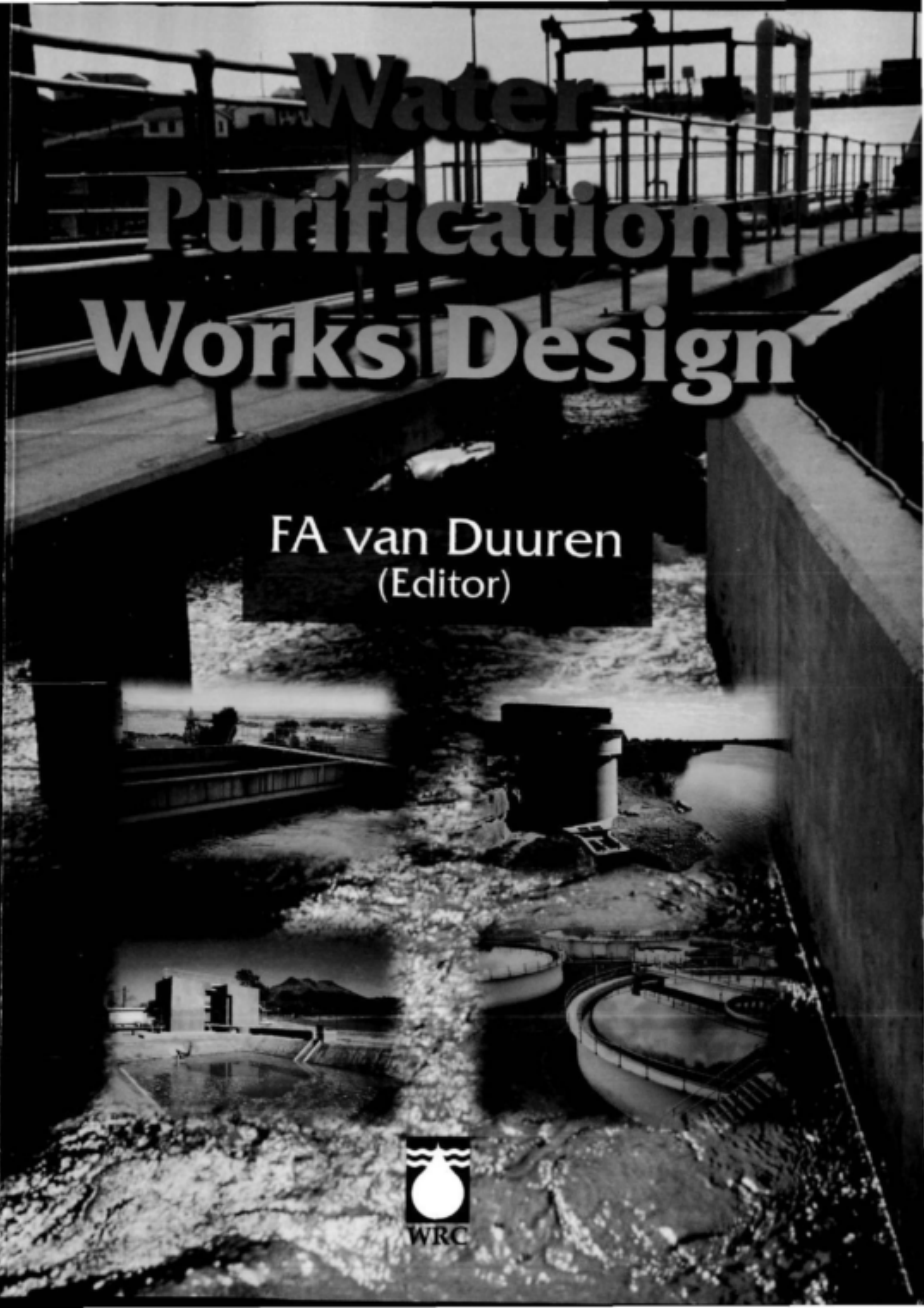

Water Purification Works Design

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Water Research Commission

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TT 92/97

Obtainable from:

Water Research Commission
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0001

The publication of this report emanates from a project entitled:
A Guide for Water Purification and Plant Design: Phases 2 & 3
(WRC Project No. 504)

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ISBN 1 86845 345 6

Printed in the Republic of South Africa
Beria Printers
1997

Preface

The objective of water purification is to produce a sufficient amount of the most suitable product economically. This is done on the basis of water quality and quantity considerations, processes and operations. The health and hygienic aspect of water is of primary importance for domestic purposes. However, purified water should also be aesthetically pleasing, non-corrosive or scale-forming and it should meet a number of other quality requirements.

The purpose of water treatment is to modify or alter one or more of the quality parameters of the water for a particular purpose.

A knowledge of the quantum of water required for domestic and industrial purposes is essential to the sizing of water works. Water needs and requirements for domestic and industrial purposes therefore have to be assessed as a first step to design. A water source has to be obtained to suit these needs.

The quality aspects of water include those of the raw water resources and of the purified and treated water. These qualities have implications on and in fact would determine the purification and treatment processes.

The processes include coagulation, disinfection and treatment for example, to control corrosion and scale formation. It is necessary to know what water works chemicals, coagulants and disinfectants are desirable and are available. The purposes to be achieved by the use of these have to be known.

The unit operations considered, include dispersion and mixing of coagulants after addition to the raw water, flocculation, sedimentation, clarification, flotation, filtration, sludge and wash water disposal. The availability of various types of plant and equipment involved in the processes and operations, especially locally has to be known. Other aspects which are of importance in water purification and treatment include raw water intakes and disinfection contact tanks.

The mechanical and electrical plant and equipment for the various unit operations are of interest and are dealt with. Instrumentation, including flow, turbidity and other measurements is also given consideration.

Planning, costing and estimation, operation, control and management of water works is given some consideration as they affect design and is affected by it.

This text has been structured for and is aimed at water works design engineers and planners. It should also be of some value to water engineering lecturers and their post graduate students. It is certainly didactic, having been largely written by engineers - the reader is nevertheless urged to use it in the good spirit in which it is offered.

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The chapters of this text were authored and reviewed by recognised South African experts on the particular subjects. The authors and reviewers of the various chapters are detailed below. In all instances the author is first mentioned and then the reviewers.

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Foreword

Being a water-scarce country, South Africa has to contend with surface waters concentrated in both natural and anthropogenic organic material. This is exacerbated by the fact that heavy thundershowers and flash floods result in high surface water turbidities during the summer months. In many catchment areas, low turbidity, eutrophic water is supplanted by high turbidity floods within minutes. Eutrophic conditions are frequently accompanied by turbidity. This, rather unwelcome, plethora of possible feed water qualities to take into account in the design of a water purification works, has caused many a designer considerable inconvenience in the past in terms of time, money and efficiency. Although excellent water works design textbooks are available internationally, these do not always cater for local conditions and requirements.

This textbook *Water Purification Works Design* is the proud product of a team of dedicated and talented engineers and water scientists in the South African water industry. It is the first true water purification design guideline to appear since the *Manual on Water Purification Technology,* which was produced by the CSIR in 1985. It is hoped that this guideline will not only be of some assistance to the designers of water purification works, but also to the designers-in-waiting, the water works operations personnel, and the many knowledge-hungry students of water purification technology and works design.

PE Odendaal
Executive Director
Water Research Commission

Dedication

To Dr G J Stander for his encouragement to the editor in this enterprise

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1

General Design Considerations

F A van Duuren

The general design considerations outlined here include a procedure that can be followed in the design of a water purification works. These considerations contain a number of elements, the first being investigation and study. This is followed by preliminary design, specification and quantities, tender documentation and procedures, detailed design, construction and commissioning, operation, control and management.

It is necessary to investigate and study the provision of a water works in the context of a water scheme. Without a reliable and acceptable source of supply, a water works will serve little or no purpose. The water scheme might be multipurpose, in which case the demands and needs of the other users will also have to be considered. Having determined the source quantitatively and qualitatively the works then has to be viewed within the context of water supply and distribution.

The quantity available, and the quality and treatability of the source has to be assessed. The purpose for which the purified and treated water will be used has to be known and taken into account in the process determination and design. The quantity needed and quality requirements for domestic and industrial purposes will be based on consideration of these aspects.

If water records are available, they should be used. Information from works dealing with similar waters should be obtained, assessed and used to the extent that it is possible. The experience gained on similar waters could be of some value. However, advances in water purification and treatment technology should be taken into account. Analyses, tests and experimentation should be carried out in case additional information is required. Pilot-plant work may be necessary where no information or experience exists; for larger works this may well be essential and decision-making methods are available for this purpose.

A water purification works should not be designed without taking the preliminary and essential steps although these may require a year or more to carry out properly. The process of providing a medium or large, functional water works takes from three to five years.

A systems approach to design considerations and procedures can be followed sequentially in terms of the following:

- Investigation phase
- Preliminary design and specification
- Tender procedure and report
- Detailed design
- Construction and commissioning
- Operation, control and management
- Monitoring

1.1 INVESTIGATION PHASE

The engineer or, in the case of larger works, the design team, should carry out the necessary investigation and study prior to design of the works.

In this phase, the following components have to be considered:

- Needs and requirements
- Source
- Process selection

- Capacity of the works
- Cost estimation and economics
- Programming
- Investigation report.

1.1.1 NEEDS AND REQUIREMENTS

Water needs and quality are considered in some detail in Chapter 2 of the text, only an outline is given here of quantity and quality.

1.1.1.1 Quantity

The historical, present and future needs and requirements have to be determined or assessed. The use of different grades of water for various purposes requires consideration, especially if the source quantum is very limited. The concept of demand has to be placed in perspective with aspects such as unaccounted-for water and waste.

Water consumption and population statistics have to be obtained from local and national sources and projections should be based on these. The local authority is usually the best source of these statistics. However, for larger or regional schemes information and data should be obtained from Central Statistical Services and used.

Domestic water requirements for design purposes, are indicated in Table 2.5; industrial water intake data are given in Table 2.6.

1.1.1.2 Quality

Quality has to be assessed in terms of the properties of the available water sources and the requirements for domestic and industrial purposes. Analyses of water at existing works or from water works drawing from the same source should be acquired and used. Water quality parameters are given in Tables 2.12 to 2.17.

1.1.2 SOURCE

The aspects of the source or sources which require consideration are:

- Availability
- Alternative sources
- Yield and assurance
- Quality
- Monitoring
- Siting of works
- Legal aspects.

1.1.2.1 Availability

Availability and constraints of supply should be ascertained from the Department of Water Affairs and Forestry. A first approach should be made to the Regional Engineer of the Department. Advice should also be taken on the legal aspects and necessary procedures for applications to withdraw and utilise water.

1.1.2.2 Identification of alternative sources

Alternative sources should be identified; the use to which the water will be put, will play a role in this regard. The two main sources of water are underground and surface. Groundwater can serve many communities well because it is naturally protected against most pollution and its cover obviates evaporation. Surface water sources usually have to be stored in South Africa as there are few freshwater lakes or perennial rivers.

1.1.2.3 Yield and assurance

Assurance of supply from alternative sources has to be determined. This can be done in the case of groundwater by testing the flow from boreholes for 6 to 12 hours; in the case of surface water it has to be determined from hydrological study and analyses.

Groundwater should be conserved for times of low rainfall; this implies some amount of installation for drawoff and protection against pollution. Drawoff of groundwater should be properly regulated by means of water flow and depth recordings.

Surface water sources can be identified more readily, but the hydrology must be known. The Department of Water Affairs and Forestry has a hydrological data bank which is valuable for assessment of surface water sources.

A combination of ground- and surface water sources is ideal because groundwater can be kept in reserve against times of drought.

1.1.2.4 Quality

It is essential to ascertain the quality of water from the alternative sources. The quality of the source should be determined over a period of time, preferably over a number of seasons. Water samples should be taken of the source for analysis and testing; flows should be determined simultaneously. The properties of the source water that have to be determined are indicated in detail in Chapter 2.

1.1.2.5 Monitoring

All the foregoing aspects should be monitored on a continuous basis and taken into account even beyond the completion of the works. Unusual occurrences such as quality changes, pollution or extreme flows should be noted and taken into consideration. Steps should be taken to ascertain the reasons for quality changes or deterioration.

1.1.2.6 Siting of works

Siting of water works relative to the source and the distribution system has to be considered as it will have cost implications. It might well be possible to purify water close to the source such as an impounding reservoir or a river. A single group of personnel could then provide the necessary services. However, aspects such as bulk supply and distribution will also be involved.

Aspects of importance are:

- Alternative sources
- Quality as regards turbidity and silt, corrosivity and scale-forming; also intake relative to sources of pollution
- Flows to the extent that conveyance of large quantities of raw water may be unacceptable
- Strategic placing, i.e. availability and cost of land, altitude, geology, proximity to electric power and transportation
- Staffing and amenities
- Future extension.

Other aspects of siting are indicated in Section 1.4.1.

1.1.2.7 Legal aspects

The legal aspects involved in source utilisation include the necessary permits for intake of water and quality of the product. The disposal of effluents from the works has to be dealt with in terms of the applicable Water Act. The grading of the works and thus of the operating personnel required are also laid down in the Act; these should be noted and the necessary steps taken to comply timeously.

1.1.3 PROCESS SELECTION

Processes should be selected on the basis of quality parameters; this can be done manually or possibly by means of computer programs. Flexibility should be kept in mind with regard to process selection in order to accommodate changes in water quality with time. Process selection is dealt with in detail, in Chapter 3.

The applicability and local availability of plant and equipment and instrumentation for potential processes should be ascertained and discussed with suppliers. They should advisably be informed of the intention to design and construct a water works and of the investigation and study being carried out so that pertinent or useful input may be obtained.

The design life for processes and equipment is 15 to 20 years. Some processes become outdated and equipment does wear out. Concrete structures and buildings last some 50 years and should be planned accordingly. Provision may well be made for future expansion. A 10 to 15 year period can be used for staged development of a water works.

The financial management and layout of the units of the works must be given due attention in deciding on the processes selected.

1.1.4 Capacity of works

Having determined the water needs as outlined above and indicated in Chapter 2, the 24-hour average and the 24-hour maximum have to be assessed and considered. Peak hourly demands have to be met from distribution storage.

If the works is remote from the distribution network and pumping is involved, care is needed in providing sufficient bulk storage at the end of the water main from the works.

Water purification works capacity can be designed for maximum 24-hour demand.

Process units have to sized with due consideration of daily, monthly and annual fluctuations in water demand and quality variation. This would affect the ranges of chemical feeds and storage.

1.1.5 Cost estimation and economics

The cost of one or more process alternatives has to be estimated and compared in order to arrive at the most economical solution. Siting and layout of the water works would be considered in this context. Unit costs for the product water should be determined and reported.

Design standards would have a cost and economics implication. These standards apply to processes, plant, equipment, buildings, electricity, roads and sanitation. Cost estimation of water works is addressed in Chapter 20.

1.1.6 Programming

The sequence in which the work on the components of the scheme has to be done should be considered and decided upon. The following aspects should be reflected in the program:

- Investigation report - draft, final report and approval
- Preliminary design - report and approval
- Raw water supply, intake, pumps and rising main
- Process plant and equipment - documentation, approval, tenders, evaluation, contract award, construction and installation
- Civil works - documentation, approval, tenders, evaluation, award, construction
- Purified water rising main and bulk storage - documentation, approval, tenders, evaluation, contract award and construction

TABLE 1.1
WATER WORKS DESIGN CHECKLIST

Item	**Action performed or required by** **C = Client body, E = Design engineer**	**Refer to C, E**	**Chapter**
Investigation phase	The following aspects have to be addressed:	E / C	
	• Is a water works necessary?		
	• What are the needs and requirements in terms of quantity and quality?	E	2
	• Source: ascertain in terms of availability, alternatives, yield, assurance, quality, siting of works and legal aspects	E	2
	• Processes: select	E	3, 19
	• Cost estimation and economics	E	20
	• Programming	E	1, 21
	• Investigation report	E	1
Report phase	Obtain client's instructions on the report and the recommendations made	E / C	
Design and specification	Modify the design or proceed	E	All
Tender procedure and report	Prepare the tender and proceed	E	1
	Prepare and present the tender report	E / C	1
	Negotiate as needed and obtain authorisation to proceed	E / C	
Detailed design	Proceed immediately after award of contracts	E	All
Construction and commissioning	Hand over site to contractor	C / E	
	Supervise construction	E	
	Commission works	E	
	Ensure that operation is under control and that as-built drawings have been finalised	E	1,21
Operation, control and management	Hand over works to management and ensure that the necessary arrangement has been made for an ongoing involvement	E / C E	1 21
Monitoring	Check functioning of the works periodically with management. Note problems to obviate. Assist in planning when works are to be modified or expanded.	E E / C & E / C	16,17,18 21 1

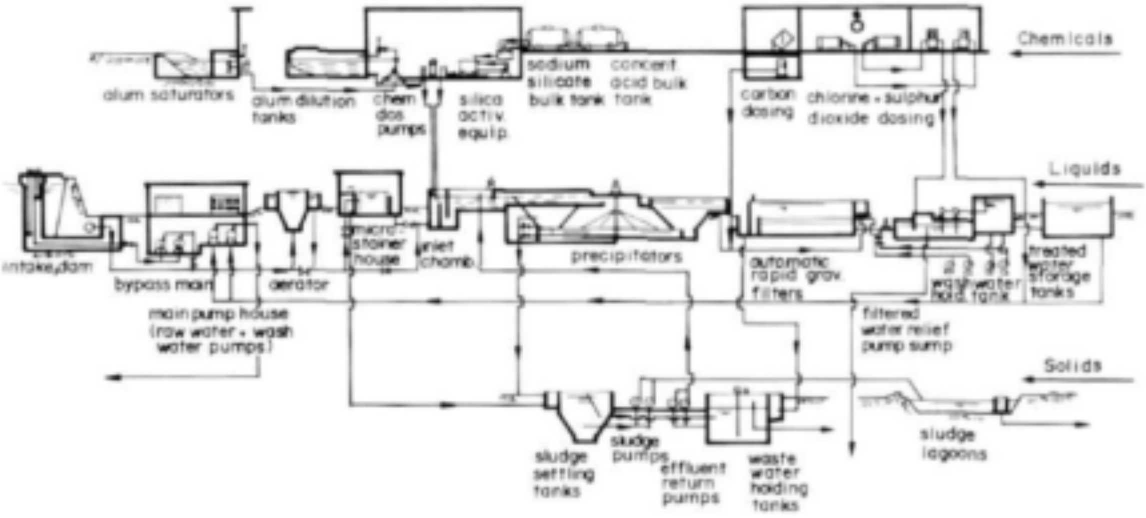


Figure 1.1
Typical water purification works

- Services, including roads and power supply to the works - investigation, report, approval and installation.

