practice, this should not be a problem for an automated plant.) Thus, nitrate removal normally uses special resins that remove nitrate in preference to sulfate.

Biological de-nitrification relies on bacteria to reduce the nitrate to nitrogen gas. In theory, there are two basic approaches: the process can use either autotrophic or heterotrophic organisms.

Heterotrophic bacteria use an organic carbon source, such as acetic acid or ethanol to provide energy for growth. In this case the water requires to be dosed with the carbon source at a concentration that provides sufficient nutrient for the bacteria while avoiding the carryover of nutrient into treated water. Requirements include a temperature of greater than approximately 8°C and a low dissolved oxygen concentration, in order to ensure the reduction of nitrate. The process uses attached growth with the bacteria growing on a fluidised bed of sand. The process requires careful control and cannot easily be automated. The dose of the organic carbon source has to be matched to the nitrate level.

The use of autotrophic bacteria, which use inorganic carbon within the water as the source of carbon for growth, is not normally practicable as they grow much more slowly than heterotrophic bacteria and the process is troublesome.

11.8. Air stripping

Where water is contaminated with a volatile organic chemical, air stripping is often the most cost-effective method of treatment either to lower the concentration to an acceptable value, or as pre-treatment prior to GAC adsorption. Many volatile organic chemicals are poorly adsorbed by GAC whereas they are easily removed by air stripping. Examples of chemicals that can be economically removed by air stripping include trichloroethane and tetrachloroethane. Air stripping is also used for removal of carbon dioxide.

The theory of air stripping does not lend itself to a simple summary. It is based on the fact that if the partial pressure of a volatile chemical is greater in a solution than in the air with which it is in contact, the chemical will move from water to air. Air stripping provides the optimum conditions necessary for the transfer to proceed. The rate of transfer depends on many factors; the most important are given below.

- The difference in vapour pressures – the rate of transfer is proportional to the difference in vapour pressures which can be thought of as the driving force.
- The solubility of the volatile chemical (which is related to the Henry constant of the chemical; soluble gases have a low Henry constant).
- The surface area across which transfer takes place.
- The mixing within the liquid, bringing water with a higher concentration of pollutant in contact with air.
- The ease with which the volatile chemical diffuses within the liquid.

Air stripping normally involves a tower filled with a packing material. Water passes down the tower while air is blown upwards. In an air stripping process the air entering the tower will contain none of the volatile chemical being stripped from the water. As it passes upwards the concentration of the volatile chemical in the air increases to a maximum at the top of the tower. The opposite is true of the water; it has a maximum concentration of contaminant at the top, and a minimum concentration at the bottom. For a given system, the rate of transfer of the chemical will depend on the difference in vapour pressures between the water and air phases. In practice, the highest transfer rate will be at the top of the tower and will decrease as the water passes down the tower.

The process is usefully thought of as a series of transfer units in each of which the air and water vapour pressures attain equilibrium. In the top transfer unit, the equilibrium is between a relatively high concentration of the chemical to be removed in both the air and water; by the time the bottom of the tower and the last transfer unit is reached the concentrations in both the air and water are low. The design of an air stripping tower is then undertaken by determining how many of these theoretical transfer units are required; this will depend on the inlet concentration and desired effluent concentration of the chemical to be removed; the airflow; and Henry's constant for the chemical at the operating temperature. Fewer transfer units are required when

- Henry's constant is high
- the airflow is high
- the contaminant concentration in the feed water is low
- the required contaminant level in the treated water is relatively high.

(The latter three of these conditions mean that the contaminant concentration in the air leaving the stripping tower will be low, as can be demonstrated from a mass balance calculation for the air and water streams.)

The converse is that when the opposite of the above holds, more transfer units are required. In practice, of course the process is not a series of discrete units but is effectively a continuous process and the number of transfer units is calculated using a logarithmic formula.

There are various forms of packing used in air stripping towers; these are normally made of plastic. The aim is to provide the maximum surface area per unit of packing volume. One of the oldest forms is the Raschig ring, thin-walled open cylinders with a length equal to their diameter. However, these are not particularly efficient and more complex shapes are common. For a given air stripping system, the key parameter of a particular form of packing is the height of a transfer unit. This and the number of transfer units defines the height of tower required. Thus, the design of an air stripping tower needs to take account of the factors touched on above. In practice, the specialist suppliers of air stripping towers will do the process design.

Once an air stripping tower is installed it is possible to vary the airflow to change the performance of the tower. Increasing the airflow decreases the number of transfer units required to attain a given treated-water quality, providing more effective treatment. Problems with air stripping include deposition of iron and manganese if present in the raw water and dirt and contamination from the air if no filter is provided.

Water that has passed through an air stripping plant may need post-treatment to stabilise it.

11.9. Arsenic treatment

The 1998 European Drinking Water Directive includes an arsenic standard of 10 μg/l, reduced from 50 μg/l in the 1980 Directive. This has meant that arsenic-removal plants have to be considered for some British water sources. Some arsenic removal may occur in treatment plants using ion exchange, activated carbon adsorption, or coagulation and filtration for iron-removal plants, but it had not been necessary in the past to design plants for arsenic removal.

The processes that can be used for arsenic removal are either adsorption onto a suitable media or reverse osmosis. Unless there are other water-quality issues that require reverse osmosis treatment, adsorption will normally be preferred. The process involves passing water through a bed of adsorbent media. Possible adsorbents include

- granular ferric hydroxide
- activated alumina
- bone char.

Ion-exchange resins and activated carbon may also be used.

Activated alumina and granular ferric hydroxide are considered to be the most likely adsorbents for a dedicated arsenic-removal plant. The design issues that need to be considered are the same as for other adsorptive processes: contact time; hydraulic loading rate; absorptive capacity; and backwashing/regeneration. The quality of the feed water needs to be high, with low turbidity and solids so adsorption is appropriate only for high-quality groundwater or treated surface waters. The empty bed contact time (EBCT) is normally a minimum of around 5 min. The permissible hydraulic loading rate depends on the particular media used, maxima of 10–20 m/h are appropriate. However, in practice, the acceptable hydraulic loading rate may be determined by EBCT and physical constraints; an EBCT of 5 min and a loading rate of 20 m/h requires a media depth of 1.7 m, and this is relatively high for pressure vessel adsorbers.

The adsorptive capacity of the media depends partly on the pH value of the water being treated. The media have a greatly increased capacity at lower pH values and a pH of 5.5–6.0 is desirable (WHO, United Nations Synthesis, 2001). However, the extra costs and complexity of pH adjustment may not be economic. Both granular ferric hydroxide and activated alumina can be regenerated by passing sodium hydroxide through the media but in practice it appears that for most sites it will be preferable to replace media when it is exhausted. As with all adsorptive processes, backwashing should fully fluidise the media to prevent mixing of media during operation.

11.10. Chemical dosing for lead control

The 1998 EU Drinking Water Directive introduced a maximum allowable lead concentration of 25 μg/l from the end of 2003, reducing to 10 μg/l at the end of 2013. Where lead is used in house connections or domestic plumbing the 25 μg/l standard is difficult to attain for many waters, and the 10 μg/l will be impossible without treating water to reduce lead solubility.

Where water comes into contact with lead in the presence of carbonates, the carbonate in the water reacts with lead forming one of several lead carbonates, mainly either lead carbonate ($PbCO_3$) or hydrated lead carbonate ($Pb_3(CO_3)_2(OH)_2$). The species formed depends on pH and alkalinity. The concentration of lead carbonate in water is a minimum at a pH value of around 9–9.5. At this pH value, the lead concentration is lowest in low-alkalinity waters, with a minimum lead concentration of around 25 μg/l (Cardew, 2000). However, low-alkalinity waters have low buffering capacity and it is often difficult to maintain the pH of the water as it moves through the distribution system.

Where the water contains dissolved orthophosphate the concentrations of lead are much lower as lead phosphate compounds are formed and these are less soluble than lead carbonates. Dosing orthophosphate reduces minimum lead concentrations to around 5 μg/l. In a low-alkalinity water, the pH range required for minimum lead concentrations is 7.5–8. As alkalinity increases a pH in the range of 7.2–7.8 is required.

Thus one approach to meeting the lead standard is to dose orthophosphate and control pH. Sufficient orthophosphate has to be dosed to maintain the necessary concentration of phosphate at the ends of the distribution system. The dose required depends on the chemical composition of the water, alkalinity, pH, and the condition of the distribution system. Typically, between 0.6 and 1.8 mgP/l is dosed to achieve the phosphate level required to minimise lead concentrations throughout the network. However, it is not sufficient simply to dose phosphate but it is also necessary to ensure that the optimum pH value is attained, and also maintained, as the water passes through distribution. This has implications for treatment, requiring that chemically stable water be produced at the treatment works. This can be difficult, particularly for low-alkalinity waters.

This approach is the only practicable treatment option to meet the new lead standards; the alternative would be to remove lead from plumbing systems.

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Chapter 12
Disinfection

12.1. Introduction

As water passes through a treatment plant, the various processes remove or inactivate many of the organisms present in the raw water. The final treatment in a water-treatment plant is disinfection of the treated water. The correct meaning of 'disinfection' is under threat and the word is often incorrectly used. Disinfection is not sterilisation, which implies the inactivation of all organisms; rather it is the killing of pathogenic organisms, those that cause disease. Disinfection is normally the most critical process from the point of view of supplying safe water. There are two aspects to disinfection: the first is the disinfection of the water to kill all pathogens that have passed through the various treatment stages, and the second is to apply a residual disinfectant so that the water leaving the treatment works remains safe as it passes through the distribution system to the point of use.

There are three types of pathogenic microorganisms that are of concern in water treatment: viruses, bacteria, and protozoa. It is not straightforward to classify microorganisms, which is complicated by there being a continuum of organisms, rather than a number of distinct types, but it is useful to understand some of the key differences. Viruses are the smallest. They are simple organisms, consisting of a core containing nucleic acid (either RNA or DNA) surrounded by an envelope of protein. They are obligate parasites able to multiply only in other living cells. It is quite possible to argue that viruses are not living organisms at all. Viruses are responsible for diseases such as polio, AIDS, rabies, and the common cold. Some pathogenic viruses can be transmitted in water. Bacteria and protozoa are both single-cell organisms. They are both normally heterotrophic, using organic matter as the source of energy. Some of the differences between bacteria and protozoa are that bacteria are smaller and lack a clear nucleus; they evolved much earlier than protozoa and tend to be less complex. Examples of bacteria of concern in water treatment include *Salmonella typhi*, which causes typhoid, and *E. coli*, which is the key indicator of faecal contamination of water. Examples of protozoa of concern include *Cryptosporidium parvum* and *Giardia lamblia*. A key factor of some of the parasitic protozoa, notably *Cryptosporidium parvum*, is that they have a complex reproductive cycle that includes the production of oocysts. These are the infective bodies that transmit the organism from host to host. The oocysts are thick-walled to protect the infective material within. The oocysts can be very resistant to chemical attack and can exist dormant for many months.

All the disinfection processes discussed in this chapter are effective against pathogenic bacteria and viruses. They may not be effective against some spore-forming bacteria, but these are not pathogenic. The major issue with disinfection relates to *Cryptosporidium parvum* and *Giardia lamblia*. These organisms are resistant to disinfection and are best removed by treatment, rather than inactivated by disinfection. It is now generally accepted that properly designed UV disinfection or ozonation can inactivate oocysts but that chlorination is ineffective at doses acceptable in water treatment. In the USA, ozonation and UV disinfection under specified conditions have, for several years, been considered effective for inactivation of *Cryptosporidium oocysts*. (This was not so in the UK where until 2007 there was a specific requirement to achieve a low level of oocysts in treated water, regardless of the disinfection regime.)

The need for a residual disinfectant to be carried into the distribution system is taken for granted in the UK, and over much of the world. However, in some countries (Rodman *et al.*, 1995) there is increasing public resistance to the use of chlorination as a residual disinfectant under normal conditions. If a distribution system is in good condition and clean, and the water entering the system is stable and low in dissolved assimilable organic carbon, then there will be little bacterial growth within the distribution system and it can be argued that disinfection is not required save where the security of the system has been compromised by a burst pipe or other event. It is quite possible that there will be increasing objections to chlorination in the future and such a view may become more widely held.

12.2. Disinfection during treatment

Clearly the disinfectant used must kill the organisms of concern while not being toxic to humans. Disinfection within the treatment plant may utilise one or more of the following processes

- chlorination
- ozonation
- UV disinfection
- dosing of chlorine dioxide.

Chlorination is now mainly used only for disinfecting fully treated water or high-quality groundwaters; this is to reduce the formation of trihalomethanes (THMs). These are chemicals which are similar in structure to methane (CH_4) but involve the replacement of three of the hydrogen atoms by halogens (normally chlorine). The compound of most concern is chloroform ($CHCl_3$), which is a known carcinogen. THMs are formed by reactions between halogens and organic matter in water, in particular the humic and fulvic acids found in peaty-coloured water. Commercial chlorine gas also contains some bromine, another halogen, and hence THMs can also contain this. In the past, it was common to pre-chlorinate water at the point of entry to the treatment works.

This controlled the attached growths of algae that otherwise occur and also killed algae present in the raw water, making them easier to remove. However pre-chlorination leads to higher THM concentrations and nowadays, it is unusual to pre-chlorinate, although it may be acceptable if an assessment of THM formation potential indicates that there will be no problems with THMs. Chlorine dioxide may be used for waters that are particularly prone to THM formation.

Ozonation has been covered in Chapter 11. It has been widely used in Europe for many years for disinfection in water-treatment plants. It is also now becoming common on large plants in Britain and North America. In Britain, it is used primarily for oxidation of pesticides, but it also disinfects. UV disinfection may be favoured where an existing plant does not have a contact tank for chlorination or to kill Cryptosporidium.

12.2.1 Residual disinfection

Because chlorine-based disinfectants are persistent, the residual disinfectant is always chlorine based, and may be

- chlorine (using either gas or hypochlorite solution)
- chloramine (using chlorine and ammonia)
- chlorine dioxide.

In distribution systems, chlorination is the most common disinfectant, but chloramination is also widely used.

12.3. Theory of disinfection

It is believed that chemical oxidants kill bacteria by rupturing cell membranes and destroying enzymes. There are many variables in the disinfection process, all of which affect its efficiency. The effectiveness of chemical disinfectants depends on several factors (Fair *et al.*, 1971) summarised below.

The organisms to be killed. Chlorination can kill essentially all bacteria and viruses, although some are more resistant than others. *E. coli* is considered more resistant than most pathogenic bacteria and some viruses and this is one of the reasons why it is used as an indicator organism. However, some pathogenic viruses are believed to be more resistant to chlorination than *E. coli*, and thus there are other requirements for disinfection than simply killing *E. coli.* Chlorination and other disinfectants do not under normal circumstances kill protozoan oocysts, most notably *Cryptosporidium parvum*, which therefore have to be removed or inactivated by treatment prior to chlorination.

The nature of the disinfectant. Chlorine and hypochlorite can form a number of different chemical species in water dependent on pH value and the presence of other chemicals in the water. The effectiveness of these as disinfectants varies greatly. This is discussed further below.

Concentration of the disinfectant. The effect of concentration is given by:

$$C^{n}T = K$$

where C is the concentration of the disinfectant, n is the coefficient of dilution (or the order of the reaction), T is the time to achieve a given percentage kill and K is a constant.

It can be seen that if n is greater than 1, then concentration has more effect than contact time, whereas if n is less than 1, then contact time has more effect than concentration. The values of n and K depend on many factors including temperature, pH, the disinfectant being used, and the organism being killed. For chlorine, n is normally taken as 1.

Contact time. In theory, the rate of kill of organisms should follow Chick's law. This states that:

$$\mathrm{d}N/\mathrm{d}t = -kN$$

where N is the number of surviving organisms and k is a rate constant for a particular disinfectant/organism combination.

In other words, the rate of kill is proportional to the number of living organisms. By integration it can be shown that when $kt = 1$, 43.4% of the original organisms will have been destroyed. However, in practice, the effect of contact time is more complex. For many organisms, the rate of kill increases with time. It is postulated that this is related to the time required for the disinfectant to enter and kill organisms.

Temperature. Increases in temperature lead to increased rates of chemical disinfection.

Other factors. These include the presence of solids in the water that may protect organisms from the disinfectant, and the concentration of organisms in the water. Thus, disinfection of turbid water can be ineffective, hence the need to disinfect water of low turbidity.

Disinfection is a complex process that is difficult to model, given the wide range of variables that affect its efficiency. In practice, simple design criteria that give adequate reassurance of effective disinfection are applied; the effectiveness of disinfection is verified using microbiological indicator parameters, notably the absence of coliform and *E. coli* organisms in the treated water.

12.4. Chlorination

Chlorination is the most common form of disinfection. It can involve two main alternatives: the use of gaseous chlorine, which is dissolved in carrier (motive) water before being added to the water to be treated; or the use of a solution of hypochlorite, normally sodium hypochlorite.

When added to water, chlorine reacts rapidly to form hypochlorous acid and hydrogen and chloride ions (effectively dissolved hydrochloric acid)

$$Cl_2 + H_2O \leftrightarrow HClO + H^+ + Cl^- \tag{12.1}$$

The hypochlorous acid may then dissociate:

$$HClO \leftrightarrow H^+ + OCl^- \tag{12.2}$$

The equilibrium points in these reactions are pH dependent. Equation 12.1 applies at a pH value of less than approximately 4, with the proportion of gaseous chlorine dropping to zero at a pH of 4. Equation 12.2 covers a pH range of 5–10, with the hypochlorous acid being fully dissociated at a pH of around 10. Figure 12.1 illustrates the effect of pH on hypochlorous acid dissociation.

Where a solution of sodium or calcium hypochlorite is used they dissociate as follows

$$Ca(OCl)_2 \leftrightarrow Ca^{2+} + 2OCl^- \tag{12.3}$$

$$NaOCl \leftrightarrow Na^+ + OCl^- \tag{12.4}$$

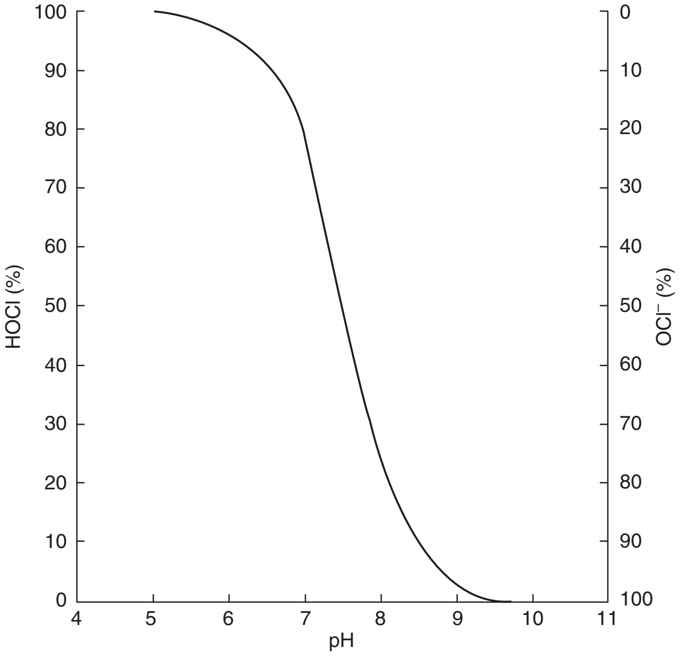
However, the equilibrium in Equation 12.2 also applies, and in addition the sodium or calcium ions will also affect the hydrogen ion concentration. Sodium and calcium hypochlorite therefore have identical reactions in water as chlorine, but the presence of the metal ions affects the concentration of hydroxide ions, resulting in an increased pH value, whereas chlorine gas lowers the pH value.

The other important set of reactions in relation to chlorination is the reactions between chlorine and ammonia. These are important, first, because they remove hypochlorous acid, and, second, because the compounds formed are disinfectants and are often used to provide residual disinfection in distribution systems. Chlorine or hypochlorous acid reacts with the ammonium ion to successively replace the hydrogen atoms with chlorine

$$NH_3 + HOCl \leftrightarrow NH_2Cl + H_2O \tag{12.5}$$

$$NH_2Cl + HOCl \leftrightarrow NHCl_2 + H_2O \tag{12.6}$$

Figure 12.1 Effect of pH on dissociation of hypochlorous acid



$$NHCl_2 + HOCl \leftrightarrow NCl_3 + H_2O \tag{12.7}$$

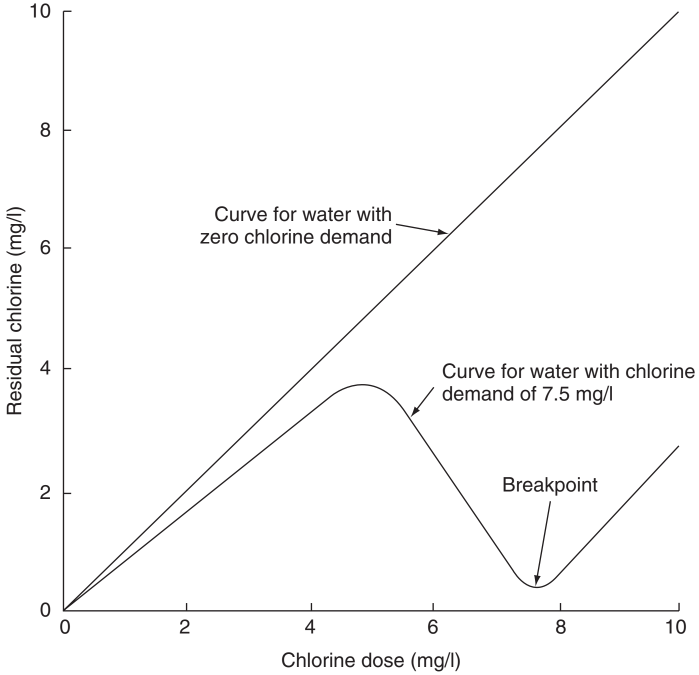
These reactions lead to the successive formation of monochloramine, dichloramine, and trichloramine. The relationship between the amounts of the three types of chloramine depends on the pH value and the NH_3 concentration in the water. As trichloramine can only form at very low pH values, the other two prevail in water treatment, dichloramine being much the more powerful bactericide. However, chlorination of water containing ammonia also leads to the production of nitrogen gas as follows:

$$2NHCl_2 + HOCl \rightarrow N_2 + 3HCl + H_2O \tag{12.8}$$

This releases nitrogen and converts hypochlorite to hydrochloric acid. There is also some production of nitrate, but this is a minor reaction.

Hypochlorous acid and the hypochlorite ion together are called ‘free chlorine’. The chloramines are known as ‘combined chlorine’.

Figure 12.2 Breakpoint chlorination curve (White, 1992; © Van Nostrand Reinhold)



12.4.1 Breakpoint chlorination

When chlorine or hypochlorite is added to water a succession of reactions occurs (Figure 12.2). Even the purest water tends to have slight traces of ammonia and other chemicals that react with chlorine. Thus, the first reactions result in the removal of some chlorine, due to reactions with ferrous or sulfide ions, for example, and the formation of monochloramine and dichloramine. This results in an increase in chlorine residual that closely follows the applied chlorine dose, with the difference due to other reactions. Once most of the ammonia has been converted to dichloramine Equations 12.7 and 12.8 start to occur. Equation 12.8 results in formation of hydrochloric acid and the chloride ion, neither of which are detectable as chlorine residual. Thus, the second stage of the curve shows the residual chlorine concentration decreasing with increased chlorine dose, with this apparently anomalous effect being associated with the formation of nitrogen. Once all the dichloramine has reacted to form trichloramine or nitrogen, the chlorine residual increases as the chlorine dose increases. The point at which the chlorine residual starts to increase again is referred to as the breakpoint. Prior to the breakpoint, chlorine is present predominantly as combined chlorine, after the breakpoint free chlorine predominates. Understanding this curve is critical to an appreciation of chlorination and chlorine demand.

The ‘chlorine demand’ of any given water is the amount of chlorine required to take the reaction to the breakpoint. In some waters, this is very low and often does not exceed 0.3 mg/l, but in other waters it may be much higher. It has to be determined by experiment.

12.4.2 Marginal chlorination, superchlorination, and dechlorination

The most effective form of chlorination is superchlorination. This involves dosing sufficient chlorine for there to be residual free chlorine; in other words, sufficient chlorine is dosed to ensure that the breakpoint is passed. This is the method generally favoured, as it is the safest method of disinfection. It is normal to dose sufficient chlorine to obtain a free residual dose of around 1.0 mg/l. However, this produces water in which the chlorine can be smelt or tasted by users, many of whom find it unpleasant. It is, therefore, common to allow sufficient contact time to ensure full disinfection and then to dechlorinate. This is normally done by dosing sulfur dioxide, or sodium sulfate, which reduces the chlorine to chloride, lowering the free chlorine to around 0.5 mg/l.

Marginal chlorination involves dosing sufficient chlorine to give the desired combined chlorine concentration. As the chlorine is combined the disinfection is much less effective. Arguably, marginal disinfection should only be used on very high-quality waters, either high-quality groundwaters or surface waters that have undergone extensive treatment including another disinfection stage. Marginal chlorination can give rise to complaints of taste and odour, resulting from the reactions between chlorine and the water being chlorinated, particularly where there is organic matter present in the water. For this reason, many suppliers prefer super-chlorination.

12.4.3 Chloramination

One of the problems often found with chlorination is that in order to maintain a residual throughout the distribution system, it may be necessary to apply a very high dose at the treatment works. This occurs where the distribution system is old and contains deposits with which the chlorine reacts, or in large systems with a high residence time. It is sometimes impracticable to dose at the high rate required to maintain a residual throughout the system; this could be because a high rate would give rise to complaints of chlorine taste and odour from consumers located near to the treatment works, or it might be because of water-quality issues – high chlorine levels can give rise to high THM levels in some waters. It may be that the problem can be overcome by booster chlorination at service reservoirs, or it may be possible to use chloramination. Chloramination involves dosing a controlled dose of ammonia to chlorinated water. This converts the chlorine to monochloramine. This is a weaker disinfectant than free chlorine, and for this reason it reacts less with material in the distribution system and is far more persistent. However, there are problems in that as the monochloramine decays it leads to the formation of nitrite. The 1998 EU Drinking Water Directive (WHO, 1998) has set a standard for nitrite at the treatment works to ensure that there is close control of chloramination. Chloramination is widely practiced, notably for much of London's water.

12.4.4 Chlorine dioxide

Chlorine dioxide is a compound formed by a reaction between sodium chlorite and either chlorine or hydrochloric acid. It is manufactured by mixing aqueous solutions in a small reaction vessel. At ambient temperatures, undissolved chlorine dioxide would be a gas. However, as the gas is explosive one of the key aspects of making chlorine dioxide is to ensure that it remains in solution:

$$5NaClO_2 + 4HCl \rightarrow 4ClO_2 + 5NaCl + 2H_2O$$

$$2NaClO_2 + Cl_2 \rightarrow 2ClO_2 + 2NaCl$$

The attractiveness of chlorine dioxide is that while it is a strong disinfectant and oxidising agent, it does not lead to the formation of THMs. It has a similar disinfection efficiency to free chlorine and is effective over a pH range of 6–9. At a pH value higher than 10 it is ineffective (Hall and Hyde, 1992).

A concern with chlorine dioxide is that its production can lead to the formation of both chlorate and chlorite as by-products. Chlorine dioxide can also break down to chlorate and chlorite in the distribution system. Chlorite has been associated with affecting reproduction in laboratory animals, and chlorate can be reduced to chlorite. To minimise chlorate and chlorite formation it is important that in the manufacture of chlorine dioxide there is precise control of the proportions of the reacting chemicals; it is also necessary to maintain a low pH value. There are no standards for chlorate or chlorite in either the current EU Drinking Water-quality Standards or current UK Drinking Water-quality Regulations. However, chlorite and chlorate are controlled in the UK by a requirement that the combined concentration of chlorine dioxide, chlorite and chlorate must not exceed 0.5 mg/l as chlorine dioxide (UK Drinking Water Inspectorate, 2012). The USA sets a limit of 1 mg/l for chlorite.

12.5. Required disinfectant dose and contact time

Disinfection is required to kill all pathogens. In practice, the design criterion will be the effective killing of *E. coli*. To do this, the two key parameters are contact time t and disinfectant dose C. It is normal to quote Ct values in the design of disinfection systems. This represents a simplification of a complex relationship but appears satisfactory provided there is a safety margin included.

Table 12.1 Contact time required for 99% destruction of *E. coli* (Hall and Hyde, 1992)

	Concentration: mg/l	Contact time: min	pH	Temperature: °C
Free chlorine	0.1	0.4	6	5
	0.1	6	8.5	20–29
	0.4	1	8.5	20–29
Monochloramine	0.1	60	4.5	15
	1	6	4.5	15
Dichloramine	1	64	9	15

12.5.1 Contact time required

The effectiveness of the various forms of chlorine differs significantly. There is a wealth of published data on this. Hall and Hyde (1992) quote the data in Table 12.1. This illustrates two of the key points relating to disinfection using chlorine.

- pH is important. This is because hypochlorous acid is much more effective than the hypochlorite ion. Thus the pH value for disinfection using chlorine should be no higher than 8.0. Above this hypochlorous acid increasingly dissociates into hydrogen and hypochlorite ions.
- Free chlorine is much more effective than combined chlorine.

WHO (1993) suggest that effective chlorination requires: a free chlorine residual of more than 0.5 mg/l; a contact time of at least 30 min; a turbidity of less than 1 nephelometric turbidity unit (NTU); and a pH of no more than 8.0. This is equivalent to a Ct of 15.

US standards (US Surface Water Treatment Rule, 1989) require a three-log removal/inactivation of *Giardia* cysts and a four-log removal/reduction in viruses – both reductions being totals applied to filtration and disinfection. Where disinfection alone is used, without filtration or clarification, a Ct of 6 is required for a four-log inactivation of viruses at 10°C and a pH value of 6–8, and a Ct of 10^4 for a three-log inactivation of *Giardia*.

12.6. Ultraviolet disinfection

UV disinfection involves passing water through high-intensity UV radiation. The radiation kills or inactivates bacteria and viruses by affecting their RNA and DNA. UV radiation at a wavelength of around 260 nm is normally used. Water is passed through a chamber or channel containing UV lamps that expose the water to a controlled dose of UV radiation for a minimum period. The UV radiation penetrates organisms and initiates photochemical reactions within the cells inhibiting or killing the organisms. It is very effective against bacteria and viruses, but less effective against larger protozoa. The radiation is produced by lamps that produce light at a wavelength dependent on the type of lamp used.

Key parameters in UV disinfection are the power intensity (mW/cm^2) and the dose applied (mWs/cm^2), the intensity multiplied by the exposure time. A minimum dose is 15 mWs/cm^2, which would normally produce water of acceptable bacteriological quality, but 25–40 mWs/cm^2 is desirable.

For UV disinfection to be effective, the water has to have high transmissivity for the UV used. The transmissivity or adsorbence of water is measured to assess the suitability of water for UV disinfection and to estimate if a higher dose is required to account for adsorbence within the water. Normally, there are no problems with potable water but where iron levels are above 0.1 mg/l or water is hard (USEPA, 1999), there can be problems with scaling of the lamps. Ideally, a water sample should be tested before using UV disinfection; the most likely problem with drinking water is scaling.

UV disinfection is particularly useful for groundwater sources which pump directly into supply and which do not have a chlorine contact tank. UV disinfection provides assurance that the water leaving the site is safe, with a residual disinfectant required only to maintain the safety of the water.

12.7. Ozonation

Ozonation has been covered in Chapter 11. It is a powerful disinfectant but is not normally used solely for disinfection in the UK. The dosages and contact times required for effective disinfection are lower than those needed for treatment of organic chemicals and thus where ozone is used for other purposes it also provides very effective disinfection. For disinfection an ozone residual of 0.4 mg/l and a contact time of 4 min will effectively kill all bacteria and viruses of concern, providing there is either plug flow or two cells to prevent short-circuiting.

12.8. Other methods of disinfection

The methods covered above are those normally used in conventional water treatment. Other processes that may be encountered include

- dosing of bleaching powder – consists largely of calcium hypochlorite
- boiling water – suitable for emergency use only
- use of metal ions – the use of silver ions to disinfect water is possible; this is only suitable for emergency short-term use for potable water
- dosing of potassium permanganate – can be used but is not very effective
- membrane processes – ultrafiltration and reverse osmosis membranes remove microorganisms
- dosing of bromine or iodine – emergency use only.

12.8.1 Chlorine dosing equipment

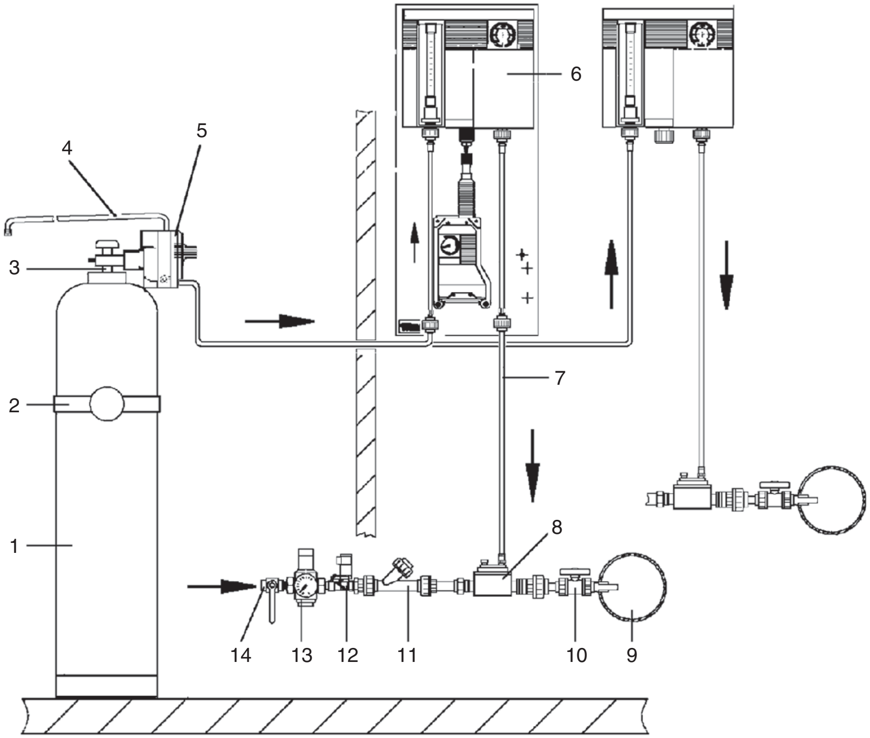
Traditionally, chlorine has been dosed using gaseous chlorine, in cylinders containing around 70 kg of chlorine used on smaller works or using drums containing up to 1000 kg on larger works. Where gaseous chlorine is dosed the system consists of the following.