An aspect which should be incorporated in the program is the involvement of plant operators and for larger works, the manager. There should be substantial technical involvement in the program of the client body at all times. The interests of the end user should be taken care of by the client body or the employer.

1.1.7 INVESTIGATION REPORT

The investigation and studies have to be reported on and recommendations must be made on the basis of the components outlined.

The report should be presented to the client body, board or council in such a way as to enable them to make an objective decision. Alternatives must be spelled out.

The scheme should eventually be adjudicated in terms of the investigation and studies conducted of the recommendations made.

1.2 DESIGN AND SPECIFICATION

The design of the units of a water purification works is dealt with in detail in the various chapters of this text. Process selection and preliminary design should receive attention to facilitate decisions before going out to tender. At this stage a checklist procedure can also assist in decision-making.

A *pro forma* checklist is presented in Table 1.1. Each particular works requires a specific checklist compiled and agreed on by the design engineer and the client body.

1.2.1 SPECIFICATION

The specification should reflect the design so that the water works may be built and operated in terms of the process considerations and processes selected. The need for effective operation and management of all the components of the water purification works must continuously be borne in mind when the specification is written.

Specification of the works, including units, plant, equipment and instrumentation should reflect the processes and the design in the tender and contract documents. Care should be taken in the specification and conditions of tender to call for offers which can be compared on the basis of the processes and units selected. Scope should, however, be allowed for alternative offers in order to encourage interest, participation, competition and new developments.

Proper specification should obviate offers of inferior, undesirable or unsuitable processes or unit operations which might include cheaper plant and equipment. The consequence of accepting the cheapest offer may well be a more expensive or less economical works or even failure. Unsuitability, larger capital expenditure required to attain the objectives, more expensive maintenance or complex management and control procedures rule out certain offers.

The specification forms the definitive part of the tender document. Tender procedures and reports on tenders received have to be followed in the light of the preceding investigation study, design and specification.

The procedures and report on tenders are dealt with here in the context of the above considerations.

1.3 TENDER PROCEDURE

It must be decided from the outset whether a turnkey or a design/construct project is desired. In the case of a turnkey project, the flow and quality parameters are salient. For a design/construct project, the tender procedure is as follows:

The documents are compiled, including diagrams and drawings. Should the process plant, mechanical and electrical equipment tender document be issued first, as a separate document, the civil/structural design can be based on the offers received. The process contractor may well be nominated to be the principal agent in this procedure; in so doing placing the emphasis and responsibility where they belong.

TABLE 1.2
EXAMPLE OF DESIGN CRITERIA FOR A PROCESS UNIT

Number of units	1
Flow through the unit	30 Mℓ/d
Velocity gradient	750 s^{-1}
Temperature	150°C
Retention time	55,2 s
Volume of tank	2,52 m^3
Size of tank	1,2 x 1,2 x 1,75 m
Mixer motor size	1630 watt
Mixer type	propeller

By calling for tender documents to be completed fully, in terms of the specification and quantities, a rational comparison can be made of the process aspects of the works.

Alternatives should be allowed for, so that suppliers of plant and equipment may compete on the basis of various patents held by them. Such alternatives should be submitted in a separate document or letter from the tenderer and should not be allowed without good reason and discussion, to replace the process specified in the tender. It must be possible to assess the effect of alternative processes, plants and equipment offered, on the water works scheme as a whole.

It is necessary to take into account the cost implications of the civil/structural components of the process plant and equipment offered. The unit rates for concrete, earth works and other civil engineering elements should be known so that they may be taken into account in the overall assessment and adjudication of the tenders received.

The advertising, receipt and opening of tenders should be done in accordance with the procedures of the controlling authority.

1.3.1 TENDER REPORT

Analysis and adjudication of tender offers should be done on the basis of the specification and the applicable conditions, with due consideration of the economics.

The report on the tenders should reflect the nature and scope of the tender, tenders issued, notices, site inspection, tenders received, comparative prices and a detailed analysis of the three lowest tenders in terms of the plant and equipment offered.

The recommendation made to the client body, can be done on the basis of the considerations outlined. It might be necessary to discuss certain aspects of their offers with the tenderers. This has to be done strictly in accordance with the procedures of the controlling authority but must also take into account the contracting association's rules for their members. The final recommendation by the design engineer (and his team), should be clear and unequivocal.

1.4 DETAILED DESIGN

The amount of detail required will generally depend on the size of the works. For a small, package plant, the basic design parameters will include capacity, raw and purified water qualities and the preferred method of purification and treatment. For larger works many other parameters are necessary as will be clear from the other chapters of this text. More detailed design would therefore be needed for larger works.

A layout of the units of the works is required at both the preliminary and detailed design stage in order to facilitate siting, placing of the units, hydraulics design and provision of services.

1.4.1 SCHEMATIC DIAGRAM AND WORKS LAYOUT

The processes and their operation, control and monitoring can be represented by a mimic diagram and a layout plan. The mimic diagram shows the processes and operations, hydraulic linkage and profile as well as instrumentation. It is a cross-section or elevation showing all of the units and processes of the works. Together with the detailed design criteria, the diagram and layout plan should yield a clear concept of the entire works and thus facilitate the preliminary and detailed designs.

The mimic diagram and detailed design criteria are developed (AWWA, 1990) by establishing:

- Main units, their size and place in the scheme of design
- Kind, amount and points of application of chemical reagents
- Monitoring and control of the processes and chemicals.

To this should be added the hydraulic linkage, gradients and levels of the units. The solids removal from the units and sludge disposal should also be given attention at this early stage. The water works may well be viewed in elevation, as is indicated in Figure 1.1 where chemicals, liquids and solids lines are clearly discernible.

Design criteria are selected in the detail necessary to establish the dimensions of the units, mechanical devices, equipment and pumps. The hydraulic profiles can then be developed for pumpage and gravitation of liquid and suspended solids. Table 1.2 shows, by means of an example, a rapid mix unit (taken from Chapter 7), how design criteria can be tabulated for the various units.

Similar tabulations have to be made for all the units in the flow path downstream; it is necessary to do so at the preliminary design stage and to make modifications as required by the tender items offered. It should be noted that the normal maximum flow is used in the design of the unit, but that the unit should be able to pass 25% more hydraulically.

The hydraulic grade line fixes the elevations of the water surfaces in the various units and their surrounding walls. The total head loss through a water works will be between 4,5 and 6 m (Montgomery, 1985). The elevation of the works is dependent on the raw water source and the point of supply to a pressure zone; it is also influenced by geotechnical considerations. In the works itself the losses through the units, pipelines and fittings must all be taken into account.

Profiles should be drawn for average and maximum flows in order to determine adequate freeboard in the units and channels. The topography and geology of the site will also play a role in the levels of the units. The flow of water through the units should be kept as direct and short as possible, especially with regard to solids and pipes and conduits, conveying chemicals.

The works control room should be adjacent to and have a clear view of the units. It should have just the right degree of comfort: enough to be acceptable but nevertheless not so much that it discourages regular visits to all components of the works. The chemicals room should be in close proximity to the dosing tanks and mechanisms.

Site plans should be drawn to a suitable scale to reflect process units, buildings and roads. This plan should also show large pipe lines, drainage and landscape. The architectural and structural details should reflect the shape and size of the structures and building materials. The structural dimensions of major walls, beams and columns need to be indicated. The site plan should also indicate access roads, electricity, geotechnical and future extension information.

1.4.2 CONSTRUCTIBILITY

During the detail design stage attention must be given at all times to ease of construction and sequence of operations. This is particularly necessary when a works has to be augmented or upgraded. Construction specialists should be involved in order to minimise problems (American Water Works Association (AWWA, 1990)). Aspects which are of importance include:

- Structures should be designed to allow for simplicity of excavation and foundations;
- It should be possible to operate existing works without interruption;
- Construction documents should be checked in detail to cut down on conflicts in the field;
- Temporary utilities and storage should be provided, by agreement and as required by the contractor; and
- Works staff should be trained by the engineer and by the process and equipment contractor.

1.5 CONSTRUCTION AND COMMISSIONING

Construction of the water works should be carried out with a proper appreciation of the concept of the preceding investigation, study, report, design, specifications and recommendations made.

Attention should again be given at this

stage to the operational requirements of the works. It is advisable that at least some of the operators observe the construction and especially the details of the works which will eventually be under water, covered or concealed. Operators could well be involved in the preparation of as-built drawings of the works.

The water works must be commissioned in close cooperation between the process plant and equipment suppliers, the operators, management and the client.

1.6 OPERATION, CONTROL AND MANAGEMENT

The aspects of operation, control and management impinge on all the facets of design and have to be borne in mind continuously by the design engineer and his design team as even the best-designed works will perform indifferently without proper operation and management. Careful attention to the operational implications of the design will clearly be reflected in the completed works, for example avoiding an unnecessary cat ladder will be self-evident, while making operational equipment inaccessible will demonstrate unacceptable indifference.

The client must be thoroughly consulted to ensure the most appropriate operation, control and management of the water works.

Operation, control and management cover a very wide range as is indicated in Chapter 21.

1.7 MONITORING

An essential element of a systems approach is feedback. An assessment must be made over the first year of operation, control and management of compliance of the water works with the objectives of its provision. The quantitative and qualitative performance of all of the units and processes of the works must be assessed and reported to the client body and the authorities.

There should be an ongoing involvement of the designers, after commissioning. This is desirable in order to deal with problems as they arise in continuous operation, with changes in seasons and conditions. This should be done to ensure proper placing of responsibility for the design. Under EPA regulations in the USA a three-year post-commissioning involvement of the designers was contractually required to ensure proper control of (massive) state expenditure.

The works should preferably be monitored by the design engineer and members of his team, over a period of time in order to assess its performance, to advise on any necessary modifications and to issue reports which can be used in future planning. Monitoring should be done periodically after the works has been handed over to the water engineer's department.

2

Water Needs and Quality

A Bell

The quest for pure water has been pursued for over 4 000 years (Baker, 1948). Due to new attitudes and preferences which also give rise to new criteria, together with the continuous development of new materials, processes and operations, this is a never-ending search.

The demand for treated water in the past was earlier understood to be the quantity that the domestic, industrial and other users wanted to have regardless of its availability or other considerations. Because of water scarcity, it is of national importance that a water management strategy be developed.

Because of the uneven growth of population and industry and the initial relatively small industrial water demands, water has traditionally been purified in a single works and then supplied jointly to the domestic and industrial sectors. This has been done despite the different quality requirements of the two sectors or the variation within either of them. Separate supply systems have, however, come into existence for major industries, power generation and the mines. It is therefore now more readily possible to deal with the quantity and quality requirements of these two sectors separately.

The shortage of water together with environmental considerations and the effects of pollution finally forced the realisation that needs can possibly be satisfied but not excessive demands. Optimum utilisation of water thus presupposes careful categorisation of quantity, quality and consumptive use (Van Duuren, 1970).

TABLE 2.1
WATER CONSUMPTION IN 1980

Demand sector	(million m^3/a)	(%)
Direct use		
Municipal and domestic	1 516	9,3
Industrial	1 031	6,3
Mining	446	2,9
Power generation	282	1,7
Irrigation	8 504	52,2
Stock-water	262	1,6
Nature conservation	178	1,1
Indirect use		
Forestry runoff reduction	1 284	7,9
Ecological use, estuaries and lakes	2 768	17,0
Total	16 291	100,0

In 1980 the estimated municipal, household and industrial water consumption in the RSA was 2547 x 10^6 m^3 (i.e. 15,6% of the total water consumption of 16 291 x 10^6 m^3). This is expected to grow to 7438 x 10^6 m^3 representing 28,7% of the total by the year 2010 (See Table 2.1). Purified water will thus have to be provided in many urban and rural areas in the country.

When designing water supply projects, whether completely new or to augment existing systems, important engineering decisions need to be taken, such as:

- selection and monitoring of supply sources;
- estimating the population and population growth and types and magnitudes of commercial, industrial and agricultural undertakings to be served during the design period;
- establishing the design period, which is the number of years during which the project will be expected to meet the consumer requirements;
- determining the level of service in terms of rate of supply, reliability of supply and quality of water to be provided; and
- fixing the capacities and operating levels of works required for the storage, treatment and distribution of water which should include provision for supply and demand fluctuations.

These factors must be carefully analysed and individually and jointly optimised when considering the primary purpose of purification, which is to improve the quality of the available raw water sources to meet the acceptable potable and industrial quality criteria.

Quality requirements for industry are extremely diverse, complex and ever-changing.

2.1 BRIEF OVERVIEW OF WATER SOURCES IN THE RSA

Water which is essential for life, for social development and for economic growth is a public commodity. It is regulated in the RSA in terms of the Water Act of 1956 (Act 54 of 1956) since it became necessary to apportion water more fairly between agriculture, industry, metropolitan, urban and rural users.

After the 1966 drought and in the light of the growing demand for industrial water, a Commission of Investigation into Water Matters was appointed in 1970 to draw up a flexible national water management strategy for all water users. In 1986 the then Department of Water Affairs, which is the Central Government body exercising overall control, published *Management of the Water Resources of the Republic of South Africa*. This publication is aimed at increasing awareness of water management issues so as to promote an effective partnership between water users and central government.

The raw water sources can be categorised as surface waters (inland and coastal), ground water, effluent and sea water.

The annual average distribution of rainfall in the RSA, the average annual evaporation, the seasonal rainfall distribution, as well as the cumulative annual runoff of the major rivers are shown in Figures 2.1, 2.2, 2.3 and 2.4 respectively. This is to a certain degree indicative of water quality, i.e.:

- Lower alkalinity water present in the higher rainfall coastal regions from Cape Town in the south-west, around the coast to the North Eastern Transvaal. A high iron, manganese and organic content is often present in the coastal region between Cape Town and Port Elizabeth.
- The higher TDS waters are found in the lower rainfall areas such as the Karoo and central and western parts of the country.
- The higher turbidity flood waters are found in the summer and late summer rainfall regions.

It is estimated that 13% of the total water consumption in the RSA in 1980 was from underground water sources of which approximately 10% (i.e. 190×10^6 m^3/a), was used for metropolitan as well as for household consumption in urban areas. The distribution of dolomitic and other primary water carriers is shown in Figure 2.5. Quite a few of the primary water carrier waters have a high TDS and tend to have a high iron and manganese content (i.e. Klein Karoo, Sandveld, Alexandria and Bitterfontein

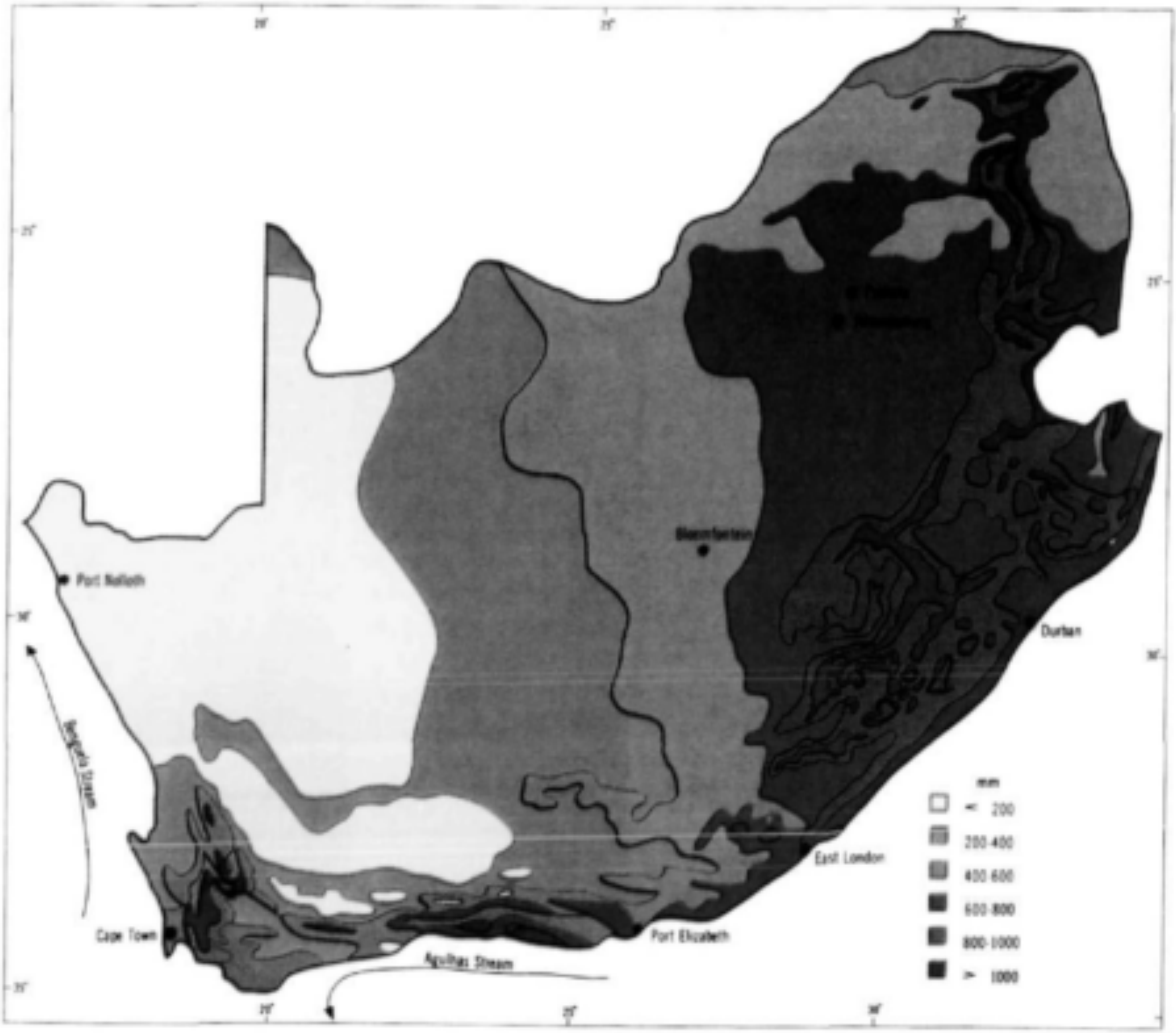


Figure 2.1 (top)
Mean annual precipitation

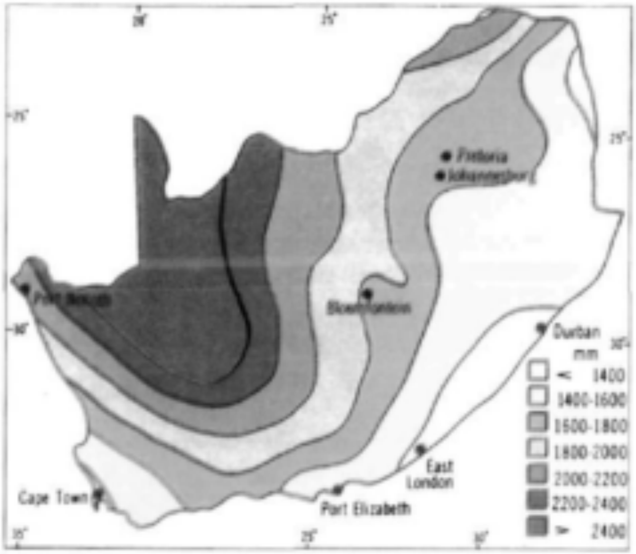


Figure 2.2 (right)
Mean annual evaporation from an open water source

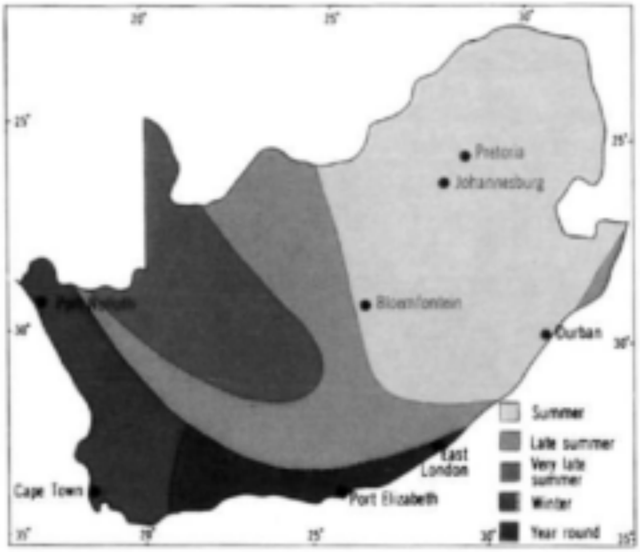


Figure 2.3
Seasonal rainfall regions

regions). The underground waters in the Southern Cape generally have a high iron and manganese content.

It can be foreseen that due to the non-availability of adequate surface and or underground waters, i.e. along the West Coast towns such as Port Nolloth, sea water may be quite a viable augmentation source. By the same token domestic and/or industrial effluent may well be a viable source of water for cities. Purified domestic effluent is being treated and distributed successfully as drinking water in Windhoek (Namibia), whilst sewage is being treated and distributed for industrial purposes in a number of cities and towns.

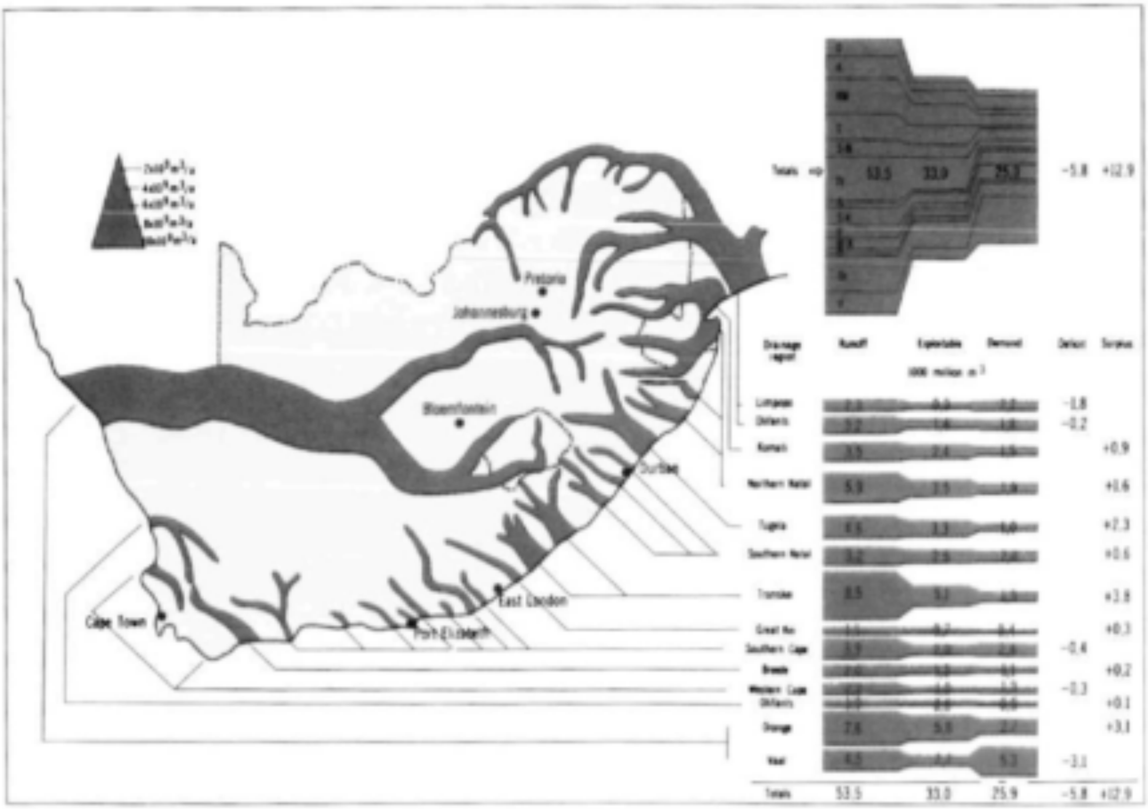


Figure 2.4
Cumulative annual river runoff (quantities in 1 000 million m³/a)

Figure 2.5 *Distribution of dolomitic and of primary aquifers*

2.2 SOURCE MONITORING

The quality and quantity of water from any source invariably change with time. These changes can either be short-term fluctuations, seasonal or long term.

As a purification works must be designed to cater for these fluctuations it is essential that they be monitored over a period of time adequate to establish the variability of all the essential design parameters. These data are necessary for the initial concept design as well as for the detailed design stages.

It speaks for itself that the monitoring programme should be devised or tailored around the ultimately required needs and final water quality irrespective of whether the source(s) is surface, underground or effluent water.

2.2.1 QUANTITY

Should the rate of supply from a source not be compatible with the required demand, the source is either inadequate and has to be augmented or its yield might have to be adapted to suit the needs of the works i.e. storage of runoff must be provided from a rainfall catchment, etc.

The inconvenience and damage that severe restrictions on water use may cause to different consumer sectors are being evaluated at present. In the interim, assurances of supply which are regarded as appropriate for various consumer or use sectors vary from practically 100% for strategic industries to the provision of the full long-term demand of 98% of the time for domestic and normal industrial requirements.

Quantity data to be monitored can, depending on the source, include the items in Table 2.2.

TABLE 2.2
MONITORING OF QUANTITY DATA

Type of measurement & application	Recommended frequency	Monitoring period - minimum
1. Rate of flow i.e. rivers, springs, effluent, etc.	. on continuous basis or intermittent (daily)	two years
2. Rainfall i.e. hydrological	. on continuous basis or intermittent (daily)	Long term
3. Geohydrological i.e. boreholes: rate of flow	. borehole - pump tests	Long term

Note: Rainfall and runoff hydrological (river flow) data of South Africa are obtainable from the Department of Water Affairs & Forestry.

2.2.2 QUALITY

As is the case for quantity, the designer must decide what provisions, if any, are to be built into a works and how predictable future fluctuations in quality must be addressed.

Relatively small seasonal algal blooms, for example, can be dealt with by killing off the algae in the raw water by pre-chlorination and allowing them to settle out in the clarifier. High loads of algae might, however, have to be dealt with differently, i.e. by flotation, screening, etc.; if not required in the early life span of a works, provision for future incorporation thereof can be considered.

Criteria for the monitoring of the quality parameters of a source are recommended in Table 2.3.

TABLE 2.3
MONITORING OF SOURCE WATER QUALITY

Application and type of monitoring	*Recommended frequency*	*Minimum period of monitoring*
1. **Health & pollution** . Bacteriological analysis . BOD, COD, NH_3 + PO_4	Monthly	Annually
2 **Aesthetics** . Turbidity . Colour . Flocculation jar test in laboratory	Monthly	Annually
3 **Economical** . Chemical analysis - pH - acidity - alkalinity - hardness - corrosivity - Fe & Mn - algae or chlorophyll	Monthly	Annually

2.2.3 POST-IMPLEMENTATION OF WORKS

It is important to monitor a purification works source(s) after start-up of a scheme, so as to take note of and address any unpredicted quantity and quality changes.

In the case of a borehole field, it must undoubtedly be monitored and managed from start-up of the project so as to compare the design and actual delivery of boreholes in the different geohydrological compartments.

2.3 WATER DEMANDS AND NEEDS

Water demands can be based on historical data whereas needs can be more rationally assessed by investigation and the analysis of data. In the absence of water metering, data are unreliable and the concept of demand prevails. This is true for domestic and industrial water use.

The water requirements or needs of these two sectors as well as of other users are indicated here to the extent that relevant information is available.

2.3.1 WATER USED FOR PUBLIC SUPPLIES

The total water needs vary for cities, small communities, rural and developing areas, i.e. public supplies, where each is influenced by geographic location, climate, standard of living, the supply system, utilisation patterns, income, ground valuation and other influencing factors.

If field measurements are not possible and actual flow data are not available, public water supply records can often be used to estimate water needs.

Water withdrawn for public supplies in the United States in 1954 and 1970 was respectively estimated by Metcalf and Eddy (1979) to be 556 and 628 *ℓ/capita*.day. These figures are however not recommended for use in South Africa.

Similarly Barnes *et al.* (1983) also give more universally representative figures of total daily water demand for large highly industrialised areas, cities with light industries and towns, where most of the supply is for domestic use, all for different climatic conditions.

Rand Water (1990), has kept water production and population served statistics since 1905 and the *per capita* consumption excluding industry and mining for the whole area of supply over the 20-year period to 1991 was between 160 and 249 *ℓ/capita*.day. Although similar figures will most likely be available for all major cities in the RSA it should be noted that Umgeni Water (1989) has developed a fairly sophisticated basis for determining the demand to be catered for by a water works.

2.3.2 DOMESTIC NEEDS

Domestic water needs can be determined from water metering data for any particular town or city. The influence of geographical factors as well as income, ground valuation, etc. in the supply area must then simultaneously be taken into account.

In this regard a design guide for services was drawn up by the Department of Community Development for the higher income groups in the RSA. These are generally not attainable. For developing communities the Department of Community Development, with the assistance of the CSIR, published a guideline which is widely used. These two guidelines have been condensed into one volume as *Guidelines for the Provision of Engineering Services and Amenities for Residential Town-*

TABLE 2.4
TYPICAL DOMESTIC WATER USAGE (DNH, 1994 AND IRC, 1981)

Type of water supply	Typical consumption (ℓ/capita.d)	Range	Notes on Table 2.3
Communal water point			(a) The type of sanitation systems affects the water consumption, particularly in the category "House connection".
. well or standpipe at considerable distance (>1 000 m)	7	5 - 10	
. well or standpipe at medium distance (250 - 1 000 m)	12	10 - 15	(b) The place where clothes are to be washed in the case of communal water points has an effect on the water consumption at that point. Most water will be used when people wash at the water point, less will be used if the people first carry water home and wash at home, and the least will be used when people still go to the traditional washing places at nearby streams.
. well nearby (<250 m)	20	15 - 25	
. standpipe nearby (<250 m)	30	20 - 50	
Yard connection	40	20 - 80	(c) The following additions should be made to this table: . a handpump should be considered as similar to a well, since additional effort is required to obtain the water; . a spring could be considered as similar to a standpipe, especially when it has been protected; . a climb of more than 60 m over a short distance should be considered as being similar to walking a distance of more than 1 000 m.
House connection			
. single tap	50	30 - 60	
. multiple taps	150	70 - 250	

ship Development by the CSIR on behalf of the Department of National Housing (DNH, 1994).

Water supply needs for developing communities can either be obtained from present consumption *per capita* data, or by studying existing water supply systems in the area, or by using the typical consumptions given in Table 2.4 (DNH, 1994).

Historical and projected total household water consumption figures for the major metropolitan areas in the RSA are given in *Management of the Water Resources of the Republic of South Africa* (1986). The average growth rate of approximately 5%, as well as the influence of drought restrictions and the rate of increase in consumption thereafter, is interesting to note.

Barnes *et al.* (1983) give water consumption figures for various purposes for a city of half a million people with a high living standard in a warm dry climate with an average annual winter rainfall of 600 mm. Similarly Fair *et al.* (1971) give approximate *per capita* daily uses in the United States.

The following water consumption rates have furthermore been established i.e.:

- average in-house use of water in major centres of the RSA and Namibia 160 ℓ/capita.day
- drinking water for an adult 3 ℓ/day
- domestic water use ranging from 50-300 ℓ/capita.day
- excluding industry and mining the consumption in the RWB service area for the 20-year period up to 1991: 160-249 l/capita.day

Based on the figures above, the domestic wa-

TABLE 2.5
DOMESTIC WATER REQUIREMENTS

Area and category	General requirement (ℓ/stand.day)	*Per capita* requirement (ℓ/*capita*.day)	
Urban areas		minimum	maximum
Upper	2 000	200	300
Middle	1 000	100	200
Lower	500	50	100
Rural areas			
drinking, cooking & personal hygiene		15	50
local (drinking water for an adult)		3	10
communal taps (400 m)		20	40
stand pipes (200 m)		30	50
private connections		100	
full services		150	

ter requirements shown in Tables 2.5 are suggested for design purposes.