- Chlorine store with chlorine gas detection equipment and automatic shutdown.
- Automatic changeover equipment on the drums/cylinders.
- Gas evaporator for plants using a large quantity of chlorine. If the rate at which the chlorine leaves the cylinder is too high, the rapid transition from liquid to gas causes extremely low temperature, which, in turn, causes chlorine crystals to form which block the feed pipe. One remedy is to lower the rate of flow leaving the cylinder by increasing the number of cylinders connected. The critical rate of draw-off varies with the size of the cylinder. A 1000 kg drum can discharge at up to 10 kg/h in average temperatures but a 70 kg cylinder will ‘freeze’ at very much lower rates. There is no theoretical limit to the number of cylinders that can be connected in parallel, but in practice, in big works, where flows exceed 60 kg/h it becomes cheaper to insert a special ‘evaporator’ between the cylinder and the regulating valve.
- A regulating valve to reduce the gas pressure down to around 150–200 millibar.
- A gas control valve to control the rate of gas supplied to the injection unit – which is known as an ‘Ejector’.
- The gas ejector that injects the chlorine into a supply. The gas is injected into a supply of high-pressure motive water using an eductor, a venturi system that creates a vacuum that sucks in the chlorine.
- An injection point where the motive water with a high concentration of chlorine is injected into the water to be chlorinated.
- A control system comprising a flowmeter and downstream sampling and analysis equipment used by the residual controller to control the chlorine gas rate by the gas control valve.

Different equipment suppliers use different control systems and the supplier should do the design of such systems.

Specialist suppliers supply chlorine systems. The concentrated chlorine solution is corrosive, and all piping and pumps have to be of suitably resistant materials. Figure 12.3 is a diagram of a typical chlorination system. The gas piping is largely under vacuum, minimising the risk of any chlorine gas escaping. There needs to be a supply of high-pressure motive water, typically at a pressure of 2 or 3 bars higher than the pressure at the point of injection into the main flow. Chlorine gas is injected into this motive water, by the gas injection system, and the motive water is then introduced into the water to be chlorinated. The motive water may be supplied by dedicated

Figure 12.3 Components of typical gas chlorination system (courtesy of Siemens Water Technologies)



(1) Chlorine gas cylinder; (2) cylinder retainer; (3) cylinder valve; (4) vent; (5) vacuum demand valve; (6) chlorine control unit; (7) chlorine suction pipe; (8) injector (vacuum eductor); (9) water main to be chlorinated; (10) main injection unit; (11) non-return valve; (12) solenoid valve; (13) pressure reducing valve; (14) isolation valve for carrier water

pumps or may use water from the treated-water delivery mains leaving the site. Whichever is used, it is absolutely vital that the supply is continuous.

Where a treatment works uses super/de-chlorination or chloramination there will be similar systems for either sulfur dioxide or ammonia.

Large gaseous chlorine installations are recognized as a major risk to life. A large escape of gas could kill people in the vicinity of the works and increasingly tight regulations apply to works using gaseous chlorine. In England, works storing large quantities of chlorine come under The Control of Major Accident Hazards Regulations (COMAH), which also apply to major chemical works, and similar regulations apply elsewhere in the UK and overseas. Concerns over the possible effects of leaks and the regulations themselves are leading to some water companies changing from chlorine gas to using either bulk sodium hypochlorite or an on-site electrolytic chlorination (OSEC) system.

12.8.2 Control and safety measures

Chlorinators are very reliable but, as a check on the amount of chlorine being used, it is common practice to mount the gas cylinders on a weighing machine. The daily loss of weight of liquid chlorine can be compared with the amount used as indicated by the chlorinator setting and any discrepancy investigated.

Chlorine is a deadly gas and extensive safety precautions are required. In the last few years, there has been a complete change of thought about safety measures. It used to be considered that the chances of an accident were negligible and that the amount of gas to be dealt with would be small. It was common practice to give chlorine rooms adequate ventilation at low level, because chlorine is heavier than air, and all doors opened directly to the outside and not to internal rooms. However, now provision is always made for a more serious accident, envisaging a large-scale escape of gas, and it is the general consensus that it is advisable to

- house chlorine apparatus in gas-tight rooms
- install sensors that can detect chlorine in the atmosphere
- provide equipment – with extractor fans, chlorine-neutralising chemicals and sprays which start automatically when the sensors signal that chlorine gas concentrations in excess of 2 ppm are present in the atmosphere; however, the extractor fans should be stopped in the event of a major leak, to prevent large quantities of chlorine being passed into the atmosphere.

Chlorine safety measures are best left to specialists.

12.9. Sodium hypochlorite and OSEC systems

Sodium hypochlorite solution is available in bulk for large works, or in smaller containers for small works. Where used it is dosed like any other liquid chemical using a standard chemical dosing pump with flow control and dosing controlled by the chlorine residual. Sodium hypochlorite comes in a standard strength solution with a nominal chlorine concentration of up to 15% available chlorine. However, the solution deteriorates over time and storage times should be limited to no more than 2 months supply.

OSEC involves electrolysis of a solution of sodium chloride solution to produce sodium hypochlorite solution. Hydrogen is also produced as a waste product. The process produces sodium hypochlorite solution with a chlorine content of 6–9%. The system uses bulk dry sodium chloride, which is dissolved in a saturator using softened water.

Approximately 3.5 kg of salt is required to produce 1 kg of chlorine. This represents approximately 50% of the theoretical potential production from the salt; the balance of the salt remains in the hypochlorite solution and is dosed into the treated water. Power use is around 5 kW h/kg of chlorine. OSEC is becoming widely used on large plants, but some companies prefer to use bulk sodium hypochlorite, which may be more expensive but is easier to operate.

Potential problems with OSEC include the production of chlorate, for which the UK government sets a limit of 0.7 mg/l (UK Drinking Water Inspectorate, 2008), and bromate from bromide present as an impurity in the salt, for which the water-quality standard is 10 mg/1 of bromate.

12.9.1 Monitoring and testing

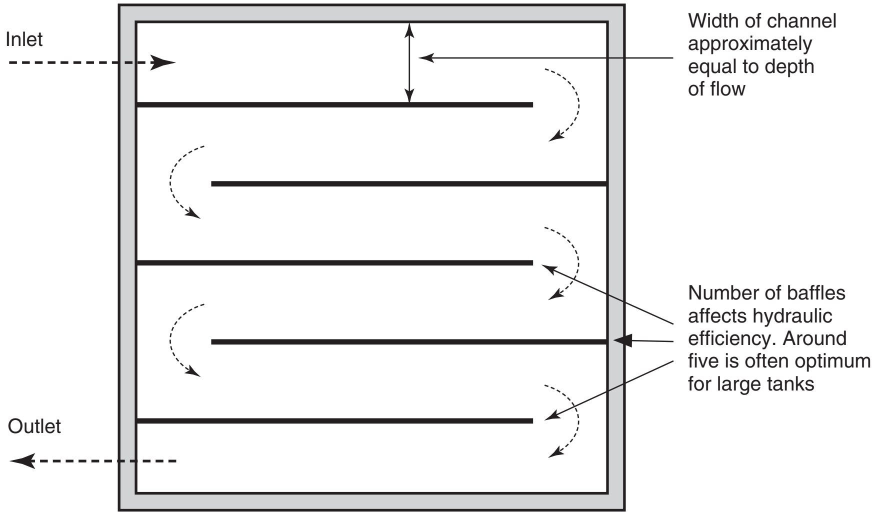
In the UK, all treated water leaving a treatment works is continuously monitored for chlorine residual, normally using a triple validation instrument. There will be high and low chlorine concentration alarms, and normally a ‘low low’ alarm that will shut down the works if it is maintained for more than a few minutes. Monitors and alarms should be regularly checked and calibrated.

12.10. Contact and treated-water tanks

Where water has been chlorinated it passes to a contact tank. This should have a fixed volume and will typically be sized to provide 30–60 min retention at full plant capacity. The contact tank should have a more or less constant volume independent of flow, with an outlet weir to maintain water level under low-flow conditions or when the downstream hydraulic head is low. The tank should be baffled to prevent both short-circuiting and dead zones. A typical contact tank for a small or medium sized works would be approximately square, with two internal baffle walls and a 60-min retention time. Water would enter in one corner and flow round the baffles to an outlet weir located in the far corner from the inlet. Such an arrangement would ensure an acceptable contact time after allowing for short-circuiting.

For larger plants, it becomes more economical to reduce the storage nearer to 30 min while optimising the design to reduce short-circuiting. This involves introducing more baffles into the contact tank, or changing proportions, to minimise short-circuiting. Figure 12.4 shows a possible layout. Clearly there will be significant mixing where the flow direction through the tank is reversed, and an ideal layout would be a very long tank, or pipe, with no flow reversal. However, in the real world a layout similar to that shown is more likely to be economic, with the width of each channel approximating to the depth of flow. Lightweight partitions can be used if securely fixed. The performance of a contact tank can either be modelled hydraulically, computer modelled using computational fluid dynamics, or for an existing tank can be assessed by undertaking a tracer study.

Figure 12.4 Typical layout of a chlorine contact tank at large works



The performance of a contact tank in terms of retention time is normally defined by two key primary parameters: the T10, which is the time for 10% of a tracer to pass through the tank; and the average retention time, which is tank volume divided by flow. A secondary measure is the hydraulic efficiency, which is the T10 time divided by the average retention, normally expressed as a percentage. Typical values of hydraulic efficiency for a well-designed tank are of the order of 60 to 65%.

After the water leaves the contact tank, the chlorine dose would be adjusted, normally trimming by de-chlorination, before passing to the treated-water storage and thence to distribution.

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Chapter 13
Waterworks wastes and sludge

13.1. Introduction

All water-treatment processes produce wastes of some form and this chapter considers the liquid wastes produced by the main processes of clarification and filtration. These wastes comprise the bulk of the waste produced by water-treatment plants. Wastewater from some other processes that involve backwashing, notably granular activated carbon (GAC) adsorption, will also normally be treated with these wastes. Other wastes are also briefly considered.

A 1998 survey on waterworks sludge production in the UK (UKWIR, 1999) gave the figures in Table 13.1 for sludge production and disposal. The survey is of interest in that it shows disposal to landfill was the predominant disposal route followed by disposal to a foul sewer. However, the works discharging to a sewer were all operated by companies or authorities responsible for both water and sewerage services, and apparently the water-treatment plants were not charged for discharging their wastes to the sewer. If the economic cost were to be charged, disposal to sewer would almost certainly reduce. This is because sewage treatment does not significantly reduce the quantity of solids that needs to be disposed of, but it does involve diluting the waterworks wastes with sewage, passing them through the sewage-treatment works, and then still having to dispose of the solids in the sewage sludge.

Another reason for treating wastes is that where lagoons are used the land occupied is large and often valuable or required for other purposes. Also, process improvements may mean a larger quantity of waste is produced leading to capacity problems in lagoons. There may also be difficulties in complying with the quality requirements of a new consent. Therefore, for a variety of reasons, it will become unusual in the UK to have anything other than mechanical treatment for the wastes produced, with recycling of water recovered from the wastes. However, in other areas of the world lagoons may well still be appropriate as a destination for sludge.

In the UK, most liquid discharges from WTWs are classified as trade effluent and thus, where the discharge is to controlled water (defined in Section 104 of the Water Resources Act 1991), consent is required from the Environment Agency (or the Scottish Environment Protection Agency); where the discharge is to a sewer, a trade effluent consent is

Table 13.1 Estimated WTW sludge production and disposal routes for the UK in 1998

Type of sludge	Production (dry solids)		Disposal (percentage of each type of sludge)				
	Quantity: t/year	Percentage	Landfill	Foul sewer	Lagoon	Novel management	Other
Natural	5 850	4.5	1.6	59.6	5.9	15.8	17.0
Alum coagulation	58 528	44.8	51.1	34.9	0.9	4.8	8.2
Iron coagulation	43 040	32.9	56.9	20.4	4.2	13.5	5.1
Softening	22 240	17.0	89.5	0.0	0.0	7.8	2.7
Other	1 098	0.8	28.2	4.0	0.0	0.0	67.8
Total	130 756	100					

Notes: 1. Based on UKWIR report Sewage Sludge SL-09, 1999; 2. Total water production estimated as 16 500 Mld

required from the sewerage undertaker. Wastes that are disposed of to a landfill are classified as industrial waste and are controlled wastes; they therefore fall under the requirements of Part II of the Environmental Protection Act (EPA) 1990 (1990), and the Waste Management Licensing Regulations (WMLR) 1994 (1994). A waste management licence is therefore needed to dispose of the waste.

In the past, it was quite normal for little attention to be paid to wastewaters. Often they were sent to lagoons or disposed of to a sewer and effectively forgotten. This is clearly no longer possible. Wastes can conveniently be thought of as falling into four main categories.

- Liquid chemical wastes.
- Washings from screens and microstrainers.
- Dirty washwaters from filters and adsorbers.
- Sludge.

These will be considered in turn after consideration of sludge quantities.

13.2. Expression of sludge quantities

The solids concentration of sludge should be expressed as a weight (w) of dry solids per unit volume (v), as w/v, or a proportion of the weight of sludge that is dry solids, as w/w. The solids concentrations found in liquid water-treatment sludge are from around 1 to 50 kg/m^3, with most at the lower end. At this level of solids the specific gravity of the sludge is approximately 1, and 1 m^3 of sludge weighs approximately 1 tonne. In practice, it is usual to see solids content expressed as a percentage. This is ambiguous as, for example, 1% DS could mean that 1% of the sludge is solids either by volume or weight. In practice, it is often used both as being the percentage of a weight of sludge that is solid material (i.e. w/w), and as the weight of sludge solid material per unit volume (i.e. w/v). The former is a correct usage whereas the latter assumes a specific gravity of 1 for the sludge and thus is incorrect. However, at low sludge solids concentrations the specific gravity of a sludge is approximately 1 and there is no significant error in expressing, for example, a sludge with 10 000 mg DS/l as 1% DS.

In accordance with common practice, percentages have been used in this chapter and unless otherwise indicated the percentage indicates *w*/*v*, with *x*% meaning *x* g of dry solids/100 ml. If the basis is weight of dry solids per weight of sludge this is indicated by the symbol *w*/*w* as in *x*% (*w*/*w*).

13.3. Estimation of sludge quantities

The sludge produced by a water-treatment plant is derived from six sources.

- Suspended solids in the raw water.
- Colour that is removed during treatment.
- Dissolved chemicals that precipitate during the process – mainly dissolved iron and manganese, and any hardness precipitated during softening.
- Coagulants added during the process that precipitate out during treatment.
- Other chemicals added during treatment such as powdered active carbon (PAC) and bentonite.
- Biological growth within the processes – difficult to estimate and not normally significant except for slow sand filters.

There are two methods of calculating sludge quantities. They are either the calculation of dry solids production based on raw water quality and likely chemical dosages during treatment, or by applying rules of thumb to estimate sludge volumes as a proportion of water treated. The former is the proper way, but such are the uncertainties that it is always sensible to check the reasonableness of the result using the latter method.

13.3.1 Suspended solids

If there are data on raw-water suspended solids then this is easy. However, for water-treatment it is more common to have turbidity data. In the absence of any other data, suspended solids (in mg/l) can be assumed to be twice the turbidity (in nephelometric turbidity units (NTU)).

13.3.2 Colour

Precipitated colour contributes to sludge quantities. Sludge production (in mg/l) of between 10% and 20% of the colour removed (degrees Hazen) are quoted; WRc have recommended 20% (Warden, 1983).

13.3.3 Dissolved chemicals

Assuming that iron precipitates as ferric hydroxide, each mg/l of iron removed produces 1.9 mg/l of dry solids.

Table 13.2 Solids production from different coagulants

Coagulant dose expressed as mg/l of	Solids production per mg/l of coagulant
Al	2.9 mg/l
Al_2O_3	1.5 mg/l
Fe	1.9 mg/l
Fe_2O_3	1.35 mg/l

Where water is softened using lime and soda the solids production is given by:

$$\text{solids (mg/l)} = \text{lime dose} + 1.26 \times \text{decrease in hardness} - 0.26 \times \text{sodium carbonate dose}$$

where dosages are in mg/l and hardness is $mgCaCO_3/l$.

13.3.4 Coagulant

Assuming that iron and metal coagulants precipitate as hydroxides the quantities of sludge produced by metal coagulants are as given in Table 13.2. Polymer doses are removed as dosed; however, these are low and are unlikely to be significant.