2.3.2.1 Population projections

Population projections are a further tool to estimate the domestic water needs of an area to be served. For this purpose population census counts carried out by the Central Statistical Services that were done in 1904, 1911, 1921, 1936, 1946 and 1951, as well as during every decade since 1960, are extremely useful. Inter-census and other *ad hoc* surveys done in mid-decades are also available.

There has been a major population shift over the past few decades, from rural to urban areas. The design engineer could make use of methods of population counts such as aerial surveys and various other sources.

From census counts the current population can be obtained by assuming arithmetic growth from the last census count or geometric growth in rapidly expanding communities. These can be adjusted with births, deaths and migrations, the latter which may be derived from school attendance numbers.

Graphic and arithmetic methods are used to estimate growths. The graphic methods are fairly simple and rely on either comparison of similar towns by arithmetic or geometric progression for best-fit purposes.

Short- and long-term (10 - 50 years) projection methods are described in Barnes *et al.* (1983) as well as in Fair *et al.* (1971). Population distribution and areal densities are also given in the latter publication.

A mathematical method based on first-order biological growth in a confined space can be expressed by the following differential equation:

$$dy/dt = k \, . \, y \, (L - y)$$

where:

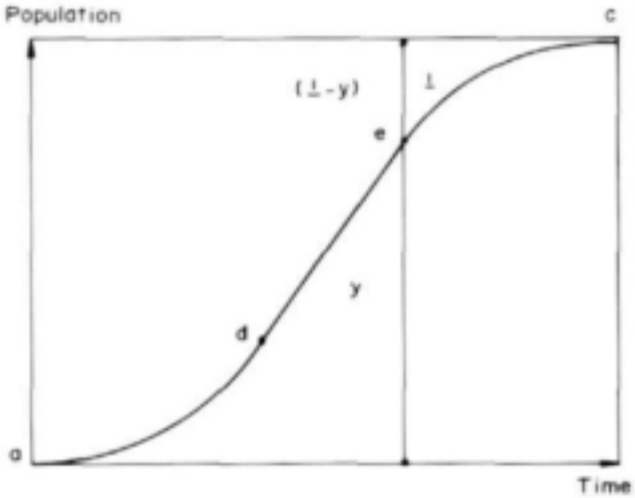


Figure 2.6
Population growth curve

y = population at time t
L = saturation value or maximum
k = growth constant

This equation yields the S-shaped curve of Verhulst which is depicted in Figure 2.6. The growth sequential phases are:

the first phase geometrical or logarithmic and $dy/dt = ky$, *(arc ad)*
the second phase is a straight line arithmetic $dy/dt = k$, *(arc de)*
the third phase is the first order progression $dy/dt = k(L - y)$ *(arc ec)*

The Verhulst projection is particularly applicable to a spatially confined city or conurbation where natural, gregarious forces prevail. Examples of the various projections are given in Chapter 19.

2.3.3 INDUSTRIAL WATER NEEDS

There has been substantial development in the water utilisation pattern in industry over the past few decades. This was brought about by a number of factors. The value of rational water usage in industry reflects on water intake quantities, on a systems approach in water use within the industry and on the quantity and quality of effluent discharged. The greater and increasing emphasis on pollution control caused critical appraisals of the water needs for the various processes in industry.

A number of guides to water management in industry have been produced under the aegis of the Water Research Commission. The NATSURV project of the Water Research Commission yielded most of the information given in Table 2.6. Equivalent figures of industrial consumption are also given by Metcalf and Eddy (1979) and Fair *et al.* (1971).

2.3.4 OTHER NEEDS

In addition to the water needs and requirements of the domestic and industrial sectors, the commercial sector of urban areas also has some water demand. Then there are needs for parks and cleansing of mid-town centres. Finally, some allowance has to be made for unaccounted-for water and losses from the distribution network. The need for cleansing of water service reservoirs, pipe mains and networks should also not be neglected.

The commercial sector demand can be estimated at 10% of the domestic needs (Umgeni Water, 1991). Further consumption and demand figures are also given in the *DCA 1983 - Blue Book*, Barnes *et al.* (1983) and Fair *et al.* (1971).

Estimated water requirements for non-domestic needs in developing communities, where field measurements cannot be taken, are given in Table 2.7 (DNH,1994).

These figures include a 20% allowance for water loss and wastage and assumes flush toilets.

In the case of dry toilets these consumptions can be reduced by approximately 10 ℓ per head per day. The allowance of 5ℓ/pupil per day is high at schools. Considering the introduction of feeding schemes at schools the figures given are not unrealistic.

Water demand values that apply to urban townships with flush sanitation are given in Table 2.8 (DNH, 1994).

Unaccounted-for water (UAW) such as water used for fire fighting, parks, certain construction activities, etc., and losses can be as much as 50% of flow into a network). Excessive losses can clearly not be countenanced in sizing a purification works. An average of 10 to 15% could be allowed for UAW and losses in a water network. A list of UAW and often unmetered uses is given in Table 2.9.

Rand Water has found that their raw water requirement is 4% higher than the potable water requirement. Water losses from their bulk supply system, including pipelines and reservoirs, are 2% (RWB, 1990).

TABLE 2.6
INDUSTRIAL WATER INTAKE PER UNIT OF PRODUCT PRODUCED

Process	Water intake per unit
Abattoirs	1,4 to 2,0 m^3/ large cattle uni
Breweries	5 to 8,8 ℓ/ℓ
Dairy products:	
Butter	1,3 to 1,5 m^3/t
Cheese	20 to 23 m^3/t
Condensed milk	3,5 to 4,4 m^3/t
Fruit juices	1,7 to 2,7 ℓ/ℓ
Ice cream	1,9 to 2,5 ℓ/ℓ
Milk powder	8,7 to 11,8 m^3/t
Pasteurised milk	0,8 to 3,0 ℓ/ℓ
Laundry	8 to 20 m^3/t
Metal finishing	0,03 to 1,25 m^3/m^2
Paper:	
Kraft paper and corrugation medium	10 to 20 m^3/t
Newsprint, magazine paper, folding box boarded	20 to 30 m^3/t
Food board	20 to 40 m^3/t
Wood-free printing paper	30 to 50 m^3/t
Poultry	15 to 30 ℓ/bird
Soft drinks	2,7 ℓ/ℓ
Sorghum and malt	2,5 ℓ/ℓ
Steel *	4 to 200 m^3/t
Sugar	30 to 100 m^3/t
Textiles:	
Cotton (wet processing)	9 to 53 ℓ/kg
Wool dyeing	115 to 152 ℓ/kg
Tanning	339 to 432 ℓ/hide
Thermal power stations *	2,5 to 8,7 ℓ/kWh
Wine-making	0,7 to 3,8 m^3/t grapes
Distillation	1,8 to 6,2 ℓ/ℓ absolute alcohol

* (Funke and Van Vuuren, 1979)
In the supply area of Rand Water, industrial consumption, including TRANSNET and mining, amounted to about 30% of the total flow for the 20-year period to 1991.

2.3.5 SIZING A PURIFICATION

TABLE 2.7: NON-DOMESTIC WATER REQUIREMENTS (DNH, 1994 AND IRC, 1981)

Category	**Typical water use**
Schools	
• day	15 - 20 ℓ/pupil per day
• boarding	90 - 140 ℓ/pupil per day
Clinics/hospitals (with laundry)	220 - 300 ℓ/bed per day
Community halls/restaurants	65 - 90 ℓ/seat per day
Mosques/churches	25 - 40 ℓ/visitor per day
Cinemas	10 - 15 ℓ/seat per day
Offices	25 - 40 ℓ/person per day
Railway and bus stations	15 - 20 ℓ/user per day
Agriculture	
• cattle	25 - 35 ℓ/head per day
• horses and mules	20 - 25 ℓ/head per day
• sheep	15 - 25 ℓ/head per day
• pigs	10 - 15 ℓ/head per day
• chickens	15 - 25 ℓ/100 per day

TABLE 2.8: WATER DEMAND

Category	**Type of development**	**Unit**	**Annual average water demand(ℓ/d)**
1	Dwelling houses (residential Zone 1)	Erf area for dwelling house	See Figure 2.6 for erven not exceeding 2 000 m^2. For erven 2 000 m^2 base demand on local conditions
2	Low-rise multiple dwelling unit buildings (Residential Zones II & III)	Dwelling	Upper limit 1 000[a] Lower limit 600[a]
3	High-rise multiple dwelling unit buildings (Residential Zone IV)	Dwelling	Upper limit 700[a] Lower limit 450[a]
4	Offices and shops	100 m^2 of gross floor area[b]	400
5	Government and Municipal	100 m^2 of gross floor area[b]	400
6	Clinic	100 m^2 of gross floor area[b]	500
7	Church	Erf	2 000
8	Hostels	Occupants	150
9	Developed parks	Hectare of erf area	≤2 ha 15 kℓ[c][d] >2 ha ≤ 10 ha 12,5 kℓ[c][d] >10 ha 10 kℓ[c][d]
10	Day schools	Hectare of erf area	As for developed parks[d]
11	Boarding schools	Hectare of erf area plus boarders	As for developed parks plus 150 ℓ/boarder
12	Sportsground	Hectare of erf area	As for developed parks
13	Creche	Hectare of erf area	As for developed parks plus 75 ℓ/person
14	Public open spaces		Nil

(a) Water demand includes garden watering of all common areas outside the limits of the buildings.

(b) Gross floor area obtained using applicable Floor Space Ratio from the Town planning scheme.

(c) Demand for developed parks to be considered as drawn over 6 hours on any particular day in order to obtain the peak demand.

(d) Where the designer anticipates the development of parks and sportsgrounds to be of a high standard, e.g. 25 mm of water applied per week, the annual average water demand should be taken as: 2 ha 50kℓ[c]; >2 ha 40kℓ[c]; >10 ha 30kℓ[c]

Table 2.9
UAW and Often Unmetered Uses

Use	
• Pipe distribution system losses (including pipe breakages, etc.)	
• fire fighting and certain city construction activities	
• cleansing of service reservoirs	10 - 15%
• meter errors	
• cleansing of mid town centres	
• water purification works losses (i.e. filter wash and clarifier sludge water losses, process water, etc.)	1 - 4%

WORKS

The design period is usually between 10 and 50 years. The longer period generally being applicable to dams, tunnels, canals, etc. that are difficult to enlarge. Scheme components that can more readily be extended or duplicated, such as pumps, pipelines and water purification works, can have shorter design periods (10 - 30 yrs). Similar figures from Fair *et al.* (1971) are given in Table 2.10.

Rand Water has found that the annual average daily demand is exceeded by 30% for the peak day, by 23% for the peak 7 days and by 15% for the peak month. For smaller undertakings these ratios can be expected to be higher, i.e. the average daily demand being exceeded by 75% for communities under 100 000.

When considering all the different needs to be satisfied the question to be answered by the designer is what the capacity of the purification plant must be. It must be sized to provide in the peak requirements of all consumer sectors via the distribution network including reservoirs. Different methods (DNH, 1994) can be adopted to arrive at this peak supply rate. WADISO software was developed (1993) to optimise distribution system (pipelines and reservoirs) and water purification works capacities.

Table 2.10
Design Periods for Water Structures

Type of structure	Special characteristics	Design period years
• Large dams & conduits	Hard & costly to enlarge	25 - 50
• Wells, distribution systems and purification plants	Easy to extend • When growth and interest rates are low	20 - 25
	• When growth and interest rates are high	10 - 15
• Pipes more than 300 mm in diameter	Replacement of smaller pipes is more costly in long run	20 - 25
• Laterals and secondary mains less than 300 mm in diameter	Requirements may change fast in limited areas	Full development

TABLE 2.11
STEP METHOD OF SIZING A WORKS

Action steps	Refer to	
	Paragraph	Table
For the area to be supplied • gather as much as possible historical and future growth and consumption data and • determine the:		
1. number and types of industries including their potential growth rate		
2. number and growth rate of people (both residents and commuters)	2.3.2.1	
3. the design period	2.3.5	2.10
4. the water needs of industries	2.3.3	2.6
5. domestic water needs	2.3.2	2.4; 2.5
6. other needs	2.3.4	2.7; 2.8
7. an UAW value	2.3.4	2.9
8. average daily water requirements of consumers i.e. (4) + (5) + (6) + (7 excluding water purification works losses)		
9. peak flow to be delivered after assuming peak flows required by the various consumer sectors and optimising the distribution network and reservoir storage capacities	2.3.5	
10. incoming flow to the works i.e. (8) + purification works losses.	2.3.4	2.10
11. *Finally decide on hydraulic overload to be allowed for the works*	2.3.4	

Allowance must be made for chemical process water as well as for losses such as filter wash and sedimentation sludge waters.

Extensions to purification works invariable do not get done timeously. Hence the allowance for a hydraulic over-design or excess throughput of approximately 25% made by some designers. Similarly a plant can be sized for a, say, 20-h operation day. This is, however, the designer's choice.

A step-by-step procedure has been set out in Table 2.11 to assist the designer in the process of arriving at the average design flow to be delivered by the works as well as the peak design flow.

2.4 WATER QUALITY

In the natural environment there is no pure water available for general use, as all water, including rainwater, contains some impurities. The impurities are commonly in the form of dissolved solids or gasses, suspended solids and micro-organisms.

The nature and degree of purification of water are obviously based on quality consideration. It is thus important that the quality of the source(s) and the quality requirements for the particular use(s) be known.

For domestic and many industrial purposes, the health aspect of water as dealt with in this text is most important.

2.4.1 QUALITY OF WATER RESOURCES

Over and above the indicative maps on surface and underground water qualities given in this chapter, the Department of Water Affairs and Forestry through its Directorate of Hydrology and the Institute for Water Quality Studies, monitors the quality of South Africa's surface water resources. The objectives of the national monitoring programme which was started in the late 1960s, were essentially to provide:

- ambient water quality data and
- interpretative information for water resource planning, management and pollution control.

This programme primarily focuses on inorganic chemical quality determinands such as the major ions (calcium, magnesium, sodium, potassium, sulphate, chloride and alkalinity) and pH, conductivity and fluoride and plant nutrients such as inorganic phosphorus, nitrate and ammonium. A synthesis of the data is often also synoptically presented in the form of a water quality map giving an overall catchment perspective. These data are also processed to evaluate changes with time. Further monitoring of organic pollution and other toxic matter is being done on an *ad hoc* basis in certain catchment areas i.e. total phosphorus, organic nitrogen and trace metals.

All these data are stored on the Department of Water Affairs and Forestry's Hydrological Information System's (HIS) data bank and can be accessed via the Computing Centre for Water Research (CCWR) and Internet. The total concentration and composition of dissolved salts for the Vaal River catchment (DWA, 1986), T.R. 131 is available on this department's Geographic Information System (GIS). Similarly the TDS, elevated fluorides, nitrates and the hydro-chemical water type of classification will soon be available on a National Hydrological Groundwater Quality Map of the GIS.

Table 2.12 shows surface water quality for the surface water drainage regions defined by the Department of Water Affairs and Forestry.

2.4.2 QUALITY REQUIREMENTS (CONSUMERS)

Water quality requirements for domestic and industrial purposes are ever-changing. This is probably due to changes in conditions as well as perceptions and needs.

The importance of water as a vehicle for the potential spread of disease is the main concern in water quality control. It has often been stated by world health authorities that disinfection of water alone has saved more lives than the whole of the medical industry. However, certain quality parameters may also be of concern, for example the aesthetics of water (clarity and taste) and the corrosivity or scale-forming potential.

Many studies have been conducted on the effects of water quality on human and animal health, its acceptability and the useful life of water distribution systems. Based on these studies guidelines have been proposed for quality standards with which domestic water supplies should comply. These in turn should rather be considered as long-term

TABLE 2.12
QUALITY OF SURFACE RAW WATER IN SOUTH AFRICA

Drainage region	**Water quality variable**						
	TDS	**Ca**	**TAL (mg/ℓ as CaCO$_3$)**	**Secchi (m)**	**Chl *a* (Δg/ℓ)**	**Hardness (mg/ℓ as CaCO$_3$)**	**pH (pH units)**
A	364	34	133	1.4	34	157	7.7 - 8.5
B	205	22	92	2.5	15	103	7.5 - 8.3
C	551	49	168	0.8	12	252	7.7 - 8.5
D	276	31	144	-	-	138	7.6 - 8.4
E	199	16	46	-	-	83	6.9 - 7.9
G	229	10	39	2.3	7	64	7.0 - 8.0
H	135	8	21	-	-	51	6.5 - 7.9
J	472	44	164	-	-	188	7.5 - 8.4
K	228	9	25	-	-	51	6.1 - 7.6
L	833	42	69	-	-	260	7.3 - 8.0
M	135	4	12	-	-	39	6.6 - 7.7
N	1204	56	318	-	-	456	8.0 - 8.7
P	1784	59	177	-	-	527	7.6 - 8.5
Q	1081	54	393	-	-	327	8.1 - 8.7
R	310	21	100	1.2	1	108	7.4 - 8.3
S	382	31	196	-	-	166	7.7 - 8.6
T	92	9	45	-	-	43	7.2 - 8.1
U	123	8	48	-	8	45	7.3 - 8.1
V	119	13	56	-	-	62	7.3 - 8.2
W	251	18	88	-	-	92	7.5 - 8.4
X	154	14	72	-	-	71	7.4 - 8.3

Note: Tabulated values are the 90th percentile for the 50th regional percentile value except for pH where the 10th and 90th percentile values are given.

goals than rigid standards.