13.3.5 Other chemicals

Where powdered activated carbon or bentonite is added, the sludge solids will increase by the applied dose.

Thus, if the quality of the raw water is known together with the likely coagulant dosage, it is possible to estimate the quantity of dry solids that will be produced as sludge during treatment. However, the estimate will only be as good as the data on raw water quality, and the accuracy of chemical dosages required during treatment. Quantities will vary with variations in raw water quality and a judgement will need to be made as to the quantities used for the design of sludge handling and treatment facilities. Having calculated the theoretical quantities it is always useful to check the results against rules of thumb for sludge production.

13.4. Rules of thumb

It is common to use rules of thumb for estimating sludge quantities for the two main sources of sludge and wastewater, clarifiers, and filters. Quantities are clearly not accurate but reflect the order of magnitude of production expected.

13.4.1 Clarifier sludge

Clarifiers produce sludge with solids contents typically between 0.1% and 0.8%. The lower values apply to raw waters with low colour and turbidity, and the higher values to waters with high turbidity treated in clarifiers using sludge cones. A typical solids concentration from a modern clarifier is 0.2–0.3%. Volumes of clarifier sludge normally range from 1–2.5% of throughput; values above 3.5% indicate an unusual water or operational problems. WRc (Warden, 1983) quotes an average of 2.2% for aluminium sludge and 1.8% for ferric sludge. The higher values for alum may reflect overdosing of alum for pH control. Note, however, that some proprietary designs of clarifier have integral sludge consolidation and these can produce much thicker sludge.

DAF produces sludge with a solids content of around 2% dry solids or higher; thus the quantity of sludge, though not of solids, is lower than for clarifiers.

There are two ways to calculate the weight of solids removed in clarifiers. Clarification should produce water with a suspended solids value of around 5 mg/l (Twort *et al.*, 2000), with the balance of the solids removed by the clarifiers. Alternatively, Warden (1983) quotes removals of 50–90% of solids in clarifiers with a typical value of 70%.

13.4.2 Filter washwater

In a normal works, all remaining solids are removed in the filters. Filter backwash water typically contains about 0.03% of solids and the volumes range from as low as 1% up to 5% of throughput, with a typical value of around 3%. It is common for the quantity to vary if raw water and clarified-water quantity vary over time causing more frequent backwashing at times of poor water quality. It is increasingly common for there to be detailed backwashing requirements in specifications for rapid gravity filters and these often result in rates of backwash water production at the higher end of the numbers quoted above.

13.4.3 Liquid chemical wastes

Liquid chemical wastes are discharged from ion-exchange plants used for softening or nitrate removal which are regenerated with brine. Depending on the degree of treatment provided up to around 5% of the total throughput of a softening plant may be discharged as waste, with a much lower proportion for a nitrate-removal plant. The wastes will contain the ions removed by treatment, mainly calcium and magnesium from a softening plant or sulfate and nitrate from a nitrate-removal plant, together with sodium and chloride.

In the UK, such wastes generally have to be discharged to the public sewers as it is not possible to obtain a permit to discharge such wastes to surface waters.

Reverse osmosis and nanofiltration also produce a liquid chemical waste. This is difficult to dispose of and is commonly best discharged to a sewer. However, in hot arid countries evaporation lagoons may be another option.

13.4.4 Debris from screens and microstrainers

Screens produce small quantities of easily drained waste that are easily disposed of. Microstrainers produce waste that will easily dewater to a consistency where it can be handled and disposed of. In practice, waste from both screens and microstrainers is not normally a problem.

13.5. Treatment options

13.5.1 Dirty washwaters from filters and adsorbers

Filters and adsorbers require regular backwashing. The washwaters from both are very dilute and normal practice is to take them to tanks which function both as flow-balancing tanks and settlement tanks. The solids are allowed to settle and the supernatant is recycled. A typical arrangement would be to have two tanks, each with the capacity for several filter backwashes. Once a tank was full, backwash water would be diverted to the second tank, and the contents of the first tank allowed to settle. After a period of two or more hours of quiescent settlement supernatant would be removed, typically by a floating arm draw-off, and pumped back to the head of the works. Settled sludge would be pumped to the sludge-treatment facilities. The tank would then be available to receive more flow. For large works with a large number of filters, it is more economic to have some form of upflow settlement tank that would operate intermittently, as filters were backwashed. Smaller works might only have a single tank capable of holding the water from a full set of filter backwashes.

Another option is not to settle but simply to use a flow-balancing tank that allows the washwater to be returned at a steady rate to the head of the works. This simpler option is now less common as it involves recycling all material removed in the filters, with the particular concern being potential recycling of *Cryptosporidium* oocysts. The maximum return rate is generally stated to be 5% of the flow being treated. If there is no provision to deal with settled solids, there will need to be some means of mixing the tank contents to prevent settlement.

The sludge from backwash settling tanks will typically have a solids concentration of around 1%.

13.5.2 Clarifier sludge

In a conventional water-treatment plant, the main source of sludge is the clarification stage. Some additional sludge may be generated from the settlement of filter backwash water. Assuming the sludge is not discharged to a sewer it will normally be thickened and dewatered. In the past, it was not uncommon for clarifier sludge to be discharged to lagoons and left to consolidate, but this is generally no longer appropriate in the UK.

Sludge from softening is of a different nature to coagulation sludge, having a higher inorganic content of inorganic material. It is considered separately later.

13.5.3 Thickening

When sludge is dosed with polymer, at the correct rate in a suitable mixer which breaks up any existing flocs, and allowed to settle, flocs reform and the sludge separates into layers of clear supernatant and flocs with a clearly defined boundary between the two. If settling is allowed to continue, the layer of flocs will consolidate such that all free-water escapes. A sludge thickener is normally designed to achieve this state of no free-water. The sludge from such a process will have a solids content that depends on the nature of the sludge and may be more than 10% (*w*/*w*) although 5–10% is more common.

On large works, thickening will be by means of a continuous thickener. This is a circular tank, either flat-bottomed or with a gentle fall towards the centre, with a conical sludge hopper in the centre. The thickener contains a rotating sludge rake that draws sludge towards the central hopper. The tank is typically 2–3.5 m deep and with a diameter to give a hydraulic loading of 1.5 m/h. The depth depends on the nature of the sludge and how easily it thickens. Pilot trials are required to determine this, although providing the tank is at least 3 m deep it can normally be operated satisfactorily.

Sludge is dosed with polymer and introduced into a central feed well at a depth of around 1 m below water surface level. Flocs form as the sludge enters the thickener and on entry the flocs start to settle and the free-water to rise. The water rises up and is collected over an outlet weir. The flocs settle, and as they settle they are compressed and dewatered so that by the time they reach the bottom of the tank all free-water has been released. The thickened sludge is then withdrawn, normally to a holding tank. There is a distinct interface between the sludge and the supernatant, and withdrawing sludge to maintain the interface at the required level controls the process. The supernatant is normally clear with low turbidity, and provided the turbidity is low, typically less than 10 NTU, it can normally be recycled without further treatment.

Providing the tank is properly sized and operated, the solids content of the thickened sludge is independent of the solids content of the feed to the thickener, depending solely on the nature of the sludge itself.

Two points to note are that polymer dosing is essential for the satisfactory operation of these thickeners and that, in the UK, there are restrictions on polymer dosing where the supernatant is returned to the main process stream and enters the water supply (UK Drinking Water Inspectorate, 2008).

13.5.4 Dewatering and disposal

The dewatering process used should be considered in parallel with the ultimate disposal route. The cost of thickening increases with the solids content of the sludge produced and the designer should therefore consider the solids content required. This depends on the ultimate disposal route including transport distance. In practice, most sludge goes to a landfill and the requirement is a solids content of more than 20% in order that the sludge can be handled as a solid waste. The performance of the various forms of mechanical dewatering plant with different sludge varies widely and is susceptible to process changes that alter the nature of the sludge. Performance claims therefore need to be treated with some scepticism, and should be supported by site testing, ideally in a pilot plant.

13.5.5 Filter plate presses

Plate presses comprise a series of vertical steel or heavy-duty plastic plates, normally square, each with a recess on either side. In each recess there is a mat of filter cloth so that in the gap between two adjacent plates there are two mats of filter cloth. The plates are mounted in a frame that allows them to be separated when required. The principle of the filter press is that the plates are locked together. Sludge is then pumped into the gap between each set of filter cloths. The cloth retains solids and the liquid passes though the cloth and is drained away through openings in the plates. As the filter mat becomes coated with solids it becomes more efficient at retaining the solids, and the pressure in the system rises. Sludge is pumped into the press over a period of several hours, after which the recesses are full of dewatered sludge, known as cake. The solids content of the cake depends on the nature of the sludge, the pressure used, and to a limited extent on the time of pressing. Typically, a solids content of around 25% (*w*/*w*) can be achieved. In general, filter plate presses produce cake with the highest solids content compared to other commonly used thickening processes.

At the end of the charging cycle the press is allowed to stand for a while to further dewater. The pressure is then released and the plates separated, allowing the cake to be knocked off the filter cloths. This drops down into a skip or on to a conveyor. The plates are then brought back together and the process is repeated. It is normal to have two or three cycles per day.

Older installations require significant operator input, particularly for removing the sludge from the cloths, but fully automatic presses are common now. As this is a batch process balancing storage of sludge prior to pressing is required.

13.5.6 Centrifuges

Centrifuges can be used to dewater waterworks sludge. They consist of a horizontal cylinder that spins at a speed of around 4000 rpm, giving an acceleration of around 2000 *g*. Sludge is introduced into the centrifuge at one end and is thrown out onto the inside of the revolving bowl. The centrifuge is designed such that there is a pre-set thickness of sludge and liquid on the bowl. Sludge solids that collect on the inside of the bowl are moved towards the cake discharge point by a screw conveyor that has a small clearance to the spinning bowl. The liquid from the sludge flows the other way over a circular outlet weir fixed to one end of the bowl. As the sludge moves along the bowl the acceleration forces cause the solids to be compressed and lose water. The key process parameters are the spinning speed; the rates of feeding and withdrawing sludge; and polyelectrolyte dose.

13.5.7 Filter belt presses

These consist of two looped belts made of filter cloth that are mounted on a series of rollers. Sludge is introduced into the space between the belts and is then carried through the press. This is designed such that the rollers progressively squeeze the sludge allowing the liquid to pass through the cloth and leaving the cake on the cloth from where it is scraped off. The process is controlled by the rate of sludge application, the speed of the belts, and the polyelectrolyte dose.

13.5.8 Vacuum filtration

Vacuum filters may be of either the drum or horizontal type. They operate by applying a vacuum to one side of a filter mat; the vacuum then withdraws water through the filter mat from the sludge being filtered.

The drum type comprises a horizontal cylindrical drum covered in filter cloth supported on a structural frame. The ends of the cylinder are sealed but have a liquid drain and suction pipe passing through. The lower part of the drum sits in a tank of sludge. The drum is rotated while a vacuum is applied to the inside. The drum picks up sludge from the tank and, as it rotates, liquid is drawn out of the sludge. Just before the drum enters the sludge, after a complete revolution, sludge is scraped off the filter cloth. The performance of drum vacuum filters depends on the speed of rotation; the depth of immersion, which affects the thickness of sludge on the drum, and the conditioning of the sludge. Clearly, there is a limitation on the vacuum that can be applied. These filters often perform poorly with gelatinous coagulation sludge but may perform well with sludge from softening. Even when used with softening sludge their performance may deteriorate if polymer is dosed.

A variation on the drum vacuum filter is the horizontal vacuum filter. These use a looped horizontal filter cloth, with sludge introduced onto one end of the horizontal belt.

13.5.9 Lagoons and drying beds

Lagoons are no longer an appropriate way of thickening sludge in the UK, although they may be appropriate in areas of the world where land is cheap and the climate ensures a high evaporation rate. For lagoons to work satisfactorily, a capacity equal to around 200 days of waste sludge production is required. If the quantity of waste produced were 2% of plant throughput then the volume would be four times the daily flow treated. Lagoons are typically 1 m deep. The solids content of the sludge taken out from lagoons is rarely significantly above 10%.

Drying beds are constructed of a level bed of sand resting on an underdrainage system of gravel and drainage pipes. The beds have shallow division walls and a system of pipes or channels to convey the sludge to the beds. It is normal to thicken the sludge before passing it to the drying beds. After drying the sludge is removed and taken for disposal. In the UK, an area of around 60 $m^2/1000\ m^3$/day of plant capacity is appropriate. In hot dry climates this can be significantly reduced and drying beds are often very satisfactory.

A potential alternative to lagoons or drying beds is to use reed beds for dewatering sludge, depending in part on the root system of the reeds extracting the moisture from the sludge. This approach is becoming well-established for sewage sludge but there appears to be no reason why it should not be used for water treatment sludge.

13.5.10 Softening sludge

The lime–soda process produces a mineral sludge, although where surface waters are softened when there may be a significant organic content. Mineral sludge is relatively easy to dewater providing no polymer has been used. The sludge is, however, very thixotropic, and even at high-solids contents may liquefy if disturbed. This makes it more difficult to handle and transport. Softening sludge containing polymer has a more gelatinous nature and this can make it more difficult to dewater. The ideal waste product from an operator's point of view is that produced from pellet reactors. This consists of hard free-draining pellets.

13.6. Disposal

This is normally to a landfill. However, there is increasing interest in using the sludge in ways that involve re-use. Such uses include

- incorporation into construction materials such as brick, cement, or manufactured aggregate
- incorporation into soils or soil improvers
- use in land reclamation or agriculture.

To date there is no clear economically viable option to landfill. This is partly because of regulations covering the classification of waterworks sludge. However, in the future, increased landfill costs and any relaxation in the regulations covering water-treatment sludge may make one or more of the above economic.

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Chapter 14
Private water supplies

14.1. Legal requirements in the UK

As with public supplies, the laws and regulations relating to private supplies are drafted separately for England, Scotland, Wales and Northern Ireland. However, all laws and regulations are derived from and have to comply with EU Directives relating to water supplies. In practice, England and Wales have essentially common wording, but Scotland and Northern Ireland differ. The broad requirement is that all significant private supplies have to comply with the standards for public supplies derived from European legislation. However, small supplies in Scotland do not have to comply with the same standards, although the quality of the water supplied must not constitute a danger to health. In Northern Ireland, the Regulations apply to all private supplies but supplies to single domestic properties do not have to be monitored for quality. In England, supplies to single domestic properties should comply with the same standards as public supplies, but the local authority need not monitor compliance and non-compliance may be ignored if there is no assessed risk to health.

The requirements of the main regulations applying to the United Kingdom are summarised below; however the precise requirements are complex and have not been set out in detail.

14.1.1 England and Wales

Private water supplies in England and Wales are covered by the Private Water Supplies Regulations 2009 and Private Water Supplies (Wales) Regulations 2010. Local authorities are responsible for the assessment of the risks associated with private supplies and monitoring of the quality of supplies. The drinking water standards that are applied to private supplies are the same as those for public supplies, but for the smallest public supplies much more emphasis is placed on risk assessment and risk mitigation rather than very occasional monitoring. The 1998 European Council Directive 98/83/EC allows countries to exempt from the provisions of their national law an individual drinking water supply that provides less than 10 m^3 per day on average or that serves fewer than 50 persons, provided the water is not supplied as part of a commercial or public activity, but this has not been done for England and Wales.

The Regulations require that local authorities carry out a risk assessment of all private supplies at intervals not exceeding five years, other than supplies to a single domestic property. For single domestic properties, risk assessments have to be done only where the owner or occupier requests one.

Private supplies that provide 10 m^3 per day or more on average or that serve premises where water is used for a commercial activity or supplied to public premises have to be regularly sampled and monitored.

Small private supplies must comply with the same water quality standards as public supplies, but the monitoring requirements are less. The local authority has to monitor water quality for five basic parameters: conductivity; enterococci; *E. coli*; pH; and turbidity, plus any other parameters identified by the risk assessment as a potential danger to human health. Supplies to single domestic properties are not required to be monitored unless requested by the owner or occupier.

The Regulations require local authorities to prohibit or restrict the use of a supply where there is a potential danger to health.

14.1.2 Scotland

Private supplies in Scotland are regulated by the 2006 regulations (Scottish SI 2006, No. 209). These adopt a different approach to the English regulations, making use of the exemption allowed by the EU Directive. Thus private supplies that provide greater than 10 m^3 per day on average or that serve more than 50 persons, or that provide water used in any commercial or public activity, are classed as Type A supplies and have to comply with the same quality standards as public supplies. The smaller Type B supplies have to comply with a much reduced suite of parameters, primarily microbiological but also with standards for lead, nitrate, turbidity, and pH: there is also a catch-all requirement that the water should not contain any substance at a level which is a potential risk to human health. Risk assessments are required for Type A supplies only, although the Regulations do also provide for risk assessments of Type B supplies. Responsibility for risk assessments and monitoring lies with local authorities.

14.1.3 Northern Ireland

In Northern Ireland, the Private Water Supplies Regulations (Northern Ireland, 2009) apply to all private supplies. The requirements are similar to those for England and Wales, save that there is no provision or requirement to monitor supplies to single private dwellings, and responsibility for monitoring is with the Department of the Environment.