In the RSA three criteria levels or guideline values have been proposed namely:

- recommended or working limit within which drinking water is considered to be safe for the consumption of a lifetime, or maximum level for no risk,
- maximum allowable limit or the maximum level for insignificant risk,
- crisis limit or the maximum level for low risk.

This three-tier system allows health authorities to approve water of a poorer quality under certain very specific conditions. Kempster *et al.* (1989) give an overall picture of the present world opinion requirements and desirable limits for 15 water quality criteria i.e. drinking water, river/dam water (protection of fish, etc.), livestock watering, irrigation and industrial uses.

In this regard as part of the Department of Water Affairs and Forestry's recently defined water quality management responsibilities, four volumes of *South African Water Quality Guidelines* based on the above-mentioned three-tier system, were published in 1993 (DWA&F, 1993), for domestic, recreational, industrial and agricultural use. Two further volumes relating to the natural and the coastal and marine environment are being prepared. Each volume also contains a summary of international water quality guidelines.

2.4.2.1 Domestic

Human health may be affected by ingesting water directly or in food, by using it in personal hygiene or for agriculture, industry or recreation.

The following water quality aspects are of general concern for domestic water supplies:

- water must not pose a health risk due to microbiological contamination,
- water must not pose a health risk due to chemical contamination,
- water should be aesthetically acceptable,
- water should not have economically detrimental effects on either the distribution system nor on consumers equipment in terms of corrosivity, scaling, hardness or sediments.

Unacceptable **microbiological** quality of water poses by far the greatest risk to the consumer. Biological hazards in the water in the form of disease-causing bacteria, viruses, protozoa and a variety of parasites and worms are responsible for a major portion of the diarrhoeal morbidity and mortality, particularly among infants.

There are also a large number of chemical compounds, both natural and man-made, which affect humans such as:

- nitrates and fluorides where excessive concentrations cause health effects, i.e. methaemoglobinaemia (blue baby disease in infants) and skeletal fluorosis (including mottled teeth) respectively,
- heavy metals, e.g. mercury and lead which interfere with the nervous system or upset the human metabolism in various ways,
- several organic micropollutants which are known carcinogens (cancer-causing substances) and their presence in drinking water, thus increasing the risk of tumour formation.

Aesthetic quality of the water, such as taste, colour and odour, can be affected by natural source water quality or man-made contaminants.

One of the constituents that affects the taste of water is the salinity of water. Water with a total dissolved salt (TDS) concentration of up to 450 mg/ℓ has a fresh taste. Increasing concentrations up to 1 000 mg/ℓ will taste slightly salty to the unaccustomed user but will not have any serious health effects. Long-term health effects may only become significant at levels in excess of 2 000 mg/ℓ.

Odour almost always indicates some form of pollution of the source water or malfunction during water purification or distribution. Since odour sensitivity is highly variable among individuals within a population, it would be realistic to provide water free of odours not objectionable to the majority (i.e. ± 90%) of the population.

There are no statutory standards for drinking-water quality in South Africa. The quality requirements for water for domestic and industrial purposes are indicated in general in a specification on water for domestic purposes (SABS, 1984). The properties specified by the SABS, WHO, EPA and EEC are indicated in Table 2.13.

The SABS specification which is a well-known guideline for drinking-water quality contains two categories of quality, namely recommended and maximum limits of sub-

Table 2.13
Domestic Water Properties Specified by the SABS, WHO, EPA (US) and EEC

Summary Comparison of US Primary Regulations with EEC and WHO Guidelines

Substance	US maximum contaminant level*	EEC maximum admissible concentration+	WHO guideline
Inorganic	mg/ℓ	mg/ℓ	mg/ℓ
Arsenic	0,05	0,05	0,05
Barium	1,0	0,1	NS
Cadmium	0,01	0,005	0,005
Chromium	0,05	0,05	0,05
Fluoride	4,0	NS	1,5
Lead	0,05	0,05	0,05
Mercury	0,002	0,001	0,001
Nitrate	10,0(N)	50,0(NO_3)	10,0(N)
Selenium	0,01	0,01	0,01
Silver	0,05	0,01	NS
Organics	µg/ℓ	µg/ℓ	µg/ℓ
2,4-D	100	NS	1
Endrin	0,2	NS	NS
Lindane	0,4	NS	NS
Methoxychlor	100	NS	1
Pesticides (total)	NS	5	NS
Toxaphene	5	NS	NS
2,4,5-TP silvex	10	NS	NS
Trihalomethanes	100	1	30($CHCl_3$ only)
Volatile organic chemicals	µg/ℓ	µg/ℓ	µg/ℓ
Benzene	5	NS	10
Carbon tetrachloride	5	NS	3
1,1-Dichloroethylene	7	NS	3
1,2-Dichloroethane	5	NS	10
para-Dichlorobenzene	75	NS	NS
1,1,1-Trichloroethane	200	NS	NS
Trichloroethylene	5	NS	30
Vinyl chloride	2	NS	NS
Microbials			
Coliforms - organisms/100 mℓ	<1	0	0
Turbidity - NTU	1-5	0-4	<1
Radionuclides			
Beta particle and photon activity	4 mrem	NS	1,0 Bq/ℓ
Gross alpha particle activity	15 pCi/ℓ (0,56 Bq/ℓ)	NS	0,1 Bq/ℓ
Radium-226 + radium-228	5 pCi/ℓ(0,19 Bq/ℓ)	NS	NS

* Enforceable
\+ Non-enforceable
NS = Not specified

NTU = Nephelometric turbidity units
1 Ci = 3,7 x 10^{10} Bq

TABLE 2.13 (CONTINUED)

SUMMARY COMPARISON OF US SECONDARY REGULATIONS WITH EEC AND WHO GUIDELINES

Substance	**US secondary maximum contaminant level***	**EEC**		**WHO guideline value**
		Guide level*	**Maximum admissible concentration**	
Chloride	250 mg/ℓ	25 mg/ℓ	NS	250 mg/ℓ
Colour	15 CU	1 mg Pt-Co/ℓ	20 mg Pt-Co/ℓ	15 CU
Copper	1 mg/ℓ	100 µg at treatment plant: 3 000 µg after 12 h in piping	NS	1,0 mg/ℓ
Corrosivity	non-corrosive		Water should not be aggressive.	
Fluoride	2 mg/ℓ		Varies according to temperature, e.g. 1,5 mg/ℓ (8-10°C)	1,5 mg/ℓ
Foaming agents	0,5 mg/ℓ		NS	NS
Iron	0,3 mg/ℓ	0,05 mg/ℓ	0,3 mg/ℓ	0,3 mg/ℓ
Manganese	0,05 mg/ℓ	0,02 mg/ℓ	0,05 mg/ℓ	0,1 mg/ℓ
Odour	3 TON	0 dilution number	2 dilution number at 12°C	
pH	6,5-8,5	6,5-8,5	NS	6,5-8,5
Sulphate	250 mg/ℓ	25 mg/ℓ	NS	400 mg/ℓ
Total dissolved solids	500 mg/ℓ	NS	NS	1 000 mg/ℓ
Zinc	5 mg/ℓ	100 µg at treatment plant: 5 000 µg after 12 h in piping	NS	5,0 mg/ℓ

* Non-enforceable; S = Not specified; CU= Colour units; TON = Threshold odour number

SABS SPECIFICATION FOR WATER FOR DOMESTIC SUPPLIES (SABS, 1984)

Determinand	**Unit**	**Recommended maximum limit**	**Maximum allowable limit**
Colour	mg/ℓ Pt	20	NS
Odour and taste	Shall not be objectionable		
Turbidity	NTU	1	5
pH	pH unit	6-9	5,5-9,5
Conductivity	mS/m	70	300
Macro-determinands	mg/ℓ		
Hardness, total	$CaCO_3$	20-300	650
Magnesium	Mg	70	100
Sodium	Na	100	400
Chloride	Cl	250	600
Sulphate	SO_4	200	600
Nitrate and nitrite	N	6	10
Fluoride	F	1	1,5
Zinc	Zn	1	5
Micro-determinands	µg/ℓ		
Arsenic	As	100	300
Cadmium	Cd	10	20
Copper	Cu	500	1 000
Cyanide	Cn	200	300
Iron	Fe	100	1 000
Lead	Pb	50	100
Manganese	Mn	50	1 000
Mercury	Hg	5	10
Phenolic compounds	Phenol	5	10
Selenium	Se	20	50
Bacteriological requirements			
Standard plate count	per 1 mℓ	100	NS
Total coliform	per 100 mℓ	0	5
Faecal coliform	per 100 mℓ	0	0
Radioactivity	If present shall be within the limits laid down by the International Commission for Radiological Protection		

NS = Not specified NTU = Nephelometric turbidity units

TABLE 2.14
SUMMARY OF WATER QUALITY GUIDELINES FOR DOMESTIC USE

Water quality constituent	**Water quality guideline for domestic use**
Physical and organoleptic properties	
• Electrical conductivity	0 - 70 mS/m 0 - 450 mg/ℓ TDS
• Odour	TON = 1
• pH	6.0 - 9.0
• Turbidity	0 - 1 NTU
Physico-chemical constituents	
• Aluminium	0 - 0.15 mg/ℓ
• Dissolved organic carbon (DOC)	0 - 5 mg/ℓ C
• Fluoride	0 - 1.0 mg/ℓ
• Iron	0 - 0.1 mg/ℓ
• Manganese	0 - 0.05 mg/ℓ
• Mercury heavy metals	0 - 0.005 mg/ℓ
• Nitrate and nitrite	0 - 6 mg/ℓ N
Biological and microbiological constituents	
• Algae	0 - 5 mg/ℓ chlorophyll *a*
• Coliphages	< 1 per 100 mℓ
• Enteric viruses	< 1 $TCID_{50}$/10 ℓ
• Faecal coliforms/*E. coli*	0 per 100 mℓ
• Protozoan parasites	V 1 *Cardia* cyst/10 ℓ
Process-related constituents/properties and problems	
• Corrosion and scaling	
• Hardness	<100 mg/ℓ $CaCO_3$ where possible
• Sediments	No visible sediments

stances contained in the water. The brief list of properties given, indicates that the specification really applies to fairly unpolluted, natural, surface waters.

A proposed three-tier drinking-water quality criteria given by Aucamp and Viviers (1990) (an update of Pieterse, 1989), is under consideration for application in South Africa. A summary of the *South African Water Quality Guidelines* for domestic use (DWA&F; 1993) is given in Table 2.14.

In the absence of statutory drinking-water standards for the RSA, Rand Water (1994) and Umgeni Water have drawn up their own guideline standards.

2.4.2.2 Industrial

Quality requirements for industrial purposes have evolved with the changes that have taken place in industry itself. With greater sophistication in industry there has been an

Table 2.15
Water Quality Requirements for Some Industries (Funke, 1969)

Property	Textiles	Viscose pulp	Fine paper	Plastics*
Colour (platinum)	0 - 20	5	5	2
Hardness ($CaCO_3$)	0 - 20	8	100	NS
Alkalinity	NS	50	75	NS
Total solids	NS	100	250	200
Turbidity (SiO_2)	3 - 27	5	10	2
Fe	0,1 - 1,0	0,0005 - 0,05	0,01	0,02
Mn	0,05 - 1,0	0 - 05	0,05	0,02
Fe + Mn	0,05 - 1,0	NS	0,05	0,02

* (uncoloured);   NS = Not specified

Table 2.16
Summary of Target Guidelines for Industrial Water

Constituent*	Industry					
	Leather & tanning	Power generation**	Iron and steel	Pulp and paper	Petro-chemical**	Textile
pH	6.0 - 8.0	7.0 - 9.0 7.0 - 9.0	7.0 - 9.0	6.5 - 8.5	7.5 - 8.5 7.5 - 8.5	7.0 - 8.5
Electrical conductivity	10 - 70	0 - 20 10 - 70	0 - 100	10 - 70	0 - 30 10 - 70	10 - 70
Suspended solids	0 - 5	0 - 10 0 - 10	0 - 5	0 - 5	0 - 5 0 - 5	0 - 5
Total hardness	0 - 250	0 - 100 0 - 150	0 - 200	0 - 75	0 - 120 0 - 250	0 - 25
Alkalinity	0 - 150	0 - 50 0 - 50	0 - 100	0 - 100	0 - 100 0 - 150	0 - 100
Sulphate	0 - 200	0 - 30 0 - 50	0 - 400	0 - 200	0 - 100 0 - 200	0 - 250
Chloride	NS	0 - 20 0 - 20	0 - 20	0 - 20	0 - 20 0 - 20	NS
Iron	0.0 - 0.2	0.0 - 0.2 0.0 - 0.2	0.0 - 0.2	0.0 - 0.1	0.0 - 0.5 0.0 - 0.5	0.0 - 0.2
Manganese	0.0 - 0.1	0.0 - 0.1 0.0 - 0.1	NS	0.0 - 0.1	0.0 - 0.2 0.0 - 0.2	0.0 - 0.1
Chemical oxygen demand (COD)	0 - 20	0 - 20 0 - 20	0 - 30	0 - 10	0 - 10 0 - 10	0 - 10
Further guideline information can be found in:	Tables 5.3 - 5.11	Tables 6.3 - 6.22	Tables 7.3 - 7.11	Tables 8.3 - 8.12	Tables 9.3 - 9.22	Tables 10.4 - 10.12

* With the exception of pH (pH units) and electrical conductivity (mS/m), all values are expressed as mg/ℓ.

** Target guidelines ranges for steam generation (upper value) and cooling/other purposes (lower value) are given separately for these industries.

NS = Not specified.

increase in the range of quality requirements, if not the purity of water. As the ability to produce improved quality has advanced, so have the demands for purity. An indication is obtained from Table 2.15 of the range of water quality requirements in industry (Funke, 1969).

A summary of the *South African Water Quality Guidelines for Industrial Use* (DWA&F; 1993) is given in Table 2.16.

2.5 DESIGN

Water quality requirements play a key role in the provision and thus design of a water purification works. Quality requirements also have economic implications. Water can be used without purification, but for hygienic and health purposes some treatment such as disinfection is essential for surface water sources. The design of a water purification works should reflect the water quality requirements. Once having decided the qualitative aspect and thus the process considerations, the quantity of the purified water required is predominant.

TABLE 2.17
WATER RESOURCE, QUANTITY, QUALITY AND DESIGN

Action steps	**Refer to**	
	Paragraph	**Table**
1. Establish the area to be served	-	-
2. Provisionally size the plant and group the main consumer sectors quality- and quantity-wise	2.3.5	2.11
3. Investigate and list available water sources that may be utilised: - gather all possible records - quality - quantity - determine assured yields - determine quality(s) of water to be supplied. More than one only where feasible.	2.4.2	
4. Optimise the cost of: - supply from different sources - providing either one or more qualities of water (including distribution costs) where/if feasible		
5. Finalise/determine number of purification works and sources to provide in needs		
6. Finally size works	2.3.5	2.11

Clearly the quantity of water purified again has economic significance as it affects the size of the works and the dimensions of the process units. In addition to the needs and requirements for various purposes there are the in-process needs of the water works. These needs include preparation, dilution and dosing of chemicals, including coagulants, desludging needs of sedimentation, flotation or clarification and filter cleansing.

A further major consideration is the production rate required from the water works. A number of factors need to be considered when determining this rate. The earliest works were operated on a batch or fill and draw bases. This can still be used in certain circumstances, but would not be considered for populations exceeding 100. Hence continuous operating works are the rule and it is only a question of the rate that has to be decided.

To assist the designer in using the water resources and quality parameters, a rough step-by-step guide is given in Table 2.17.

In considering alternative sources, due cognizance of the importance of intakes of draw-off localities from dams, rivers, etc. must be taken with regard to raw water quality with say dissolved iron and manganese that may complicate the treatment process. Grit, sand and other suspended and non-suspended particles greater than approximately 0,07 mm diameter should be removed from the raw water prior to entry into the works.

Dual distribution systems are known to have been supplied for smaller communities at coastal areas and mines where the potable water is supplied through one piping system, and toilet flushing and gardening water via the other system.

If the general rule of a 48-hour storage capacity for pumped systems and 24 hours for a gravity system is applied, then it is acceptable to have a 24-hour average daily demand capacity to be satisfied by the water purification works.

3

Process Selection

J Haarhoff

Water purification works design begins with the selection of an appropriate combination of treatment processes. This implies that the process designer must have a thorough understanding and knowledge of the vast number of processes available, as well as knowing the performance limits of each process. At present, it is not yet possible to approach this problem in a strictly quantitative manner, despite the recent laudable work (e.g. Patrick, 1993) to place process selection on a systematic, quantitative footing. The following factors contribute to the fact that process selection is still mainly seen as an art rather than an exact science:

- A treatment process usually does not fail catastrophically when it is overloaded or wrongly applied, but it is not as efficient as it should be or gradually diminishes in efficiency. Unlike a beam or other structural member, for example, where the limits of serviceability can be precisely determined by test or calculation, the limits of application of a treatment process are diffuse and difficult to quantify.
- Processes must be operated and maintained, and the experienced process designer will make a choice between alternative processes on the basis of water quality and process efficiency, *as well as* ease of operation, required skill level of operators, and the costs of construction and operation. These factors are all difficult to quantify and compare in an algorithmic way.
- If a process *removes* pollutants (this extractive process is defined as *purification*), different treatment processes could simply be concatenated until all the unwanted contaminants are removed. Some processes, however, change the nature of the water by the addition of chemicals (defined as *treatment*). This complicates process selection by making it an iterative process, whereby pollutants are not merely extracted, but the water chemistry is altered.

The importance of proper process selection cannot be overemphasised. A selected process train cannot be changed, once constructed. It is, therefore, of the utmost importance to allocate an adequate proportion of the project budget and the available time to the thorough analysis of the process options.

This chapter follows the decision-tree shown schematically in Figure 3.1.

3.1 CHARACTERISATION OF RAW WATER

Rational process selection requires reliable and representative samples of the raw water source, for determination of its quality, and for laboratory-scale process evaluation. The problem of *reliable* and *representative* samples is addressed first.

Water quality from any source continuously changes due to flow variations, sporadic pollution, diurnal and seasonal changes, changes in upstream land use, and impoundment. Some sources, like a stream

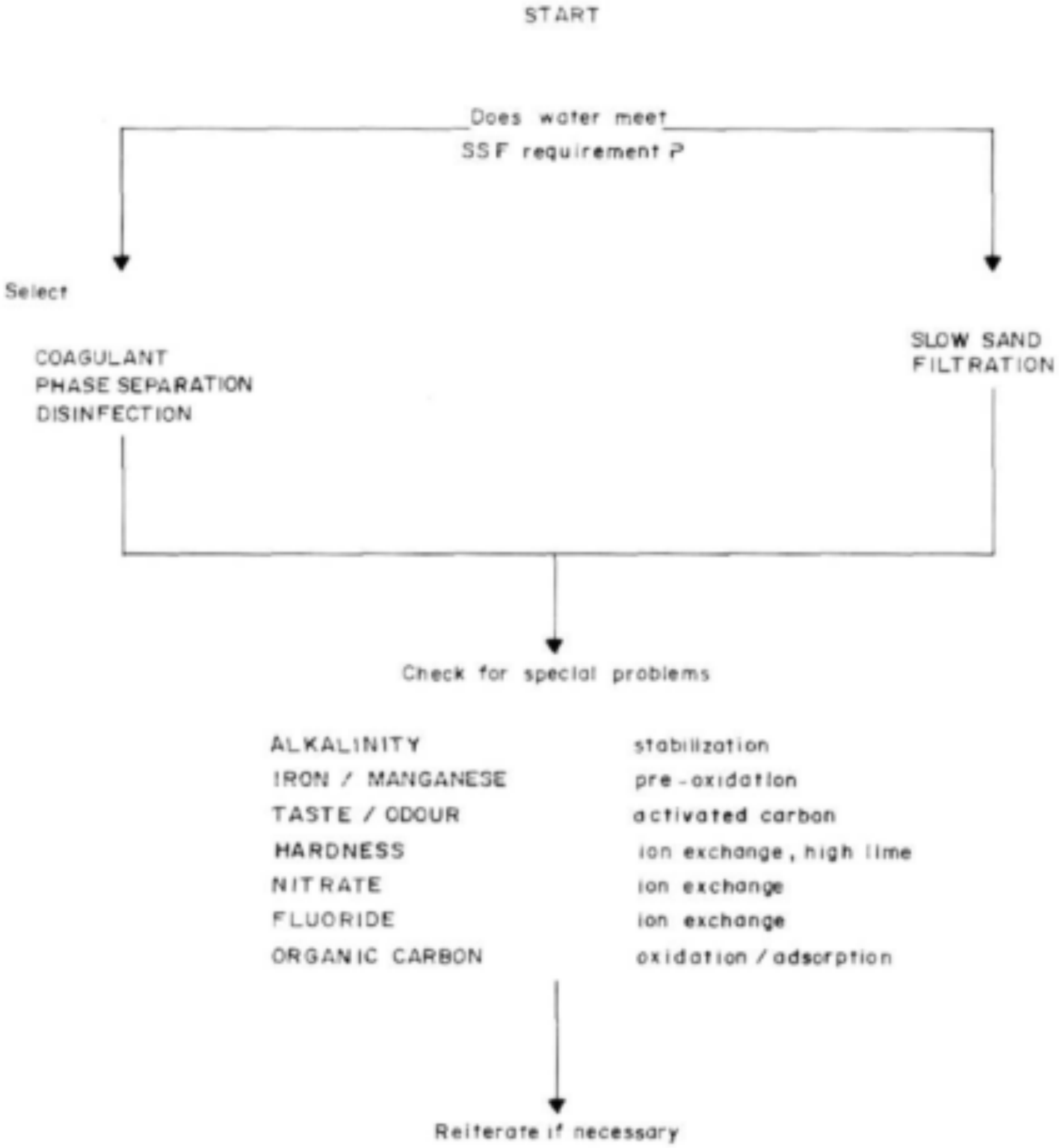
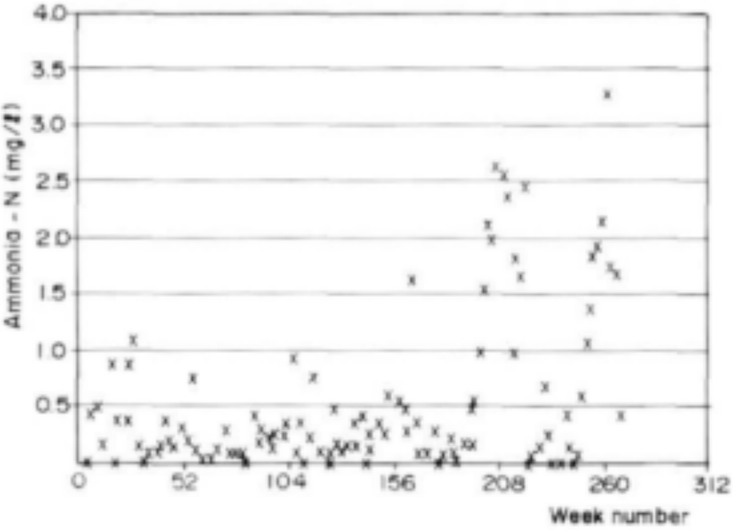


Figure 3.1
Process selection decision tree

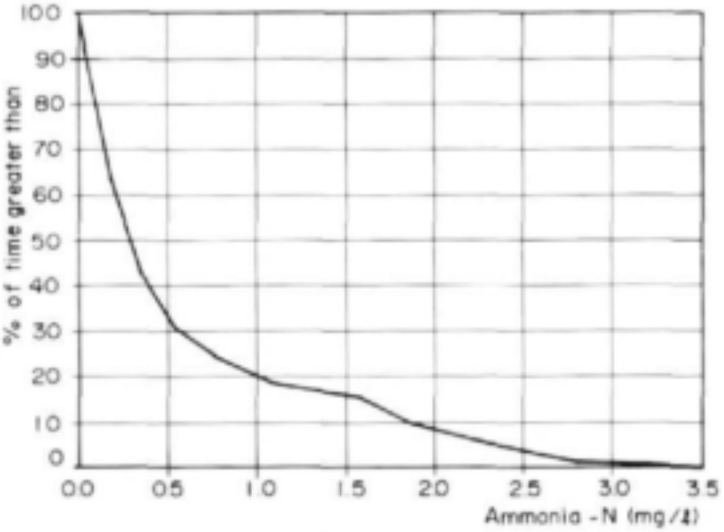
draining a small urban catchment area, will have very high variability, while others, such as groundwater abstracted from a deep unconfined aquifer, will have almost no variability. The vast majority of South African sources are surface water subject to considerable flow fluctuations, which generally makes water quality highly variable.

A single grab sample will therefore only reveal the water quality at that time, but will not provide sufficient data to assess the limits of variability for which the process designer must cater. Multiple samples are required, taken at frequent intervals over a period long enough to cover the most important quality variations. This could require years of sampling, which presents a serious problem to the process designer if no historical quality records in the vicinity of the source are available. In most cases the designer, fortunately, can resort to historical records. In the simplest case, the designer can draw on the water quality records of an existing treatment plant and the performance of the existing process units. The designer can also resort to the water quality records of the Department of Water Affairs and Forestry (DWAF), which cover most South African rivers through numerous sampling stations, of which 265 were in use during 1989. These records are readily available in computer-processed format and cover 21 different constituents (DWAF, 1990 & 1991).

Although a time-series plot of a water quality parameter rapidly reveals the maximum and minimum values, it is cumbersome to work with. It is more useful to the process designer to transform the time series plot into a probability plot, as illustrated in Figure 3.2a and 3.2b. From the probability plot, the designer can pick, for example, the 98th percentile (the parameter was below this value for 98% of the time in the past) for design purposes.



(a) Time series plot of water quality data (Ammonia at Rietvlei dam, 1988-1993)



(b) Probability plot of the data

Figure 3.2
Water quality data

It is possible to derive some general characteristics of water quality in different parts of South Africa, e.g. the mineralised, eutrophic waters downstream of the PWV complex, the soft, coloured waters from the Southern Cape, or water from dolomitic compartments. Such data are published by the Department of Water Affairs (1990). While these values are useful to alert the designer to the typical problems to be aware of in that region, they are inadequate to form the basis for process design:

TABLE 3.1
SUMMARY OF WATER QUALITY PARAMETERS TO BE ROUTINELY TESTED FOR PROCESS SELECTION

Quality parameter	Significance
Alkalinity	Used for calculating CCPP Influences coagulant choice Determines buffer capacity
Ammonia	Indicates potential sewage pollution Exerts large chlorine demand
Calcium hardness	Part of total hardness Used for calculating CCPP
Colour	Influences aesthetic appearance
Chlorophyll	Influences phase separation choice Determines suitability of SSF
Conductivity	Used for calculating CCPP
Fluoride	Affects teeth
Iron	Causes red stains and deposits
Magnesium hardness	Used for calculating CCPP
Manganese	Causes black stains and deposits
Nitrate	Dangerous for infants
Organic carbon	Impedes disinfection efficiency Increases potential for aftergrowth
pH	Used for calculating CCPP
Sulphate	Stomach disorders
Temperature	Used for calculating CCPP
Turbidity	Influences phase separation choice Influences aesthetic appearance Impedes disinfection efficiency

- Surface water quality has high spatial and temporal variability, and is subject to local changes by point polluters or impoundment. Groundwater has better temporal consistency, but quality differences amongst isolated aquifers could be significant.
- Process design is heavily dependent on such parameters as turbidity and algal concentration, but these parameters are normally excluded from general surveys due to their critical dependence on point and time of sampling, and upstream conditions.

Process designers are well advised to insist on appropriate and representative water samples of the specific raw water source being considered, and not to rely on general water quality trends only. Table 3.1 summarises the water quality parameters normally required for process design purposes.

3.2 SLOW SAND FILTRATION

Slow sand filtration shares many fundamental and design principles with rapid sand filtration, and both processes are therefore covered in Chapter 12. From a *process selection* viewpoint, however, slow sand filtration offers the unique benefit of a single-step treatment process, requiring no chemical coagulants (having the ability to remove uncoagulated particles through a combination of fine sand and low filtration rate) or disinfectants (having the ability to remove bacteria and viruses). It has further advantages in the form of limited requirements in terms of construction, maintenance and operational skills. The first option to be considered is whether the raw water source meets the relatively stringent quality requirements for slow sand filtration.

Quality requirements for slow sand filtration are usually expressed in terms of turbidity, a readily available parameter. The shortcomings of turbidity as a predictor of filter cycle length have been discussed at length elsewhere (Cleasby, 1991). The essential problem is that filter blockage is determined by the *particle volume*, and that different types of solids have different turbidity-to-volume ratios. Another problem, especially relevant to South Africa where sunlight and nutrients are abundant, is that the particle loading is not only derived from the raw water source, but also from the algal growth in the head space above the filters. The important quality parameters for considering slow sand filtration are shown in Table 3.2, taken from Cleasby (1991).

If a raw water does not meet the above quality requirements in terms of inorganic turbidity, pretreatment could be considered. There are a number of options, for example riverbank filtration, plain sedimentation, pebble matrix or horizontal roughing filtration. These processes have very specific requirements in terms of suitable geology, labour availability and labour costs and their design details fall beyond the scope of this book. Readers may refer to books listed in the bibliography. These processes will reduce short-term turbidity of up to 1 000 NTU to a

TABLE 3.2
IDEAL RAW WATER QUALITY FOR SLOW SAND FILTRATION WITHOUT PRETREATMENT (FROM CLEASBY, 1991)

Parameter	Upper limit	Remarks
Turbidity	< 5 NTU	This is a conservative guideline. Others have suggested values up to 50 NTU for prolonged periods, with the further relaxation that it could be as high as 120 NTU if limited to 1 or 2 days per year.
Chlorophyll *a*	< 5 mg/m^3	This value relates to the raw water concentration, and does not differentiate between different algal genera.
Iron	< 0,3 g/m^3	To prevent iron precipitation and subsequent filter clogging.
Manganese	< 0,05 g/m^3	To prevent manganese precipitation and subsequent filter clogging.
True colour	< 25 g/m^3 Pt	True colour (remaining colour after membrane filtration) is only reduced by about 25% by slow sand filtration. Based on proposed criterium of 20 g/m^3 for SA.

value suitable for slow sand filtration (refer to Table 3.2).

3.3 Choice of Coagulant

All treatment options other than slow sand filtration require the use of a coagulant to agglomerate the microparticles to larger flocs which are removable from the raw water. An early choice of the coagulant dosing strategy is required.

Coagulants can be broadly grouped into two categories. The first category comprises those coagulants which form reaction products with the raw water. The products could be polymeric species (which will neutralise particle charges and provide interparticle bridging) or bulky precipitates (which will "sweep" up the natural particles during sustained turbulence). Metal coagulants such as ferric chloride or aluminium sulphate could be polymeric or precipitating, depending on the pH domain. Lime, when added in sufficient excess, will precipitate magnesium hydroxide and calcium carbonate. In all cases, the coagulant reacts with the water and so changes the ionic composition of the raw water. The second category comprises the polymeric coagulants which do not react with the water, but will neutralise particle surface charges and provide interparticle bridging. These polymers do not affect the ionic composition of the water.

Polymeric coagulants generally do not work well for clear waters with a low concentration of particles. In such waters, there is limited opportunity for bridging due to a lack of particle collisions. In these cases, precipitating coagulants are more efficient.

O'Melia (1978) used the ability of the water to react with the coagulant (as measured by alkalinity) and its particle concentration (as measured by turbidity) to develop a two-way classification system for raw waters. The guideline limits are shown in Table 3.3. Note that this classification only covers the extreme ends of the water quality spectrum, and that most practical cases will fall in the intermediate zones. The value of the O'Melia system, however, is that it firstly provides a rational basis for classification, and secondly that it will suggest a treatment strategy to the designer if his particular water is close to one of the defined classes.

Table 3.3

Limits for the O'Melia Raw Water Classification System. Units for Turbidity Are NTU, Units for Alkalinity Are g/m^3 CaCO$_3$

	Alk > 250	50 < Alk < 250	Alk < 50
NTU > 100	Type 1 water		Type 2 water
10 < NTU < 100			
NTU < 10	Type 3 water		Type 4 water

3.3.1 Type 1 water : High turbidity, high alkalinity

Type 1 water is the easiest water to treat, because the precipitating as well as polymeric coagulants will be effective. For precipitating chemicals, the water has sufficient alkalinity to allow coagulant precipitation without a sharp drop in pH in the case of metal coagulants. For polymeric coagulants, there are sufficient particles to ensure efficient flocculation.

3.3.2 TYPE 2 WATER : HIGH TURBIDITY, LOW ALKALINITY

Polymeric coagulants work as effectively for Type 2 water as for Type 1 water. Precipitating coagulants have to be used with the addition of a base to neutralise the acidification effect of the metal coagulants.