14.2. Treatment objectives

In principle, private supplies should be treated to produce water of a quality similar to that of public supplies. There is, however, some leeway in Scotland for Type B supplies, and, to a lesser extent, in England and Wales for domestic supplies serving a single property. Whilst accepting this it is still possible to prioritise treatment requirements in a general way.

- The supply has to be microbiologically safe. Microbiological contaminants are the main concern with respect to disease. For old people and infants diseases associated with microbiological contamination can be life threatening.
  A bacterium not normally considered for public supplies but of concern in private supplies is *Campylobacter* which is found in high numbers in the faeces of infected pigs and poultry, and for which the infective dose in contaminated water is relatively low (Scottish Executive, 2006). This organism is not a problem in public supplies since it is easily killed by disinfection.
- Those inorganic contaminants that can cause health problems relatively quickly, notably nitrate which can give rise to blue baby syndrome in infants are a priority. Other inorganic contaminants of particular concern are lead, arsenic, and radionuclides; the latter two being associated with groundwater in some areas, and the former often associated with domestic piping and plumbing.
- Other health-related inorganic and organic chemicals that have health implications in the longer term.
- Aesthetic parameters: colour, turbidity, taste and odour, which in themselves often have no direct health implications but which can make water unacceptable.

Outside of the UK and Europe, the main concerns are the first two types of parameters, and in particular the requirement that the water be microbiologically safe.

It is worth noting that where water that is microbiologically unsatisfactory is supplied to a dwelling, it may well be that it has little effect on the residents, who become acclimatised over time, but can badly affect visitors.

14.3. Risk assessment

For private water supplies, as for all new treatment works, it is important to carry out a risk assessment of the water source to identify the probable contaminants that could be encountered. It would be very imprudent to provide treatment solely on the basis of a limited number of samples that may not reflect the worst quality of water likely to be encountered. Where the source is surface water it is certain that there will be seasonal or climatic variations. Groundwater or spring sources are normally less variable but if shallow may be liable to seasonal or climatic changes in quality, for instance, the quality of a shallow groundwater source can deteriorate after heavy rainfall or at times of high water table when the normal filtering effect of the ground is by-passed. Clearly for a small private supply a major risk assessment is impracticable, but it would be foolish to unknowingly provide treatment that was inadequate for all or part of the time. Examples of risk assessments for private supplies are given in the technical manual for private supplies published by the Scottish Executive (2006) and available on the Drinking Water Inspectorate's website.

14.4. The source

In considering treatment of private supplies, the source is critical. Particular concerns reflect the treatment objectives listed above.

- Pollution from cess pit or septic tank overflows, or from farm or animal waste are of most concern.
- In agricultural areas nitrate can be an issue and in areas where there is granite, groundwater can be high in radioactivity.
- Pesticide contamination is a potential concern in many areas.
- Colour and turbidity can make water unacceptable.

There are also differences between groundwater and surface water. The former usually have low levels of colour and turbidity, and consistent microbiological quality; the latter are much more variable. Groundwater from shallow wells and springs may be of variable quality. Quality issues with groundwater can include high or low pH and high concentrations of iron, manganese, nitrate, chlorinated solvents, or pesticides. Surface waters are more likely to have high levels of colour and turbidity and poor microbiological quality. Quality often deteriorates following heavy rainfall. Surface waters may also have high concentrations of nitrate and pesticides.

In practice, there are often very few options as to where water is sourced, but any available options should be considered, simply moving an intake upstream of a discharge can improve the safety of a supply.

14.5. Treatment

14.5.1 General principles of treatment

For a private water treatment works to be practicable it must be reliable, simple to operate, preferably automatic, affordable and effective. The treatment may either be provided at the point of use, in which case it will normally only treat the water to a single tap, or at the source or at the point of entry to a property. Treatment options for point of use are more limited than for point of entry or upstream options.

Whilst it is entirely practicable to design a one-off water treatment plant for a particular source of water, in practice, this is unusual. In Europe design and procurement of most private water treatment systems will involve the use of package plants from established manufacturers, often with the supplier contracted to maintain the plant. There may be some options for sizing of unit processes to provide bespoke treatment optimised for the water to be treated, but essentially such plants all use off-the-shelf units. In areas of the world where labour is cheap, power supplies unreliable, and it is difficult or expensive to obtain spare parts, treatment might use purpose built small treatment plants using, for example, slow sand filtration.

There are many publications relating to treatment for small water treatment systems (Jackson *et al.*, 2001; Clapham, 2004).

14.5.2 Treatment processes

Table 14.1 lists the water quality issues that are normally encountered with respect to private supplies and the treatment processes that may be employed to address them. It also indicates whether the process is suitable for point-of-use treatment. The parameters of concern are those most commonly encountered. The treatment unit processes vary in practicability for use in private treatment. The most practicable are those that can be employed at point of use, and as such these are usually preferred. The most important treatment process is disinfection, and this is an area where point-of-use treatment can be effective. The processes in the table are briefly considered below, but the most important requirement for a private supply is a good quality source, particularly for supplies to a small number of properties where elaborate treatment is just not practicable. Most of the processes have been considered in earlier chapters, to which reference should also be made.

It is appropriate to note that many of the processes theoretically available for treatment of private supplies are unsuitable for small supplies unless there is a competent operation and maintenance regime in place to ensure the system is operated properly, and, as such, great care should be taken in proposing treatment for private supplies. Some widely available treatment plant, notably some domestic reverse osmosis plant, is designed to treat water of near-potable quality, as a polishing stage and may not be suitable for raw water; this needs to be borne in mind when considering the need for treatment and specifying a process.

14.5.2.1 Aeration

For private supplies, the most likely use of aeration is to be for removal of radon. Aeration is also useful for oxygenation of water containing iron and manganese, assisting in its oxidation to insoluble forms, but must then be followed by a settlement or filtration stage. Aeration is also used for removal of volatile organic chemicals but sources containing volatile organic chemicals are rarely encountered and are not suitable for private supply.

14.5.2.2 Sedimentation

Generally not used for small private supplies that are likely to use filtration for solids removal. Sedimentation may be used for larger supplies where surface water is being treated or in the developing world. It does require regular de-sludging and a means of sludge disposal. To be more effective sedimentation should be preceded by coagulation

Table 14.1 Treatment processes for private supplies

Treatment process[a]	Point of use?	Bacteria	Cysts (oocysts)	Viruses	Algae	Turbidity	Colour	Arsenic	Iron, manganese	Radon	Uranium	Nitrates	Pesticides	Taste and odour
Physical processes														
Aeration	No								*	**				*
Sedimentation	No					*			*					
Pressure filtration	No	*	*	*	*	*			*					
Cartridge filtration	Yes	*	*	*	*	*			*					
Slow sand filtration	No	**	**	**	**	**	*		*					
Membrane treatment	No	**	**	**	**	**	*		*					
Reverse osmosis	No	**	**	**	**	**	**	**	**	*	**	**	**	**
Chemical processes[c]														
Activated carbon	Yes[b]						*			*			**	**
Activated alumina	Yes[b]							**						
Ion exchange	Yes[b]							*			**	**		
Disinfection														
Chlorination	No	**		**	*		*							
UV disinfection	Yes	**	*	**	*									
Ozonation	Yes	**	*	**	**		*						*	*

* Indicates process partially effective or effective but not very practicable; ** indicates process effective and practicable

[a] Processes may need to be combined for effective treatment; [b]These processes can be used as point-of-use but all have some drawbacks;

[c] Term loosely used – activated carbon and activated alumina are adsorption processes

and flocculation, another level of complexity and effectively impracticable for private supplies.

14.5.2.3 Pressure filtration

This is typically used in small automated package plants with sand as the filtration media. The technology is similar to that used for swimming pool filtration, and such plants are readily available and very reliable. As with any sand filter, they require regular backwashing and produce dirty backwash water that has to be disposed of. Pressure filtration combined with point of use disinfection is probably the most practical form of treatment for supplies to more than three or four properties, being relatively effective at solids removal, and when combined with disinfection should provide a safe and acceptable supply. This does, however, depend upon the source being of acceptable quality, as the treatment will not address many contaminants. If a source were assessed as being at risk of having oocysts present then it would be appropriate to also provide point-of-use cartridge filtration. Sand filters should have a treated water storage tank between them and the consumers, to allow them to be sized for average flow and to allow for them being out of service whilst backwashing. Pressure filters can use media other than sand to provide other treatment, notably granular activated carbon for addressing taste and odour and pesticides; however for private supplies this may not be appropriate, as the media requires regular replacement; if such a system is provided it is essential that proper operating and maintenance arrangements are put in place.

14.5.2.4 Cartridge filtration

For very small supplies where pressure filtration is uneconomic or impracticable, point-of-use cartridge filtration often represents the only practical method of solids removal. Cartridge filtration can remove very small particles, including oocysts from water. The drawback is that all solids removed are retained within the filter cartridge, which needs replacing once it becomes clogged. Thus, cartridge filtration may only be practicable with water low in solids. Ceramic filters are also a form of cartridge filters generally used for polishing of high-quality water. Where the aim is protection against oocysts, a one micron filter is ideal, but is only practicable for supplies with a very low solids content. A more realistic approach would be a two-stage filter, a 15 μm filter followed by a 2 μm filter; this would provide a longer life for the expensive fine filter and would be expected to offer a high degree of protection.

14.5.2.5 Membrane filtration and reverse osmosis

These are considered together, as conceptually they are identical. Membrane treatment is taken to refer to ultrafiltration. Whilst there are many small reverse osmosis plants available, many of these are designed to treat water of potable standard and will require to be used in conjunction with suitable pre-treatment if used for treating raw water. Except where the raw water is of very good quality, reverse osmosis is not very practicable for private supplies as it requires pre- and post-treatment and represents a high technology process. The attraction is that the water produced is likely to be free of harmful concentrations of all contaminants, but the disadvantages include the need for regular and specialised maintenance, the use of chemicals for regular cleaning, and the production of waste water.

Reverse osmosis is widely used on board ships and increasingly in emergency situations, but it should be recommended with caution for private supplies.

Membrane filtration, using membranes with a larger pore size than for reverse osmosis, provides a good degree of treatment, removing essentially all particulate matter from water. Whilst operating and maintaining membrane plants is simpler than reverse osmosis plants, it is still a technologically advanced process that requires proper maintenance. Membrane filtration is more suitable than reverse osmosis for private supplies, and is of particular applicability where there is a risk of oocysts being present in the raw water. Membrane treatment can handle a poorer quality feed water than reverse osmosis, which normally requires a minimum of cartridge filtration prior to the main process. It is possible that the development of ceramic membranes, which are more robust with more tolerance of poor O&M, will make membrane filtration more practicable for private supplies but this remains to be seen.

14.5.2.6 Activated carbon

This process is suitable for the removal of organic chemicals present in water: pesticides, colour, or other organic chemicals. It can also be used to remove radon. It is normally used in granular form (GAC), either in a point-of-use cartridge or in larger adsorbers. In addition to its prime purpose of adsorption, the carbon also acts as a physical filter, and it is therefore necessary to be able to periodically clean the filtered material from the carbon, or replace the cartridge. The effectiveness of the carbon in removing chemicals depends upon the contact time between the water and the carbon. This means that cartridges with short retention times are ineffective.

It is common to combine filtration and adsorption by using suitably graded activated carbon as the filtering media in place of sand. In order to obtain a reasonable contact time, of more than 5 min, lightly loaded filters are required.

Whilst it is relatively straightforward to use, because of the need to regularly replace the GAC it is not normally appropriate to use GAC for small private supplies if it is to be used to remove chemicals of health concern, as if the GAC is not replaced when necessary, treatment is lost.

14.5.2.7 Activated alumina

This is used to remove arsenic and fluoride. It is similar to GAC in that it can be used either as a point-of-use system as a cartridge or, in theory, as a media in a filtration system. It also requires replacement when exhausted. In practice, its most likely use would be as a point-of-use filter. As with GAC great care should be taken before using activated alumina where there are health concerns over the quality of the source water.

14.5.2.8 Ion exchange

This process is widely used in domestic properties for water softening. Thus there are a large number of suppliers of this process. The system can be optimised for removal of specific ions by using the appropriate media. The process requires regular regeneration, usually using sodium chloride, and produces a waste that has to be disposed of. Ion exchange is most useful for treatment of uranium and nitrate.

14.5.2.9 Chlorination

Chlorination is not appropriate for systems serving a very small population but is suitable for larger private supplies. The most suitable method is normally either dosing sodium hypochlorite solution or using solid calcium hypochlorite that is made up into a solution in a dissolving tank. Clearly, regular monitoring and operational input is required, making chlorination normally unsuitable for small supplies.

14.5.2.10 UV disinfection

This is the most appropriate point-of-use disinfection treatment. UV disinfection can be effective at inactivating oocysts if an appropriate dose is applied. As with most of the processes described above, UV disinfection also requires proper operation and maintenance, with regular lamp cleaning and replacement. A key requirement is, of course, a reliable power supply.

14.5.2.11 Ozonation

Ozone is a powerful oxidant and disinfectant but it is not normally used for private supplies. Despite the benefits it offers, it requires the use of complicated plant and is often therefore unsuitable for private supplies. This is due to the need for professional operation and maintenance, without which the system will fail.

14.6. Conclusions

There is no doubt that the key to a safe private supply is a high-quality water source. Without such a source there are, in theory, a wide range of treatment options; in practice, however, many of the possible processes are unsuited to small supplies, particularly where there are limited resources for the operation and maintenance of a plant. Where treatment is provided it is essential that due attention is paid to the operational needs of the plant supplied, and that these be taken into account when specifying the process to be used, and considering the long-term costs of the options.

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Chapter 15
Water safety plans

15.1. What is a water safety plan?

A water safety plan (WSP) represents a system-wide approach to ensuring that the quality of water delivered to consumers is of good and consistent quality. To do this it is necessary to consider the source of the water, how it is treated, and the storage and distribution of the treated water. A WSP is based on a comprehensive assessment of risks of the factors that could adversely affect the quality of the water delivered to consumers, and sets out how risk factors are to be monitored and managed. ‘Water Safety Plan’ is the term most widely used in Europe but elsewhere the term ‘Risk Management Plan’ is often used.

A water safety plan is a location-specific assessment of a water supply system, from the source, or sources, of the raw water through to the points of delivery, considering risks and hazards, means to address and monitor the hazards, and procedures for managing and operating the system, under both normal and exceptional circumstances. It comprises of the following.

- A description of the system, from catchment through to point of supply.
- Identification and quantification of current and future risks to the quality of the water across the system and any risks derived from the quality of the water.
- Actions to be taken to manage risks and as appropriate to develop appropriate monitoring and mitigation measures. These could include actions relating to: system operation under normal conditions; monitoring by appropriate sampling and analyses and/or continuous monitoring; capital investment to amend or improve the system; the definition of trigger levels to initiate changes to system operation; defining how to react to emergencies and exceptional conditions.
- The approach to be taken to regularly review and revise the WSP.

15.2. Why water safety plans?

As discussed in Chapter 2, there has been an explosion in quality standards for potable water, with standards or guideline values set for well over 100 chemicals or other parameters. These range from purely aesthetic parameters, such as colour, through parameters which are harmful when ingested over a long period, such as carcinogens or heavy metals, to parameters which indicate an immediate potential impact on health, such as the presence of *E. coli*, indicating potential faecal contamination.

The traditional approach to ensuring that water was of acceptable quality was to carry out regular sampling and analyses of water to confirm the water complied with all the relevant standards. It is inappropriate to consider all legal standards or values equally, and thus it is normal to specify the frequency of sampling depending upon the health or other importance of the various parameters, and the assessed likelihood of a parameter being a concern. The sampling could be specified for the point of supplying water or at the point of use, or both, depending upon the parameter being analysed. The frequency of sampling would normally also depend upon the size of the system. UK practice is to specify different frequencies of sampling, differentiating between check and audit monitoring, and reduced and standard frequencies of sampling. The sampling and analysis covers both the treatment works and the water in distribution. In the UK, monitoring of water quality at intakes has been limited, mainly depending on automatic on-line monitors.

Overall, there is a very complex system of sampling and analyses, and in the UK there are very few samples that fail to comply with the standards. As additional chemicals are identified as being potentially present and a health risk in water the number of standards and analyses is likely to increase, and most samples will continue to comply with the relevant standards. However, whilst the overall quality of water supplied in the UK is very good, there are still occasional instances of water quality incidents leading to the supply of unwholesome water, and it is clear that there are issues other than those addressed by the sampling and analyses required by the regulations.

The problem with water quality monitoring is that it is retrospective. It informs as to the past quality of water but is often unable to detect incidents in time to prevent possible public health impacts. By the time a failure of a parameter that requires laboratory analysis has been detected, it is likely that the affected water will already be in the distribution system. Even where faecal pollution is suspected due to *E. coli* being found, where the analysis involved is simple, reliable and quick, there is a significant time lag between sampling and obtaining confirmation of there being a problem. For some other parameters, for example, arsenic or pesticides, a failure to meet the standard does not imply any immediate significant risk, and providing appropriate action is taken to address the issue, there may be no justification for taking a source out of supply. To ensure water sampling and analyses better protect public health it is necessary to shift some of the focus away from the treatment works to further up the supply chain, where the risks originate.