3.3.3 TYPE 3 WATER : LOW TURBIDITY, HIGH ALKALINITY

Polymers do not work as primary coagulants because of low particle concentration, unless additional particles are added in the form of clay or recycled sludge (an option rarely used in SA). The metal coagulants work well when added in sufficient concentration to form enough precipitate to sweep the particles from the raw water. A small dose of polymer may then be added as coagulant aid to produce more settleable floc.

3.3.4 TYPE 4 WATER : LOW TURBIDITY, LOW ALKALINITY

This is the most difficult water to treat. Polymers do not work, for the same reason as for Type 3 water, while the metal coagulants will also not work, for the same reason as for Type 2 water. In the case of the metal coagulants, base must be additionally added. In the case of polymeric coagulants, clay or sludge could be added to change the water to Type 2.

3.4 WATER STABILITY

The corrosivity of water is a complex, as yet poorly understood phenomenon. Although there have been numerous reports that the commonly used equilibrium of calcitic calcium carbonate has not always been an accurate predictor of corrosivity (Stumm, 1960), it still is the only simple method which is used for routine water analyses. A number of indices based on the calcite equilibrium have been formulated and are routinely used, such as the Langelier and Ryznar indices. In South Africa, it has become commonplace to use the calcium carbonate precipitation potential (CCPP) index to express the degree of under- or oversaturation with respect to calcium carbonate (in the calcite form). This index was introduced by researchers at the University of Cape Town and has the advantage that it expresses the *potential quantity* of calcite that could be precipitated before equilibrium is reached, whereas other indices are merely indicative of the *tendency* to precipitate calcite.

The CCPP for treated water, recommended by Loewenthal *et al.* (1986) is between 5 and 10 g $CaCO_3/m^3$, with the additional requirement that the alkalinity should be at least 30 g/m^3 as $CaCO_3$ for sufficient buffer capacity to keep the CCPP reasonably constant during distribution. If the alkalinity is high enough, it will usually only be required to neutralise the acidifying effect of the metal coagulants, if they are used. In some cases where highly buffered water is treated with small coagulant dose, pH correction may not even be required. If the alkalinity is lower than 30 g/m^3, it has to be increased with lime in combination with carbon dioxide. This is considered to be a special problem, addressed later on in the chapter.

The CCPP is a value calculated from six water quality parameters, namely the water temperature, total hardness, calcium hardness, total alkalinity, pH and conductivity. The calculation procedure, if attempted by hand, is extremely tedious. Friend & Loewenthal (1992) have developed a convenient software package and the Water Research Commission has made it available as the STASOFT program, which allows the designer to rapidly analyse a raw water for a number of treatment scenarios and determine the chemical dosage required to stabilise the water.

At process selection stage, it is not only necessary to assess the overall requirements for water stability, but to consider the *point* of stabilisation. Three practical considerations are:

- Metal coagulants, if used in the charge-neutralisation domain, have an optimum pH range which is quite low. In this case, water stabilisation should be delayed until after coagulation.
- Not only the corrosivity of the final water, but also the corrosivity of the water *during treatment* should be calculated. If the water is excessively corrosive during a part of the treatment process, special measures should be taken to protect structures and equipment in that part of the process.
- Commercial lime, the most common and cost-effective base used for pH control, contains considerable quantities of insoluble impurities (mostly silica). Lime slurries will therefore impart turbidity to final water, and lead to deposits in pipes and storage tanks. If slurries are used, they should be added ahead of final filtration. If pH must be corrected after filtration, a clear solution of lime should be prepared.

3.5 PHASE SEPARATION

Phase separation will always include rapid sand- or multimedia filtration if coagulants are used. Of interest in this section is the phase separation process immediately ahead of filtration. The practical options currently are:

- In-line or contact filtration, where coagulant is injected immediately ahead of filtration, without a flocculation step.
- Direct filtration, where a distinct flocculation step is used before filtration.
- Series filtration, where water is passed through a coarse upflow flocculator/filter before filtration. This is a new process in South Africa, used with great success on smaller treatment plants where raw water conditions are suitable.
- Sedimentation/clarification, where water is clarified after flocculation. A distinction is made between settling tanks (where settling only takes place) and clarifiers (where flocculation and settling take place as an integrated process). This distinction, and criteria for choosing between them, will be discussed in Chapters 9 and 10.
- Dissolved air flotation, where particles are concentrated in a float layer by the injection of a very fine air suspension. This process is rapidly gaining ground in South Africa and the rest of the world for treating eutrophic waters.

There are also a number of membrane options for phase separation, which are not covered in this book as they currently have limited current application.

Particle suspensions are ideally characterised by the number concentration, the volume and density of the particles. With this known, one can select the most appropriate phase separation process, based on rational mathematical models of the different processes (Wiesner & Mazounie, 1989). In practice, this very specific information is almost never available to the designer. At best, selection of the phase separation process has

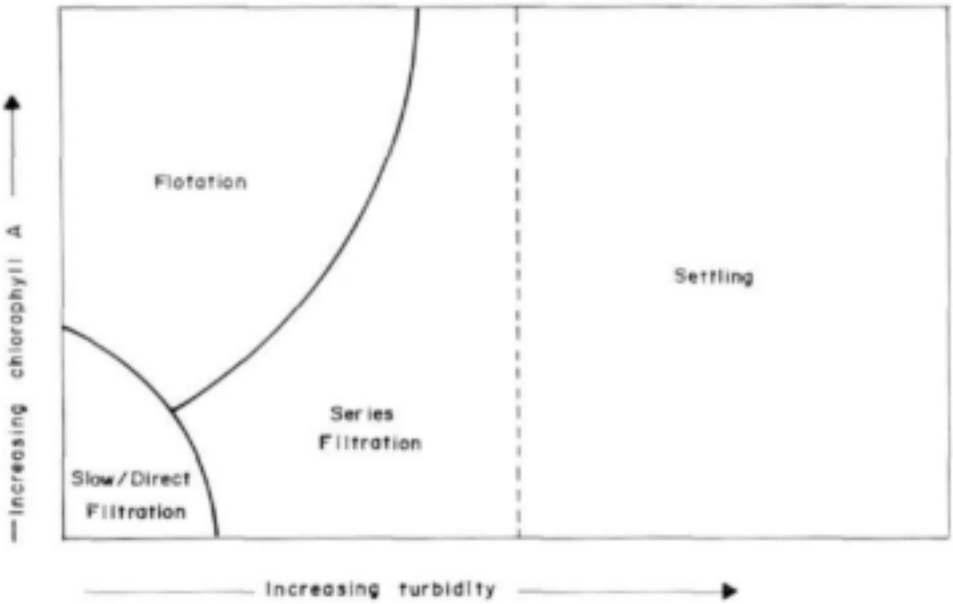


Figure 3.3 *Qualitative diagram for the selection of phase separation processes (after Haarhoff* et al.*, 1989)*

to be based on turbidity (an indicator of particle concentration of all particles, inorganic and algal) and chlorophyll *a* (an indicator of the particle concentration of the algal particles).

In suspensions where only relatively heavy, inorganic particles are found, sedimentation/clarification and filtration will be adequate. In suspensions where algae predominate, dissolved air flotation and filtration will be the processes of choice. In mixed suspensions, *sedimentation and filtration* will be adequate, provided that the ratio of inorganic to algal particles is high enough to form settleable flocs upon flocculation. If not, dissolved air flotation and filtration will be required. In cases where the ratio of inorganic to algal particles varies, it is required to use both sedimentation and dissolved air flotation, as for example at the shallow Mhlatuze impoundment in Richards Bay (Botes & Van Vuuren, 1990) or Vaalkop Dam (Anonymous, 1994).

A first attempt at a systematic approach to this problem, by only using turbidity and chlorophyll *a*, was published in the form of a qualitative diagram, reproduced as Figure 3.3 (Haarhoff *et al.*, 1992). This was subsequently further developed, by using South African and international experience, into a quantitative decision tree (Janssens & Buekens, 1993), summarised in Table 3.4.

3.6 DISINFECTION

The consequence of inadequate disinfection of drinking water is human disease or death. This gives disinfection the highest priority of all the treatment processes. There are two objectives with disinfection, namely firstly to *inactivate* all harmful organisms during treatment, and secondly to *maintain* this quality during water distribution.

The first step is achieved through a combination of the concentration (C) of the disinfectant, and the time (t) during which this concentration is maintained. The design objective is to maintain a required C.t product, which is specific to each disinfectant. This approach, for example, underlies the current US regulations on minimum disinfection requirements. The designer has a

TABLE 3.4
TABLE FOR SELECTING PHASE SEPARATION PROCESSES
(ADAPTED FROM JANSSENS & BUEKENS, 1993)

	Chl *a* < 10	10 < Chl *a* < 25	Chl *a* > 25
NTU < 10	-	DAF	DAF
10 < NTU < 20	series filtration	DAF	DAF
20 < NTU < 50	series filtration	pilot testing	DAF
50 < NTU < 100	settling	pilot testing	DAF
NTU > 100	settling	settling	settling

choice as to which disinfectant and contact reactor type to employ - these aspects are dealt with in Chapter 14.

The second step is to maintain a residual concentration of disinfectant throughout the distribution system to safeguard the supply against ingress of contaminants or the products of bacterial growth within the distribution system. Here the designer has the choice of disinfectant.

In South Africa, chlorine was used very commonly for both steps in the past, and currently still is. Addition of ammonia after chlorination to ensure an effective residual, is also practised by some of the larger water supply utilities. During the past decade or two, however, there has been significant developments overseas in the area of disinfection, mainly driven by the health fears around the disinfection by-products of chlorine. Although South African water quality guidelines currently do not necessitate an immediate departure from chlorination practice, designers should stay abreast of the different disinfectants and latest overseas developments in their application. Further information is given in Chapter 14.

3.7 SPECIFIC PROBLEM COMPOUNDS

3.7.1 INSUFFICIENT ALKALINITY

In most cases South African surface waters are well buffered and water stability is ensured by simply adjusting the pH, mostly by base addition, to obtain a water with a slight tendency to precipitate calcium carbonate (a CCPP between 5 and 10 g/m^3). If, however, the total alkalinity will be lower than about 30 g/m^3 *after* coagulation and disinfection, alkalinity must be added before the pH is finally adjusted. This is required to ensure a stable pH while the water is retained in the distribution system. This is mostly done by the addition of lime and carbon dioxide to create more alkalinity in the form of bicarbonate. The details of this process are further described in Chapter 13.

3.7.2 REMOVAL OF COLOUR

Excessive colour diminishes the aesthetic appearance of water, and may drive consumers to other sources with higher health risks.

For this reason, a limit of 20 g/m^3 Pt is proposed for drinking water in South Africa (Aucamp & Vivier, 1990).

It is important to differentiate from the outset between *apparent* and *true* colour. The former is the total colour of the water sample (caused by particulate as well as dissolved components), while the latter is the remaining colour after 0,45 µm membrane filtration (assumed to be caused by dissolved components only). Raw water with high apparent colour but low true colour will very probably be within the guidelines after conventional phase separation. If the apparent and true colour are both high, special processes may be required for colour removal.

Colour is a non-specific substitute parameter which represents a wide array of compounds. No general predictions can thus be made on how well different processes will remove colour, which explains the lack of quantitative limits for the process options outlined below. In all cases, it is essential to conduct a comprehensive laboratory evaluation of the proposed process train to verify that colour is adequately removed.

There are a number of strategies for dealing with colour that does not comply with the guideline value:

- For low, but persistent colour, the designer could select a strong oxidant. Ozone is the most effective oxidant for this purpose, followed by chlorine, chlorine dioxide and permanganate. These oxidants are discussed in Chapter 16.
- For low to moderate levels of colour, adsorption by activated carbon could be considered. If the problem occurs sporadically for short periods only, powdered activated carbon would be cost-effective, but if the problem persists for longer periods or permanently, granular activated carbon should be used. Adsorption is also discussed in Chapter 16.
- For highly coloured water (typical of the mountain water of the coastal regions of SA), coagulation and flocculation under very specific conditions are required. In this case, coagulation must be performed at carefully controlled pH and dosage. These specific coagulation and flocculation requirements are further discussed in Chapter 6.

3.7.3 REMOVAL OF IRON AND MANGANESE

Iron and manganese are often found together in raw water sources, and have similar chemical behaviour. They do not pose any health risks at concentrations normally encountered, but have severe aesthetic effects; iron in the form of red-brown deposits or stains, manganese in the form of black-brown deposits or stains. Under reduced conditions (such as in an anaerobic hypolimnion, or in some groundwaters), both metals exist in the dissolved ferro (Fe^{+2}) or mangano (Mn^{+2}) forms. When they are oxidised (by chlorination, for example) they precipitate in the oxidised ferri (Fe^{3+}) or mangani (Mn^{+4}) forms. If iron and manganese are present in excessive amounts, the oxidation process must be forced to completion during treatment, and precipitates removed before final water is discharged into the distribution system.

Iron is oxidised more readily than manganese. Aeration may be adequate to precipitate iron rapidly at pH values higher than about pH 7. Manganese, on the other hand, requires a stronger oxidant than oxygen, such as chlorine or potassium permanganate. In both cases, the oxidation rate rapidly increases with increasing pH. In the case of manganese, it is often necessary to raise the pH to speed up the rate of oxidation. Guide-

TABLE 3.5
GUIDELINE CONCENTRATIONS FOR DIFFERENT IRON AND MANGANESE REMOVAL METHODS (ADAPTED FROM DWAF, 1993). CONCENTRATIONS IN g/m³

	Iron	Manganese
Guideline value for drinking water	< 0,1	< 0,05
Adequate removal during conventional coagulation, flocculation, sedimentation and filtration	0,1 - 1,0	0,05 - 0,1
Oxidation, with pH correction if required, followed by flocculation and filtration	1 - 5	> 0,1
As above, but with sedimentation ahead of filtration	> 5	

line values for process selection are given in Table 3.5.

Manganese can also be removed by the high lime treatment process (where manganese hydroxide is precipitated at high pH and removed with the other precipitates), or by catalytic filtration (where manganese precipitation is enhanced by the presence of previously precipitated manganese dioxide).

Iron and manganese removal is further discussed in Chapter 16.

3.7.4 REMOVAL OF TASTE AND ODOUR

Taste and odour in drinking water lead to a severe, often emotional reaction among consumers. Taste and odour usually also are a transient problem, which makes any analytical investigation at process selection time almost impossible. Even if representative samples could be obtained, a further complication is that quantification of taste and odour is notoriously difficult.

The most common taste and odour problems in SA are caused by algae or other microbiological organisms. Of particular importance in the eutrophic waters are the compounds geosmin and 2-methylisoborneol (MIB). Taste and odour can also be caused by chemical contaminants, such as phenol or other chemical products. In these cases, pilot studies have to be conducted to determine appropriate design parameters.

The process designer has the following options if taste or odour could be a problem:

- Odour due to dissolved gases, such as hydrogen sulphide, can be stripped from water by simple aeration. Aeration will not be effective for the less volatile MIB and geosmin.
- If taste and odour are a likely problem to occur, but only for short periods, it is recommended that facilities are provided for the dosing and subsequent removal of powdered activated carbon. The capital cost of these facilities is low; the main cost lies in the activated carbon itself. Provision should be made for dosage to a

maximum of about 25 g/m^3.

- If it is likely to persist for longer periods, it may be more cost-effective to use granular activated carbon.
- It may prove feasible to remove taste and odour by ozone, which is the most effective of all the oxidants for taste and odour removal. Ozone will react differently with different types of organic compounds, and tests are essential to determine its efficiency. Ozone in combination with UV or hydrogen peroxide is an advanced option which removes taste and odour more efficiently than ozone alone.

3.7.5 REMOVAL OF HARDNESS

Excessive hardness in drinking water causes scale-formation in distribution systems and especially in hot water systems; this has vast economic consequences. An international hardness scale is shown in Table 3.6. A recently published SA guideline suggests that hardness should be limited to 100 g/m^3 (DWAF, 1993).

TABLE 3.6
INTERNATIONAL CATEGORISATION OF HARDNESS (FROM DWAF, 1990). CONCENTRATIONS IN g/m^3 $CaCO_3$

Hardness range	Category
0 - 50	soft
50 - 100	moderately soft
100 - 150	slightly hard
150 - 200	moderately hard
200 - 300	hard
> 300	very hard

Water of unacceptable hardness may be softened by lime or soda ash precipitation of the calcium and magnesium ions. Although this requires pH reduction afterwards, it is the most economic treatment method if large volumes and large concentrations are involved. For smaller, or point-of-use applications, ion exchange may be more appropriate. Ion exchange is further discussed in Chapter 13.

3.7.6 REMOVAL OF NITRATE

Nitrate and nitrite should be considered together because conversion from the one form to the other occurs in the environment. It is primarily of health concern because nitrite causes methaemoglobinaemia, which is particularly dangerous to infants under the age of three years. The recommended maximum value for SA is a total of nitrite and nitrate of 6 g/m^3, expressed as nitrogen.