Another issue is that where a water quality incident occurs it is nearly always the case that investigations reveal that similar incidents have or could have occurred previously. (In this respect, water supply is similar to aviation and to most other areas of human activity where after a significant harmful event occurs, the subsequent investigations almost always reveal a history of near-misses with the same causes.) Following a water quality incident it is often clear that it would not have been practicable to carry out sufficient monitoring to allow the impending incident to be identified before it occurred but that there were indications that an incident was, or could be, developing but the warning signs were not detected or heeded.

Water Safety Plans (WSPs) have therefore been proposed as a means of assessing the risks and consequences of incidents occurring and developing approaches to minimise the probability of poor-quality water being supplied. WSPs will identify the range of failures that could occur and the most appropriate way of addressing them, either by system improvements or modifications, or by improved monitoring relevant to the risks identified.

In Europe, there is a general consensus amongst those responsible for regulating water supply that the approach to ensuring the supply of consistently high-quality water should involve the use of WSPs. There should be a link between the WSP, the treatment required, and the frequency of sampling and analyses for a system. If this were done it would lead to a reduction in the risk of poor quality water being supplied whilst minimising capital and operating costs by: (i) ensuring that the standard of treatment is related to level of risk and (ii) focusing analyses on those parameters which are most appropriate to monitor the particular hazards and risks in a system. In the UK, the Drinking Water Inspectorate's information letter 02/2008 (DWI, 2008) sets out a requirement for English and Welsh water companies to prepare risk assessments for all systems, using WHO's Water Safety Plan methodology, representing the introduction of WSPs to England and Wales. In addition, the English and Welsh, Scottish, and Northern Irish Regulations relating to private water supplies all require risk assessments of private supplies, although there are some differences in the requirements. More generally there is a movement towards requiring risk assessments across much of the world, driven to a great extent by WHO's guidance (2006).

15.3. Data needs

In order to prepare a WSP, it is necessary to collect data from a wide range of sources, relating to: the standards required for the water as delivered; the catchment and the sources of water; the treatment provided including a full understanding of the treatment provided under all conditions/circumstances; and the distribution system from the works to the points of delivery to consumers. In addition, wider issues need to be considered, including the procedures for taking and analysing water samples, communications within the water supply organisation and between the supplier and other relevant bodies. It is not proposed to list all data needs but typically the data that should be collected include data from: water sources, water treatment and the water distribution system.

15.3.1 Water sources

Data from water sources should include the following.

- Details of the catchments from which the water at the source is derived. For groundwater, this should include geohydrological information as well as details of surface activities around the extraction point. For surface water catchments the information should include details of land use including agricultural and industrial activities that could affect water quality, and details on discharges of treated, or untreated, waste discharges. For underground sources, details on historic activities may also be significant and should be collected. The aim is to be able to identify potential contaminants that could enter the water.
- Hydrological data relating to flows and data on groundwater levels. Data on any requirements to maintain minimum flow in rivers. The aim is to assess the likelihood of there being insufficient water available.
- Details of current monitoring of raw water in the aquifer or surface water source. The aim is to understand the effectiveness of the current monitoring compared to the risks, to define if possible the normal range of raw water quality, and to identify any shortcomings in the current monitoring regime.

15.3.2 Water treatment

Data should include the following.

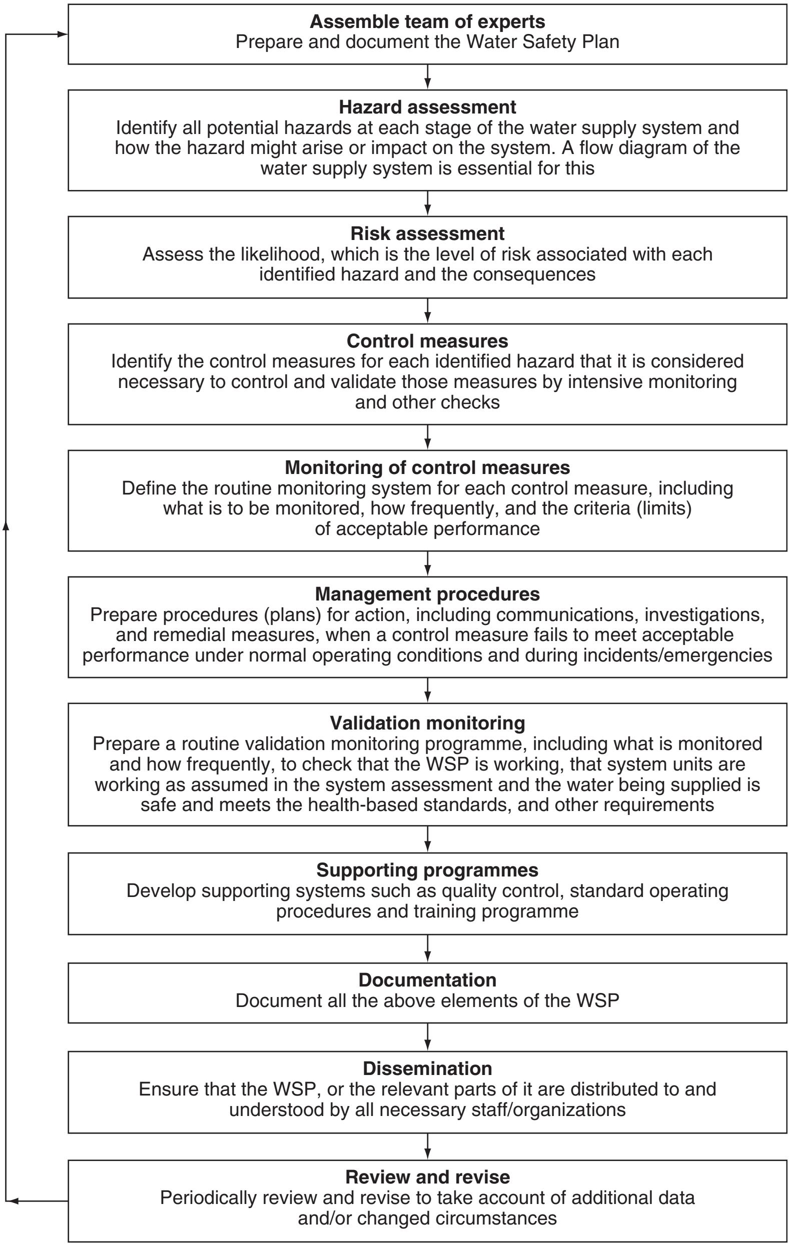
- Details of the water treatment process and processes used. This should include details of potential routes by-passing treatment; treatment and disposal of sludges and wastes produced within the treatment process; and an understanding of normal and abnormal operating regimes for the treatment works. The aim is to understand the degree of treatment provided both normally and at other times, and the potential for water which has by-passed all or part of the normal treatment to enter distribution. Linked with the assessment of raw water quality and risks, the aim would be to identify any potential process deficiencies relative to the potential raw water quality.
- Details of the monitoring and control systems used within the treatment works together with the results of analyses and water quality monitoring of both treated and partially treated water. The aim would be to understand how the works performs and to identify any processes that do not always perform to the standard required to guarantee safe performance of the works.
- Details of the existing standard and non-standard operating procedures used at the treatment works. The aim is to understand how the works is operated, how exceptional events are defined, identified, and dealt with.

15.3.3 Water distribution system

Data should include the following.

- Details of the distribution system including trunk mains, service reservoirs, additional treatment within distribution, and of the network used to deliver water to customers from trunk mains down to service pipes. This should include details

Figure 15.1 Typical approach towards the preparation of a water safety plan



Assemble team of experts
Prepare and document the Water Safety Plan

Hazard assessment
Identify all potential hazards at each stage of the water supply system and how the hazard might arise or impact on the system. A flow diagram of the water supply system is essential for this

Risk assessment
Assess the likelihood, which is the level of risk associated with each identified hazard and the consequences

Control measures
Identify the control measures for each identified hazard that it is considered necessary to control and validate those measures by intensive monitoring and other checks

Monitoring of control measures
Define the routine monitoring system for each control measure, including what is to be monitored, how frequently, and the criteria (limits) of acceptable performance

Management procedures
Prepare procedures (plans) for action, including communications, investigations, and remedial measures, when a control measure fails to meet acceptable performance under normal operating conditions and during incidents/emergencies

Validation monitoring
Prepare a routine validation monitoring programme, including what is monitored and how frequently, to check that the WSP is working, that system units are working as assumed in the system assessment and the water being supplied is safe and meets the health-based standards, and other requirements

Supporting programmes
Develop supporting systems such as quality control, standard operating procedures and training programme

Documentation
Document all the above elements of the WSP

Dissemination
Ensure that the WSP, or the relevant parts of it are distributed to and understood by all necessary staff/organizations

Review and revise
Periodically review and revise to take account of additional data and/or changed circumstances

of the type, age and condition of pipes, assessments of the service reservoirs, booster pump stations, and any additional treatment in the distribution system. The aim is to identify both potential sources of external contamination of the water and the scope for water quality changes within the network.

- Results from sampling and analyses of water samples from the distribution network, and from any monitoring systems used. The aim is to understand the changes to water quality within the distribution and assess the risks of aesthetically unacceptable or poor-quality water being supplied. Also, this should allow the normal range of water quality to be defined and understood.
- Serviceability data from the network including data on hours of supply, unplanned interruptions to supply, frequency of pipe bursts, vandalism, and data on any other issue which could impact on water quality.

15.4. Preparation of a water safety plan

Having obtained all available, or sufficient data, the plan itself can be prepared.

Figure 15.1 is based upon DWI's guidance on the preparation of WSPs and shows a typical approach.

Figure 15.2 shows how WSPs fit within a water supply framework; it is based on WHO's framework.

Figure 15.2 Water safety plan in context of overall system management

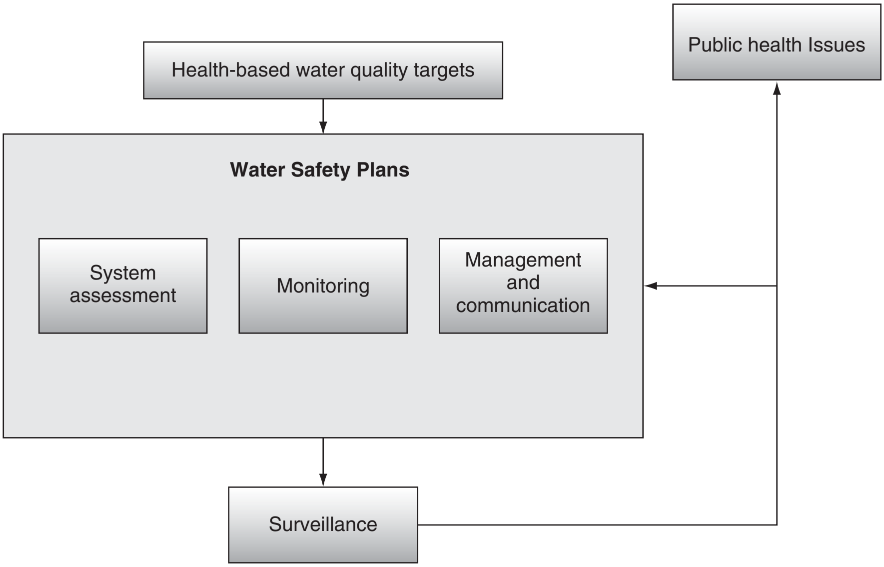


Table 15.1 Framework for a water safety plan

Component	Requirement	Comment
Setting health-based standards	Standards should be based on an evaluation of health concerns and set at tolerable levels for the community. They should be risk-based	In many countries, the standards are mainly those set by national legislation. Otherwise WHO guidelines may be relevant
System assessment	Assess the whole system from source to points of supply, to assess risk that the water could fail to meet the health-based standards	
Operational monitoring	Define the critical control points and monitor them	The monitoring should be at multiple points, from source to supply points, using appropriate monitoring criteria, preferably on-line to ensure speedy response to any out-of-specification parameters
Management plans	Establish plans for the operation and management of the system. Define when plans to be reviewed and revised	The plans should cover both normal and abnormal operation, and define lines of communication and responsibility
Surveillance	Establish a system of independent surveillance to verify system is operating effectively	

Table 15.1 sets out the framework for a WSP.

15.5. Relevance to developing countries

Whilst the impetus for the development of WSPs has largely come from Europe, and it is in developed countries that there will be the largest quantity of data to populate WSPs, they are also a valuable resource in developing countries. In a developing country, for a WSP to be of any practical use, the main difference must be that the health-based targets should be drafted and agreed such that they represent realistic targets. Even if the health-based standards are less stringent than used in Europe, the data and approach should be the same, although the plan will almost certainly be far simpler, involving significantly less monitoring. Where there is limited capacity to monitor water quality, the importance of surveillance of the integrity of the water supply system is much greater.

A further step could be to assess the costs, both capital and operating, developed from the WSP and evaluate these in the context of the likely benefits derived from a better quality and more reliable water supply. The use of such a cost–benefit approach should help in supporting and attracting finance to water supply systems in the developing world.

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Chapter 16
Sizing of water treatment plants

16.1. Introduction

The quantity of water required to be produced by a treatment works is dependent on the demand for water and how this is met. The capacity depends upon many factors: these include daily fluctuations in use, seasonal variations, long-term trends, and the need to cater for emergency situations. This chapter sets out some of the factors affecting demand and how the capacity of the water treatment works should be considered in relation to them.

16.2. Daily demand variation

Demand for water fluctuates during the day. For domestic demand there are generally peaks in the morning as people get up, wash, breakfast and go to work. There is then generally a lower peak in the afternoon and early evening as they return home. In countries or areas with an appreciable garden water use component there can be a significant early evening increase in water use: in the UK, this is a seasonal factor.

In water supply zones, where there is a large leakage component, the relative variation in demand can be dampened significantly.

It is seldom sensible to try to make the water treatment works process throughputs follow the demand pattern. It is much better to operate a works at a largely constant throughput and to have the variations in demand be taken up by water in storage. Although in theory each water resource zone is different and would need analysing individually, traditionally, a figure of about 6 to 8 hours storage was considered sufficient to provide for diurnal variations in demand.

There is also the need to ensure that there is sufficient water to cover for an emergency situation such as a trunk main burst or a breakdown in the water treatment works itself. Thus, an old rule of thumb was to provide 24 hours of reserve storage in the system. With modern instrumentation identifying bursts and failures more quickly and passing the information to permanently manned control rooms, such an amount of storage can probably be reduced but this needs to be carefully worked out and the basis of the calculation recorded.

It would be normal for part of the reserve storage to be in a clear water tank at the outlet from the treatment works. This has the advantage of having part of the storage available to all the water supply zones served by a works, and it also dampens any variations of quality of the water produced by the treatment works. However, it is also normally essential to have some storage at appropriate strategic points in the distribution system.

16.3. Seasonal demand variation

Almost all water supply zones have a variation of water use between seasons of the year. For instance, in summer in Europe there is an increase in the amount of garden watering. At other times, there can be increases in demand due to pipe bursts, for example, due to frost and associated pipe bursts or due to clay shrinkage leading to movement and burst of the pipe. The latter can be particularly prevalent for old pipes with old-fashioned lead run joints in clay conditions such as can be found in London.

Design of the water treatment works has to take account of the peak seasonal demand that is likely.

16.4. Long-term demand projection

No water supply zone will continue to require the same amount of water. Historically, the demand would have been calculated on the basis of increasing population and increasing per capita demand, based on historic projections. There would then have been an assessment of how much forward growth it would have been appropriate to allow for. This might well have been some 20 to 30 years, dependent significantly on the rate of growth.

In recent times, population movement, immigration, emigration, and fertility have become much more changeable. Additionally, in Europe there is now a greater appreciation of the value of water. Even in Britain water metering of domestic properties is becoming widespread, thus curtailing water waste. Water-using appliances are also becoming more water efficient. Thus, in England, the quantity of water delivered is now relatively stable. In much of western Europe, per capita consumption has dropped appreciably. In Europe, it is still necessary to project demand forward in order to check a water treatment works would still be able to meet future demands, but in practice this now often shows no need to increase the capacity of a works.

In developing nations, rising standards of living are still driving up per capita consumptions and this, combined with population growth, means there is often a need either to build spare capacity into a works, or to allow for adding further treatment capacity in the future.

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Chapter 17
Water demand and use

17.1. Introduction

This chapter presents an introduction to water demand and use. The chapter is focused on the situation in England and Wales, as the position in other parts of Great Britain is generally similar. The principles outlined are however universal, although water demands may be very different, particularly in tropical countries. Demand is considered in broad terms with the aim of identifying the major components and giving an indication of their magnitude, and how demand varies over a year and the strategies available to control it. Unaccounted-for-water is discussed and the key issues relating to it are discussed.

17.2. Water demand

Water demand and use is a subject that has become very complex over the past few years. This reflects its increasing importance and the amount of effort spent in studying the subject.

In the UK, until 1988, water-supply systems were generally publicly owned, domestic water supplies in the UK were unmetered, and it was widely accepted that increased incomes and improved standards of living automatically meant increased water use. Demand was unmanaged and increased at a steady rate. It was assumed that countries with low water usage would in the fullness of time reach North American levels of prosperity and water usage. Water-supply systems were designed with generous amounts of spare capacity.

Things are very different now. This has come about as a result of public resistance to new reservoirs, of increasing concern over low flows in rivers and streams as a consequence of perceived over-abstraction, and the unwillingness of the water-industry regulators to sanction increases in the price of water to fund the cost of new water-resources works. Now, water suppliers have duties to manage demand and to continuously monitor water resources and demands.

In the UK, it is difficult to find new usable water resources in many parts of the country. English and Welsh water companies are required to manage demand through the promotion of efficient water use and are required by law (Water Industry Act 1991

(1991) as amended by Water Act 2003 (2003)) to produce a water resources management plan every five years. The plan has to set out how a water company intends to maintain the balance between supply and demand for water over the subsequent 25 years. In the past, water companies in England and Wales have tended to limit transfers of water between companies or sharing of water resources. This can lead to inefficiencies and duplication of water resources, and Ofwat are now actively encouraging companies to work together to develop the most economic solutions to meeting demands. In many areas, there is pressure to reduce abstractions, most notably where groundwater abstraction is perceived either to have led to formerly perennial rivers to become seasonal, or to have led to occasionally unacceptably low flows in perennial rivers. The Environment Agency is required by the Habitats Directive and the Water Framework Directive to study water systems and to identify where reductions in abstractions are required to meet sustainability criteria or to maintain or restore the ecological status of the water environment.

The normal approach to water resources planning now is to follow the twin track approach using both demand management techniques and water resource development.

Partly as a result of the change in attitudes towards meeting demand, there has also been a change in attitudes towards paying for water. In the UK, it is now increasingly accepted that water should normally be paid for on the basis of metered use. This is supported by the EU Water Framework Directive which requires that ‘Member States shall ensure by 2010 that water pricing policies provide adequate incentives for users to use water resources efficiently and thereby contribute to the environmental policies of this Directive’. Additionally, a 2009 government sponsored review of how water and sewerage services should be paid for recommended ‘the future charging system should generally be based on the volume of water used and therefore on a metered system.’ (Walker, 2009) All new houses in England and Wales now have meters. Most non-domestic supplies are now metered. The Water Industry Act 1999 empowered water companies to compulsorily meter customers with high non-essential use of water, such as those with a swimming pool or garden sprinkler. As well as compulsory metering of potential high users of water, there is also a voluntary move towards metering. Many householders in England and Wales still pay for water by a formula based on the rateable value of their property. Occupants of houses of higher rateable value who pay a high flat rate but use a relatively small quantity of water would therefore be better off with a metered supply. Such people are increasingly opting for a metered supply. In areas classified by the Environment Agency as ‘seriously water stressed’, most of southern and eastern England, water companies can apply to meter compulsorily and this is now happening. Once a house is metered, there will be much greater incentive to conserve water and, when buying new appliances, to choose those with lower water usage.

In England, the proportion of metered domestic properties is expected to rise from around 35% in 2010, to about 80% in 2030. One result of increased metering is that there is increasingly accurate information on water consumption, and on water losses. This information enables demands to be predicted more accurately. It will in the future also enable water companies to implement selective tariffs such as rising block or seasonal, thereby encouraging consumers to save water during periods of potential shortage. In parallel with the economic pressures associated with metering of existing properties, the Code for Sustainable Homes (UK Department for Communities and Local Government, 2006) sets minimum standards for water efficiency for different sustainability ratings, ranging from a water consumption of 120 l/person/day for a one-star rating down to 80 l/person/day for a six-star rating, significantly lower than current rates in existing properties.

17.3. Components of demand

In practical terms, the demand of a water system is the quantity of water that has to be put into the system to meet the rate at which water is withdrawn. There are five main components to this demand.

- Losses in the trunk main and distribution system, including losses from service reservoirs.
- Losses in the customers' supply pipes.
- Domestic use – including garden and other external uses.
- Industrial and commercial use, including offices, hospitals, and government.
- Operational use by the water supplier.

The first two components are closely pressure related but the latter three are less dependent on the pressure within the water-distribution system.

17.4. Losses from water systems

All water-distribution systems 'lose' water. The most obvious losses of water are through leaks or bursts. However, from the supplier's point of view losses cover all water for which no payment is received, such as operational use and water taken illegally.

Leakage is water that is physically lost from the water transmission and distribution system. It is simple to define, but surprisingly difficult to measure and compare. In England and Wales, water companies have to report to Ofwat total leakage including supply pipe losses on the customers' properties. Ofwat's guidance was that leakage should be calculated from 16 inputs, a measure of the complexity of the subject. More simply, leakage is a function of five main factors (Lambert *et al.*, 1999).

- The length and condition of the distribution mains.
- The number of customer connections.
- System pressure.
- The location of customer meters.
- The number of hours of water supply.

Leakage is actually a rate of loss of water. However, giving an absolute value to leakage is of no use whatsoever as a comparator of a performance indicator, except for a particular system. To know that a particular town is losing 100 m^3/h in leakage offers very little illumination on the performance of the town's distribution system. The headline way of expressing leakage is often to express it as a percentage of the water put into the distribution system. This is straightforward and readily understood but again not very useful. It takes little account of the five factors listed above that affect leakage. For example, consider two cities one of which has a distribution system in good condition and which provides a continuous supply of water at high-distribution pressure; and the other which supplies water through a system in poor condition for only a few hours a day at very low pressure. The first city may well lose a similar percentage of the water entering supply as the second city, indicating similar performance. However, consumers in the two cities would see the performance of the systems very differently.

Also, an urban system will have a high number of consumers in a small area served by a relatively short length of distribution main. However, a rural system will have a long length of main per customer connection. If the condition of the mains and the house connections are the same for both the city and the rural case, and the systems operate at the same pressure, the rural system will inevitably have a much higher absolute level of leakage, and thus a worse rate when expressed as a percentage of water supplied.

Best international practice (Lambert *et al.*, 1999) for measuring leakage is to introduce three key concepts.

- Leakage is best expressed as litres/service connection/day, when the system is pressurised. This, however, takes no account of system pressure, length of main, hours of supply, or location of meters.
- There is a minimum unavoidable leakage rate for a system, the base level of leakage. This accepts that leakage is always with us, even in the best-constructed and -maintained systems. The minimum unavoidable leakage can be calculated for a particular system based on mains length, system pressure, number of connections, hours of supply, and position of water meters. The calculation has to be done using appropriate factors for the various leakage components. The unavoidable leakage can also be expressed in litres/service connection/day, when the system is pressurised.
- By dividing actual leakage rates with the minimum unavoidable leakage rate, the relative performance of a system can be defined. A value of 'one' indicates a system operating at the minimum attainable leakage rate, whereas a value of 'two' indicates it would be technically possible to halve leakage. Well-managed systems have a ratio of between 1.2 and 1.8 (Lambert *et al.*, 1999).

This, of course, is a fine theoretical approach to leakage. In reality, it is expensive to maintain leakage at low levels. In practice, each system will have an economic level of leakage, at which the costs of maintaining leakage at that particular level are balanced by the costs of supplying the additional water that leaks from the system. Thus, a supplier that has a low-cost (and abundant) supply of water would normally have a higher economic level of leakage than a supplier that has a high-cost (or limited) supply of water.

Water suppliers often have more practical interests other than leakage alone. A water supplier is often more interested in 'non-revenue' water. Clearly, to maximise revenue, a water supplier wants as much water as possible to be 'revenue' water, water for which the supplier receives payment. Where a system is fully metered the water meters record revenue water and non-revenue water is the balance of the water put into supply. In systems that are not fully metered the calculations are more complex. Revenue water is the quantity of water that is measured at the point of delivery to a customer, plus non-metered water legitimately used by a customer. 'Non-revenue' water includes leakage in the distribution system, leakage in unmetered customers' supply pipes, meter under-registration, and water taken from the system and used without payment – which could be illegal use, operational use such as mains flushing, or fire-fighting. Where not all customers are metered (such as the UK), a key challenge is to separate physical leakage from other elements of non-revenue water. This requires suppliers to estimate the un-metered per capita consumption (pcc) by means of a monitoring study of a discrete area. Total water use can be monitored within a defined area and the population can be estimated from census data or from household questionnaires. From this data, together with knowledge of metered consumption and minimum night flows, the leakage and un-metered pcc will be estimated.

17.5. Measurement of leakage

Leakage measurement developed greatly over the 1990s. At the beginning of the period, leakage was normally measured by isolating a water-supply zone supplied by a service reservoir at night. Water entering the zone was then measured as accurately as possible over a period of a few hours in the early hours of the morning, often by measuring the rate of fall of water level in a water tower or service reservoir. This gave the total volume supplied over the period of the test. The volume used by large metered users would then be deducted from this total, based on meter readings before and after the test. The quantity of water assumed to be used for legitimate domestic purposes would then be deducted, leaving a quantity of water assumed to be leakage. The most obvious problem with this method is that the figure assumed for legitimate night use was often the most important and critical item. A small change in this figure would greatly affect the apparent leakage quantity. The figure generally used for legitimate night use was originally 6 l/property/h, based on research carried out in the 1980s. However, this was reduced to a suggested figure of 1.7 l/property/h in the 'Managing Leakage' document (WRc, 1994) and currently a figure of around 2 l/property/h is widely used in the UK water industry. This reduction in the allowance for legitimate night use from 6 to 2 l/property/h would have greatly increased the reported leakage rate in any zone and is one of the factors that mean that although there is generally good and consistent data on leakage back to the late 1990s it is not normally possible to make use of earlier reported leakage rates.

Nowadays, in the UK it is usual to use accurate on-line meters and automatic recording and comparison of night flows using telemetry systems to monitor actual instantaneous flow rates. Large water-supply zones have been divided into smaller district meter areas (DMAs). This has been done partly to avoid large pressure variations in zones, and partly to allow leakage rates to be more accurately monitored. A DMA will have a limited number of feeds, each of which will have on-line flow measurement. This allows monitoring of minimum night flows and permits the supplier to detect any sudden increase, perhaps due to a burst pipe, and any longer-term increase, indicative of an increasing number of smaller leaks. DMAs typically serve between 1000 and 3000 properties. This size range has been derived from the need to establish the smallest number of zones, to minimise costs, while maintaining the ability to detect pipe bursts. In a large zone, smaller pipe bursts will not be quickly detected.

17.6. Non-household use

In the UK, most non-households are metered, with some of the larger users now continuously monitored with loggers. Projection of non-household use is not easy. Conventionally, it has been done using an econometric model relating manufacturing consumption to output statistics and service sector consumption to employment, in each case utilising a small water efficiency factor. However, for most water companies, this has not been a reliable model, the actual increase in non-household use often being lower than that projected. This could be because of the economic situation, increasing real cost of water, and/or greater awareness of the need for water efficiency. It also appears that use in non-households is often inefficient, and not very responsive to financial incentives to reduce use.

17.7. Peak flows

Demands are normally quoted as specified absolute figures. However, domestic demands vary throughout the year, mainly dependent on temperature and rainfall. In the summer, there is more water use in houses for bathing, which is dependent on temperature. There is also more external use of water in the summer, mainly for garden watering, and this is very sensitive to rainfall. The nature of an area affects the size of the seasonal variation. Seaside resorts have higher peak to average demands than city areas. However, there may also be very high flows in the winter, normally during extended cold spells. These flows are associated with a high rate of winter pipe bursts, particularly in areas with clay soils.

Table 17.1 gives some typical values of some of the numbers used in demand forecasting in England and Wales. The numbers are not absolute but are indicative only. Of

Table 17.1 Typical values of key parameters used in demand forecasting

Parameter	Typical value
Domestic use	150 l/head/day (metered supply)
Hospital use	300–500 l/patient/day
Office use	70 l/staff/day
Hotel use	400–500 l/guest/day
Average day peak week demand	120–140% of annual average demand
Peak daily demand	140–160% of annual average demand
Leakage (typical range for well-managed large system)	15–25% of water put into supply
Leakage (typical range for well-managed system)	0.8–1.2 l/connection/m of system pressure/day (derived from Lambert *et al.*, 1999)

particular note are the typical ratios of peak flows to average flows. It is usual to size the water-treatment works to supply the average day peak week demand. This is insufficient to meet the peak daily demand, and the difference between the two is provided by water stored in service reservoirs. Thus, treatment works are sized based on the average daily flow in the peak week, service reservoirs on peak day demands, and distribution systems on maximum instantaneous flows.

For comparison, some values presented at a 2012 conference on actual domestic water usage by one of the authors are set out in Table 17.2 below. Caution should be used in using countrywide figures for water usage, as there are always variations dependant on climate and standard of living, and in reality there are no accurate hard and fast figures. Per capita consumption in the UK is relatively high compared to other European countries, possibly because fewer supplies are metered, but well below Australian and North American levels.

Table 17.2 Per capita domestic water demands

Location	Recently quoted per capita domestic water consumption: litres/head/day
England	145
Denmark	131
Finland	115
Netherlands	127
Germany	115
Zaragosa, Spain	96
Belgium	107
Sydney (2000)	250

17.8. Demand management

In the UK, there is increasing consideration of demand management. This has been driven partly by legislation, and partly by pragmatic need. Given that in practice there are limited water resources available, how does one manage demand such that it can be met? Demand management requires a comprehensive knowledge of all issues relating to water use. Factors include cost, the consumption of water-using domestic appliances, the condition of the distribution system, social changes, and climate and weather.

Suppliers are therefore adopting a variety of techniques to manage (limit) demand.

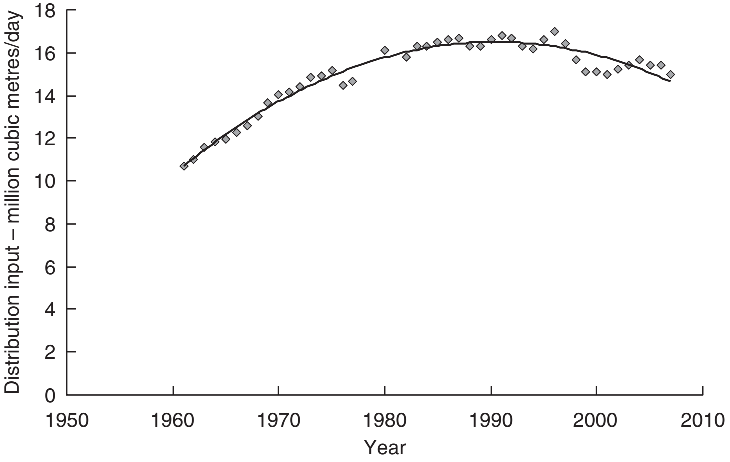
- Tariff – tariffs can be designed to manage demand. For instance, it is possible to have rising block tariffs such that those who use little water have a low unit charge rate and those who use much pay much more for the extra water. It is also possible to have seasonal tariffs such that water used during times when the water resources are not stressed, normally during the winter, are charged less; whereas water used when the water resources are stressed, during the summer, are charged at a much higher rate. Variable tariffs are not currently used in the UK but they will become more practical as meter penetration increases and remote meter reading is introduced.
- Metering of households with non-domestic use – the use of garden hoses and the filling of swimming pools are classified as non-domestic uses. The metering of consumers who are considered discretionary high users is required in England and Wales by the Water Industry Act 1999 (1999).
- Publicity and education – aimed at achieving greater awareness of the value and importance of water and to encourage lower use and less wastage.
- Selective restrictions – these include such measures as hosepipe and sprinkler bans but are only normally used during droughts.
- Regulation of water using appliances is by means of the 1999 Water Bylaws of England and Wales (Water Supply (Water Fittings) Regulations SI No.1148, 1999) that cover appliances such as WCs, washing machines, and dishwashers. As an instance, the size of the WC cistern has been reduced from 9 to 6 l. Many new cisterns are dual flush 6/4 l size, whilst cisterns of 4/2.6 l are now being installed and a 1.5 l system has passed its trials at the WRc.
- Lower use fittings – as well as more efficient appliances, significant reductions in water usage could be made by installing spray head taps instead of conventional ones, showers instead of baths (but power showers can use as much as a bath) and sensors on urinals and other appliances which would then provide water only when required.
- Encouragement of recycling by industrial users – it is increasingly economic for industrial users to pay for water audits, which identify wastage, processes that could be made more water efficient, and where recycling can be utilised.

- Domestic water audits – water audits are a good way of identifying both wastage and where more efficient use could be made of water. These are most effective when carried out by trained personnel, but this is generally not economic. The cost-effective way is to send a pro-forma to users, for them to complete, but this results in much lower benefits.
- Leakage reduction – English and Welsh water suppliers now have to assess and achieve their economic level of leakage. This is monitored by the water-industry regulator. Total leakage in 1994/5 was 5112 Ml/day but by 2006/7 this had reduced to 3420 Ml/day (Ofwat, 2007). This improvement was a result of great efforts by the water companies. The methods used were reducing the pressure in the water-distribution network wherever possible; establishing DMAs to monitor flows, using data loggers to identify new leaks; using leak noise correlators to identify the location of leaks; and the prompt repair of leaks and pipe bursts.

As well as the above, there are future possibilities.

- Water re-use in households – about one-third of household use is for flushing the WC. This function does not need high-quality water. Experiments are being made to recycle grey water from baths, showers, and wash-hand basins. This needs some treatment to remove impurities and some disinfection to ensure no scum build up in the pan. It also needs some storage and generally a pump. This system is currently too expensive to retro-fit into existing properties.
- For new properties, a water re-use system, or the use of rainwater from roof drainage, is entirely practical, but in practice such systems are rare. They need regular cleaning and maintenance and are not yet ‘fit and forget’. The basic issue remains that it is not economic to re-use water as the cost of water from the public network is low.
- Re-use of effluent – Article 12 of the Urban Wastewater Directive indicates that treated wastewater should be re-used wherever appropriate. Currently, only Windhoek in Namibia directly returns treated sewage effluent to the water-treatment works. However, the Langford re-use plant of Essex & Suffolk Water takes treated sewage effluent from the Chelmsford Sewage Treatment Works, and after additional treatment returns it to a river for subsequent abstraction and potable treatment (chapter 3). Several UK water companies are considering effluent re-use schemes in their water resources planning.
- The use of smart meters coupled with seasonal tariffs. Such tariffs would both discourage use of water at periods when suppliers wish to reduce consumption and cover the costs associated with higher cost sources of water, such as desalination. Technically, this is entirely practicable but in the UK there are barriers, difficulties with regulatory approval, and presumably the additional costs.

Figure 17.1 Water demand in England and Wales 1961–2007



The Environment Act 1995 (1995) requires water suppliers in England and Wales to produce water efficiency plans to promote the efficient use of water by its customers and these strategies are reviewed annually by the industry's commercial regulator, Ofwat.

Research has shown that demand management can significantly affect water consumption. Installation of metering has been shown to reduce average water demand by around 10% (WSA/WCA/Ofwat/WRc/DoE, 1993) and peak demand by up to 30%. This scale of reduction is significant in its own right, either allowing a reduction in the capacity of any new water-supply infrastructure needed or postponing the need for demand driven new investment. Figure 17.1, based on data from the Environment Agency and Ofwat, shows the average daily amount of potable water put into the distribution system since 1960, by the English and Welsh water companies. The quantity therefore includes water lost through leakage. It can be seen that the annual increase in water demand has levelled out and started to fall. It remains to be seen what happens in the future but there is little doubt that use will not increase as it has in the past.

17.9. Water-resource planning

Demands and demand management obviously need to be considered within the restraints of a particular supplier's water resources. As noted earlier, every five years water companies in England and Wales are required to produce an assessment of available water resources and projected demands over a 25-year period for review by, and agreement with, the industry regulator.

These assessments establish the long-term need for water-resource development within a particular supply area. The supplier's plans take into account actual and projected changes in demand and available resources, including the probable effects of climate change.

The demand forecasts for households can be prepared by projecting the levels of ownership of water-using appliances, the frequency of their use, and the water consumption per use (or cycle for washing machines). Projected demand management savings such as those achieved from the installation of new household meters and leakage reductions to economic levels are also allowed for. Where new resources are found to be required, following the implementation of demand management measures, the supplier's long-term resource development profile is established by prioritising new schemes, such as new boreholes and storage reservoirs, in least cost order.

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Appendix 1
Sample calculations

A1.1. Examples of calculations for coagulation

A1.1.1 Example 1: Sinuous channel

A sinuous channel has 15 round-the-end cross-walls. Water is passed along with a velocity of 0.2 m/s between the cross-walls and 0.5 m/s round the ends. The flow is 0.3 m^3/s and the nominal retention time is 25 min. For a temperature of 108°C, estimate the additional loss of head, the power dissipated, and the *G* and *GT* values (Figure A1.1).

$$\begin{array}{l}\text{Additional head loss}\\ \text{(given by } v^2/2g)\end{array} = \frac{16 \times 0.2^2 + 15 \times 0.5^2\ \text{m}}{2 \times 9:81} = 0.224\,\text{m}$$

Using Equation 5.2:

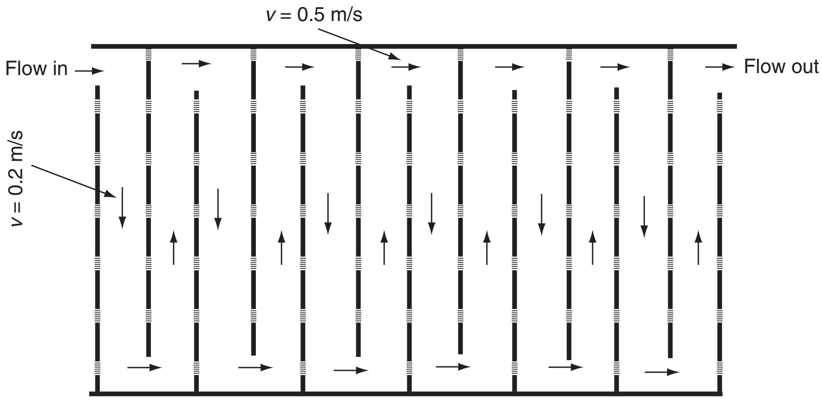
$$P = 0.3 \times 9{:}80 \times 10^3 \times 0.224\ \text{W} = 659\ \text{W}$$

and

$$T = 25 \times 60\ \text{s} = 1500\ \text{s}$$

$$V = 0.3 \times 1500\ \text{m}^3 = 450\ \text{m}^3$$

Figure A1.1 Sinuous channel for flocculation



Using Equation 5.1:

$$G = [659/(1.31 \times 10^{-3} \times 450)]^{0.5}\ \text{s}^{-1} = 33.4\ \text{s}^{-1}$$

Thus

$$GT = 33.4 \times 1500 = 5.01 \times 10^4$$

A1.1.2 Example 2: Weir mixing

A flow of 50 000 m^3/day is to be dosed with a coagulant. Destabilisation will be by double layer compression. To ensure proper mixing, a G of 1500 s^{-1} is required. Assuming that the flow over the weir discharges into a chamber with a volume of 2 m^3, what height of weir is required?

$$\text{Flow} = 0.58\ \text{m}^3/\text{s}$$

Therefore

$$\text{retention time in mixing chamber} = 2\ \text{m}^3/(0.58\ \text{m}^3/\text{s}) = 3.4\ \text{s}$$

Apply Equation 5.3:

$$G = (\rho g h/\mu T)^{0.5}$$

Thus

$$h = G^2 \times \mu \times T/\rho g$$

$$h = 1500^2 \times 1.31 \times 10^{-3} \times 3.4/(9.8 \times 10^3)$$

Thus

$$h = 1.02\ \text{m}$$

A1.2. Examples of calculation of chemical dosages

A water-treatment works treats a flow of 50 000 m^3/day. The water is dosed with alum at a dose of 5 mg Al/l. This permits the removal of an average of 5 mg/l of colloidal material in the clarifiers. Calculate the following.

1 The quantity of 8% Al_2O_3 alum solution required (specific gravity of 8% solution is 1.32).
2 The amount of alkalinity required to react with the alum.

3 The volume of sludge produced assuming a sludge solids content of 2.0% (on a weight basis) and a specific gravity of 2.2 for the sludge solids.

1 The quantity of aluminium dosed per day is:

$$50\,000\ \text{m}^3 \times 5\ \text{mg Al/l} \times 10^3\ \text{l/m}^3 \times \text{kg}/10^6\ \text{mg} = 250\ \text{kg Al/day}$$

The molecular weight of Al_2O_3 is 102 and the atomic weight of aluminium is 27; therefore 54 kg Al is equivalent to 102 kg Al_2O_3.

Thus, 250 kg Al is equivalent to

$$250 \times 102/54\ \text{kg}\ Al_2O_3/\text{day} = 472.2\ \text{kg/day}$$

The alum is being dosed as an 8% Al_2O_3 solution. Thus the weight of alum solution being dosed is:

$$472.2\ \text{kg}\ Al_2O_3/\text{day} \times 1/0.08 = 5903\ \text{kg/day}$$

At a specific gravity of 1.32 this is equivalent to a solution volume of:

$$5903/(1.3 \times 1000)\ \text{m}^3/\text{day} = 4.47\ \text{m}^3/\text{day}$$

2 The reaction between alum and natural alkalinity is given by Equation 6.1. Adding molecular weights we get:

$$\underset{594}{Al_2(SO_4)_3 \cdot 14H_2O} + \underset{3\times162}{3Ca(HCO_3)_2} \leftrightarrow \underset{2\times78}{2Al(OH)_3} + \underset{3\times136}{3CaSO_4} + \underset{6\times44}{6CO_2} + \underset{14\times18}{14H_2O}$$

Thus, 2 mol Al (with a molecular weight of 2 × 27) are equivalent to
1 mol $Al_2(SO_4)_3$ and require 3 mol $Ca(HCO_3)_2$ to react. Thus on a weight basis:

$$1\ \text{g Al will react with}\ (3 \times 162)/(2 \times 27) = 7.56\ \text{g}\ Ca(HCO_3)_2$$

Thus, 5 mg/l Al will react with 37.8 mg/l $Ca(HCO_3)_2$.

Expressing this as $CaCO_3$, it is equivalent to:

$$37.8 \times \text{equivalent weight}\ CaCO_3/\text{equivalent weight}\ Ca(HCO_3)_2$$

$$= 37.8 \times 50/81 = 23.3\ \text{mg/l as}\ CaCO_3$$

3 The quantity of sludge produced by the solids removed from the raw water is given by:

$$5\ \text{mg/l} \times 50\,000\ \text{m}^3/\text{day} \times 10^3\ \text{l/m}^3 \times \text{kg}/10^6\ \text{mg} = 250\ \text{kg/day}$$

In addition the quantity of hydroxide floc can be calculated from Equation 6.1. 1 mol Al (with a molecular weight of 27) produces 1 mol $Al(OH)_3$ with a molecular weight of 78. Thus, on a weight basis:

$$1 \text{ g Al will produce } 78/27 \text{ g } Al(OH)_3 = 2.9 \text{ g}$$

Thus, 250 kg will produce 725 kg $Al(OH)_3$. Thus, total weight of solids produced is 975 kg/day. This comprises 2% by weight of the sludge produced. The remaining 98% is water and this will be a weight of $98/2 \times 975 = 47\ 775$ kg. Thus, the total weight of sludge $= 47\ 775 + 975 = 48\ 750$ kg/day.

The volume of sludge solids will be 975 kg/2200 kg/m^3 = 0.443 m^3, and the volume of water will be 47.775 m^3, giving a total sludge volume of 48.218 m^3/day.

Note that for sludges having a low solids content an acceptable answer is obtained by simply assuming that the quantity can be calculated on the basis of a specific gravity of 1 for the sludge solids. In this case the volume calculated on this basis would be $975 \times 100/2 \times 1/1000 = 48.75$ m^3/day. The difference is small, representing the difference in volume between 975 kg of solids at a specific gravity of 2.2 compared to a specific gravity of 1.

A1.3. Example of calculation of detail design of plate settler – simplified example

As an example of the factors to be taken into account, the following example is a simplified procedure to design plate clarifiers to treat a flow 'Q' of 250 000 m^3/day (10 416 m^3/h), a very large flow equivalent to a population of the order of 1 million. Assume a particle settling velocity of 0.6 m/h (a low value for alum floc). For a surface-loading rate (Q/A) of 0.6 m/h the effective area 'A' of plate required is:

$$A = 10\ 416/0.6 \text{ m}^2 = 17\ 361 \text{ m}^2$$

Assume that the plates are inclined at 60°, and are 2 m long and 5 m wide. Then the projected area of one plate, 'a', would be:

$$a = 5 \times 2 \times \cos 60^\circ = 5 \text{ m}^2$$

Thus a total of 3472 plates would be required.

The clarifiers require quiescent settling and as stated in chapter 8, this requires a Reynolds number of less than 800.

$$Re = \rho v L/\eta$$

where Re = Reynolds number, v = velocity of the water, η = dynamic viscosity (temperature dependent – assume minimum water temperature of 0°C = 0.001 791), and L = the spacing between the plates.

Consider a plate spacing of 100 mm. Then

$v = 10\,416/3600/(0.1 \times 5)/3472\ \text{m/s} = 0.001\,667\ \text{m/s}$

$Re = 1000 \times 0.001\,667 \times 0.1/0.001\,791 = 93$

which is satisfactory.

In practice, the plate area would be increased to allow for entry and exit effects, typically by around 25% to 4340 plates.

It is of interest to calculate the net surface-loading rate on an installation like this. Clearly this depends on such factors as plate thickness and the number of tanks used. Assume that 16 tanks are provided and plate thickness is 10 mm. The surface area of each tank, which would contain 272 plates, would be:

$5 \times (272 \times (0.1 + 0.01)/\cos 30^\circ + 2 \times \cos 60^\circ) = 178\ \text{m}^2$

Thus, total area is 2848 m^2. Thus the apparent surface loading is 250 000/2848/24 m/h, which is 3.65 $\text{m}^3/\text{m}^2/\text{h}$. This is more than six times the loading that would be possible for the floc considered in a conventional rectangular settlement tank.

The retention time in the settlers would be approximately 20 min, or 25 min if the area of plate were to be increased by 25%. Note that although decreasing plate spacing would not affect the Reynolds number, it would increase flow velocity, and decrease retention time, requiring better process control in order to react more quickly to water quality changes or process upsets.

A1.4. Examples of calculation of chemical dosages required for soda ash softening

In order to calculate the quantities of lime and sodium carbonate required to soften a given water it is necessary to know the concentrations of magnesium and calcium, the concentrations of the anions, including bicarbonate, and the pH of the water. This allows the calculation of calcium and magnesium carbonate and non-carbonate hardness. Consider, for example, water with the following analysis.

Parameter	Concentration: mg/l
Sodium	20
Potassium	30
Calcium	5
Magnesium	10
Chloride	40
Bicarbonate HCO_3^-	67
Sulfate	5
Nitrate	10

The first step is to construct an equivalence table.

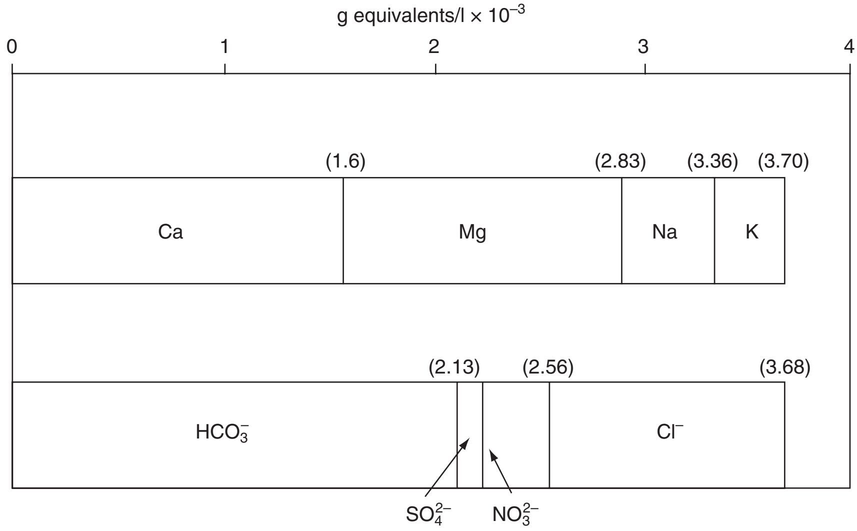
Species	Molecular weight: g	Equivalent weight: g	Concentration:		Cumulative concentration: g equivs/l
			mg/l	g equivs/l	
Ca^{2+}	40	20	32	0.00160	0.00160
Mg^{2+}	24.3	12.15	15	0.00123	0.00283
Na^+	23	23	12	0.00052	0.00336
K^+	29.1	29.1	10	0.00034	0.00370
HCO_3^{2-}	61	30.5	65	0.00213	0.00213
SO_4^{2-}	96	48	5	0.00010	0.00224
NO_3^{2-}	62	31	10	0.00032	0.00256
Cl^-	35.5	35.5	40	0.00113	0.00368

The final column shows the anions and cations to be approximately in balance, as would be expected, or at least hoped. By constructing an equivalence diagram (Figure A1.2) it can be seen that the hardness is as follows:

Type of hardness	Concentration: g equivs/l
Calcium carbonate hardness	0.00160
Magnesium carbonate hardness	0.00053
Calcium non-carbonate hardness	0
Magnesium non-carbonate hardness	0.00070

Finally the quantities of lime and sodium carbonate can be calculated from the equations set out above and the equivalent weights of lime ($74/2 = 37$) and sodium carbonate ($106/2 = 53$).

Figure A1.2 Equivalence diagram



Type of hardness	Concentration: g equivs/l	Lime required:		Sodium carbonate required:	
		g equivs/l	mg/l	g equivs/l	mg/l
Calcium carbonate	0.00160	0.00160	59.2	0	0
Magnesium carbonate	0.00053	0.00106	39.2	0	0
Calcium non-carbonate	0	0	0	0	0
Magnesium non-carbonate	0.00070	0.00070	25.9	0.00070	37.1
Total		0.00336	124.3	0.00070	37.1

The above represents a simple calculation. It ignores the effect of dissolved carbon dioxide that will create a lime demand in addition to that calculated above. The free carbon dioxide can be found by means of a fairly complex calculation if the pH value, temperature and TDS are known.

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