Nitrite and nitrate are not removed by conventional treatment and require special measures. It is most common to use ion exchange resins for removal, which is further discussed in Chapter 16. It is also possible to reduce these compounds biologically, which falls outside the scope of this book.

3.7.7 REMOVAL OF FLUORIDE

A small amount of fluoride is necessary for healthy human bone and teeth structure, but too much will cause mottling of teeth and skeletal fluorosis. In SA, fluoride addition to drinking water is not allowed, and the process designer is primarily concerned with ensuring the maximum concentration. For this reason, the SA guideline value is 0,0 g/m^3 to 1,0 g/m^3.

Fluoride below 20 g/m^3 is not removed by conventional treatment. For the concentration range between 1,0 and 20 g/m^3, it will be necessary to use ion exchange with activated alumina or reverse osmosis. At higher levels, treatment with lime or aluminium sulphate will reduce the concentration to 20 g/m^3 or less, after which ion exchange is required (DWAF, 1993).

3.7.8 REDUCTION OF ORGANIC CARBON

Dissolved organic carbon is not a problem *per se*, but it is an indicator of the organic content of raw water. It may be an indicator of sewage pollution, could be precursor to undesirable disinfection by-products, and will interfere with the efficiency of coagulant and disinfectants. It also points to a greater potential for biological activity which usually leads to difficulties in controlling biofilm formation and bacterial activity in distribution networks. For these reasons, a SA guideline value of 5 g/m^3 was set (DWAF, 1993).

If values higher than this are encountered, the process designer should investigate whether the source is natural or of industrial or sewage origin. In the latter case, special care should be taken to reduce the organic content or to target specific harmful constituents of the organic carbon.

Conventional coagulation will remove roughly 30% of the organic carbon. If higher removals are required, strong oxidants (ozone, hydrogen peroxide, UV radiation), activated carbon adsorption (powdered or granular), or a combination of oxidation and adsorption could be used. These are advanced treatment options and general predictions and rules of thumb are not available. In such cases, the process designer should revert to pilot-scale studies.

3.8 MANAGEMENT AND DOSING OF CHEMICALS

The process designer, after conducting the appropriate analytical tests and bench-scale testing, should have a clear idea of the types of chemical, and the dosage rates required. In addition, the process designer is normally also the best equipped to advise on the following:

- The maximum, minimum and average dosage rates of each chemical, which determines the running cost of operating the plant, and also determines the maximum turndown ratio which the dosing equipment should be capable of.
- The form in which the bulk chemical should be purchased, as well as the suggested chemical inventory to be carried at the treatment plant.
- The suggested means of handling, conveying, measuring, preparing and mixing each chemical prior to dosing.
- Suggested materials of construction to counter the often corrosive or abrasive properties of each chemical.
- The safety requirements of each chemical, in terms of ventilation, protective gear and rinsing of eyes, skin or other surfaces.

3.9 MANAGEMENT OF SLUDGE AND WASHWATER

The management and disposal of sludge and washwater are an integral part of process selection and design. The following are examples of how this may impact on process selection:

- For applications where a high premium is paid for raw water, it may be part of the process designer's brief to reduce the treatment losses to an absolute minimum through recovery of sludge and washwater supernatant. Recycling of the recovered water into the plant will impact upon the hydraulic design of the plant.

- In other cases, there may be environmental or topographical restrictions which dictate sludge thickening and land filling. This may call for additional processes such as centrifuging and chemical dosing, or it may affect the choice whether the float layer on a flotation tank is scraped or washed off.
- It was shown that the counter-current reuse of powdered activated carbon may lead to a reduction in the quantity of virgin powdered activated carbon required (Le Roux & Van der Walt, 1991). This possibility should be investigated by the process designer at bench or pilot scale.

It is essential that the process designer clearly depicts the point of sludge or washwater abstraction, the means of thickening or disposal, the chemical dosage requirements if any, and the point of adding any return streams to the main process streams. The expected flow rates (maximum, average and minimum) and solids content should be estimated and indicated. Sludge and washwater are dealt with in Chapter 15.

3.10 PROCESS SELECTION DIAGRAMS AND REPORT

Up to this point, the process designer had to make numerous decisions, based on a wide array of data. It is essential that this information is clearly conveyed in a comprehensive, summary report and process selection diagrams (whether the process selection is performed by in-house specialists or outside consultants) for a number of reasons:

- It serves to organise, document and preserve the information available at the time of process selection (which often entailed a considerable expenditure in terms of time and analytical testing).
- It enables the client or designer to have the process selection report reviewed by outside parties for final verification before the detail design stage is commenced.
- It serves as a clear, unambiguous brief for the detail designers to proceed with maximum speed and efficiency.
- This report will be an invaluable document whenever the treatment plant has to be upgraded or extended in the future.

The final selection of processes should be summarised in a series of process diagrams, which should include at least the following:

- The main process stream, indicating the unit processes required, and showing the flow rates at each point in the process stream.
- The points of chemical dosage, with chemical flow rates.
- The points of sludge and washwater abstraction and return, with flow rates.
- The points of level and flow measurement to enable accurate operation and control.
- Separate diagrams for chemical storage, handling, preparation and control, indicating requirements in terms of safety.
- Separate diagrams to indicate the handling of the sludge and washwater streams.

3.11 COMPUTER-AIDED PROCESS SELECTION

Expert systems, implemented on computer, aim to support practitioners in all disciplines by encoding the type of information which experts subconsciously use when making decisions. A successful implementation of an expert system is, for example, in the medical field where it supports doctors to make rapid and accurate diagnoses. Process selection for water treatment falls in a similar domain, where experts use a multitude of semi-quantitative decision rules and experience almost subconsciously to arrive at the "best" solution.

A pioneering attempt to develop an expert system for water treatment process selection was recently completed in South Africa (Patrick, 1993). (This expert system is currently being revised and strengthened in a number of areas, as part of a project sponsored by the Water Research Commission.) The raw water properties are entered, and the designer can see the changes in water quality brought about by different treatment processes. Different processes can be added until the water quality complies with the drinking water criteria. The system allows for 15 water quality criteria and 10 different treatment processes. The weakness of this system is that it is not presently possible to accurately predict the effect of each process on each contaminant. It is, however, a promising starting point to develop and formalise a computerised knowledge base containing many of the decision rules used by experts.

The system has two additional features to assist the designer in selecting a process combination. The first is an "appropriateness" index to quantify each process in respect of the capital, operational skill and maintenance required to keep it going. The second is a number of typical South African raw water quality profiles, which enable the designer to place his own problem near one or two of the "typical" waters, to allow verification of his own process selection against the treatment processes normally used for somewhat similar water.

3.12 PILOT-PLANT TESTING

Pilot testing is normally required for three possible reasons:

- To determine whether a process is capable of treating the water to the desired standard.
- To determine the design parameters required for full-scale implementation, for example hydraulic loading for filters, flocculation intensity and time, etc.
- To assess the operational and economic merits of alternative processes, for example to determine the dosage rates required for chemicals, backwash frequency of filters, etc.

Planning and execution of pilot studies are a rather specialised field on which the process designer should consult other sources. Detailed information is, for example, available on mixing and flocculation (Newkirk & Trussell, 1991), filtration (Adin *et al.*, 1979; Hudson, 1981), activated carbon adsorption (Sontheimer *et al.*, 1988) and ozonation (American Water Works Association Research Foundation *et al.*, 1991). The following general considerations are vitally important:

- It is essential to include the widest possible range of raw water quality variations. Normally, a year of testing is suggested to cover the seasonal changes, but even this may not be enough in South Africa, where water quality cycles often follow the longer term dry and wet cycles.
- The objectives of the pilot test program must be narrowly defined and clearly formulated. Pilot testing cannot be used to replace the normal systematic procedure of process selection - it can at best give clarity on a limited few uncertain points.

- Early consideration should be given to the scale of testing. Some processes which rely heavily on the hydraulics, such as inlet and outlet arrangements of settling tanks, must be performed near full scale for sensible results. For filtration, narrow columns can be used successfully, but filter backwash studies require larger diameters. Other processes, such as dosing and adsorption studies, could possibly be performed at bench scale only. As the logistics of pilot testing is so heavily dependent upon the flow rate through the pilot plant, the scale is one of the first decisions to be made after the testing objectives have been determined.

4

Intakes

F A van Duuren

Intakes to water works are required to supply an adequate quantity of raw water of the most suitable quality. Reliability of supply and economics are important considerations for intakes. An insufficient quantity or indifferent quality of water is obviously unacceptable. The reliability of the draw-off is thus of prime importance. Raw water quantity, quality and detailed design have economic implications for the whole works.

Adequate steps have to be taken to facilitate proper draw-off and thus to obviate problems downstream in the purification system. Intakes must be designed as an integral part of the water purification works because they affect the train of units, processes, the final product and economics. Detailed design of an intake will affect the operation and control of the entire water works; it may also well affect management.

A number of factors are involved in the shape, size and type of intake. These aspects are influenced by topography, location, environmental conditions and the source.

Topography is of physical importance to inlet works, intake conduits and access shafts.

The location of an intake is determined by the characteristics of the river, reservoir or lake to the extent that it affects intake water quality. The quantity of water to be drawn off affects the location and submergence of the intake.

Environmental conditions determine the amount and type of debris, sand, silt, algae, aquatic plants, aerobic or anaerobic water and thus affect the intake. Movement of sediments in and around an intake, can be determined by means of model studies.

There are various types of intake; the type of intake used is affected by the nature of the source.

Surface water intakes are sited on rivers or in impoundment reservoirs. River intakes might even be well points embedded in sand. An intake for a small water works can be simple but must nevertheless be reliable as regards quantity and quality of water. Major intakes can be elaborate and require particular attention as regards siting and hydraulics of the system.

4.1 TYPES OF INTAKE

The general types of intake are river banks and lake or impoundment reservoirs. There are a number of variations of these types of intake. These include either exposed or submerged river inlet towers and shore pumping stations, well point systems, siphon intakes and suspended, floating or movable intakes. Intake towers are used in lakes and reservoirs.

The major aspects of importance to types of intake are: location, submergence, inlet structure, intake conduits and access shafts. The selection of type of intake can further be based on debris handling, screening, capital and operating costs and environmental constraints (Public Works, 1988). Pumping or gravitation of flow also has to be considered.

The main choice i.e. river, dam or lake intake will significantly be determined by selection of the source, development and design.

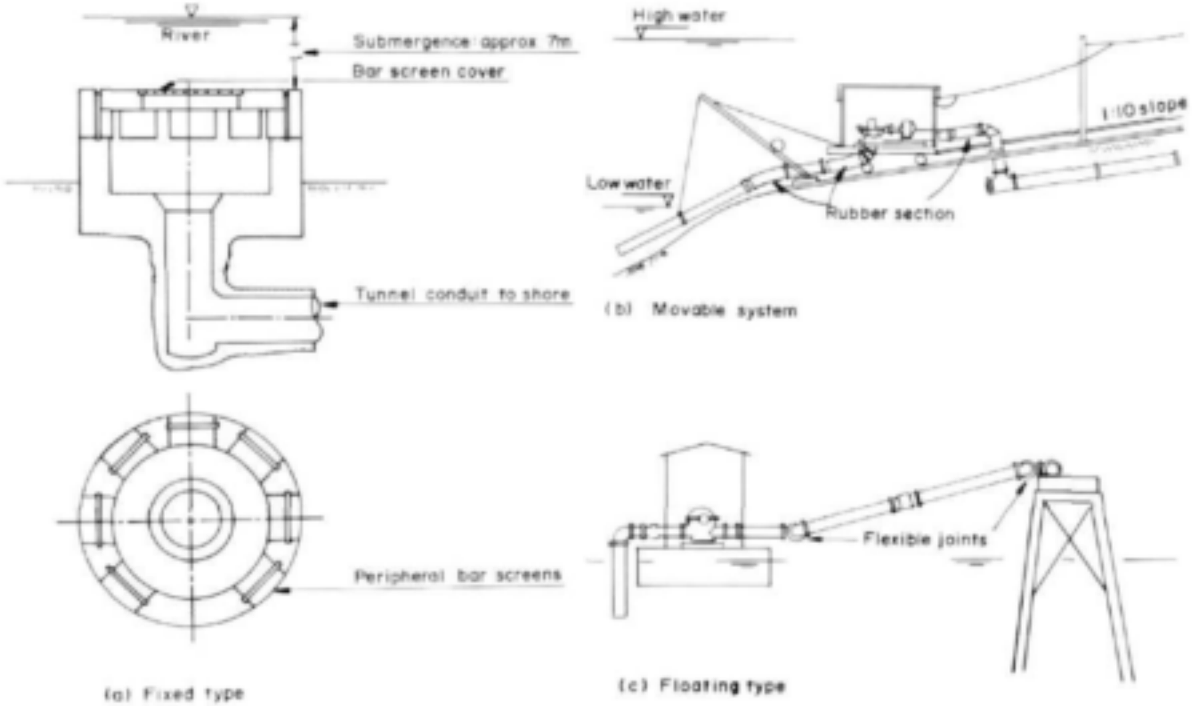


Figure 4.1
River intakes

River intakes include fixed, movable and floating types. Lakes, reservoirs or dam types of intake are used where water is held in an impounding reservoir.

4.1.1 RIVER INTAKES

Alternative intake systems should be considered because they may offer equally reliable service at lower cost. Various types of river intake are shown in Figure 4.1.

In a low sediment load carrying river, with fairly stable flow, an offshore intake can be provided, as shown in Figure 4.1 (a).

For a small or temporary water works it may be necessary to place the intake on a movable carriage. The intake can then be moved towards or away from the river as the water level rises or falls. This may prove useful in overcoming low-water and flood difficulties and bed-load problems. This type of intake is shown in Figure 4.1 (b).

Where river levels may change appreciably, either multiple inlets or floating inlets, as shown in Figure 4.1 (c), should be used.

The draw-off systems include fixed levels, adjustable, movable, and floating draw-offs. Well point intakes can be effective except where there are quality problems such as iron and manganese which are aggravated under anaerobic conditions.

For large water supply systems using rivers as a supply source, a shore intake is commonly provided. Such intakes combine inlet works and raw-water pumping facilities. They might well incorporate pretreatment facilities or be close to them.

A river bank intake system would include gated ports protected by coarse racks and screens. Finer, mesh screens or micro strainers would be placed inside the intake structure, ahead of the raw water pumps which are located in a dry well. Alternatively, screened pipe inlets can be located on the river channel and a wet well type of pump station can be provided in the river bank. Towers can be used as river intakes, but accessibility must then be assured. Variations of these arrangements are used.

4.1.2 LAKE AND RESERVOIR INTAKES

Impoundment intakes have often in the past been provided as part of the dam wall. The three level type of intake tower has commonly been used in South Africa for irrigation water but might be inadequate for intakes to a water purification works. This type of intake can create serious problems when for instance main stream flow or prevailing wind direction is not taken into account. Location away from the dam wall and multiple level intakes may well be needed to obviate these problems.

Both tower or submerged intakes are employed for raw water supplies, from lakes and reservoirs. Intake towers are usually placed in or on dams, at the deepest point of the water. They are often incorporated in the dam wall for the sake of simplicity. Intakes should preferably be placed where the best quality water can be drawn off.

Inlet ports should be placed at vertical intervals, in order to permit water quality selection. Intake towers can be costly, but careful siting has economic implications for water purification and is therefore desirable. Siting of intakes remote from dams, should be determined from a study of density currents and prevailing winds as these would involve turbidity, algae and water weeds.

A tower intake may well include a pumping station and may be part of the dam that creates the impoundment. Such an intake is shown in Figure 4.2.

An intake tower may well be designed as an independent structure located some distance from the shore, in the deepest part of a reservoir. Access to the tower is then provided by bridge, causeway, or boat. These towers must be designed to cope with flotation, wind, wave and log jam forces.

Submerged intakes do not obstruct boating and are usually less costly than exposed towers. They consist of a submerged inlet structure and intake conduit and a shore shaft or suction well. Under suitable conditions and if properly located and designed, submerged towers experience a minimum of clogging difficulties. Silt, grit, sand and bedload problems can, however, be serious for submerged intakes.

River bank intakes are commonly used despite log and sediment problems which reduce their effectiveness for periods of time. Operation and maintenance of these intakes are therefore necessary.

4.2 WATER QUALITY AND LOCATION

Important factors influencing the location of intake-system inlet works are water quality, aquatic life, silt and sand problems, wind, waves, floods, channel characteristics, accessibility, foundation conditions and economics. An intake might well have to be abandoned if excessive water pollution, floating vegetation or debris or substantial deposits of silt and sand are encountered.

The location of an intake should be selected well upstream of local sources of pollution even if these are only potential sources. Considerable variation in water quality can result from the entrance of point-source pollution and tributary streams upstream of the proposed intake location. Thus water quality near one bank may be inferior to that near midstream or the opposite shore.

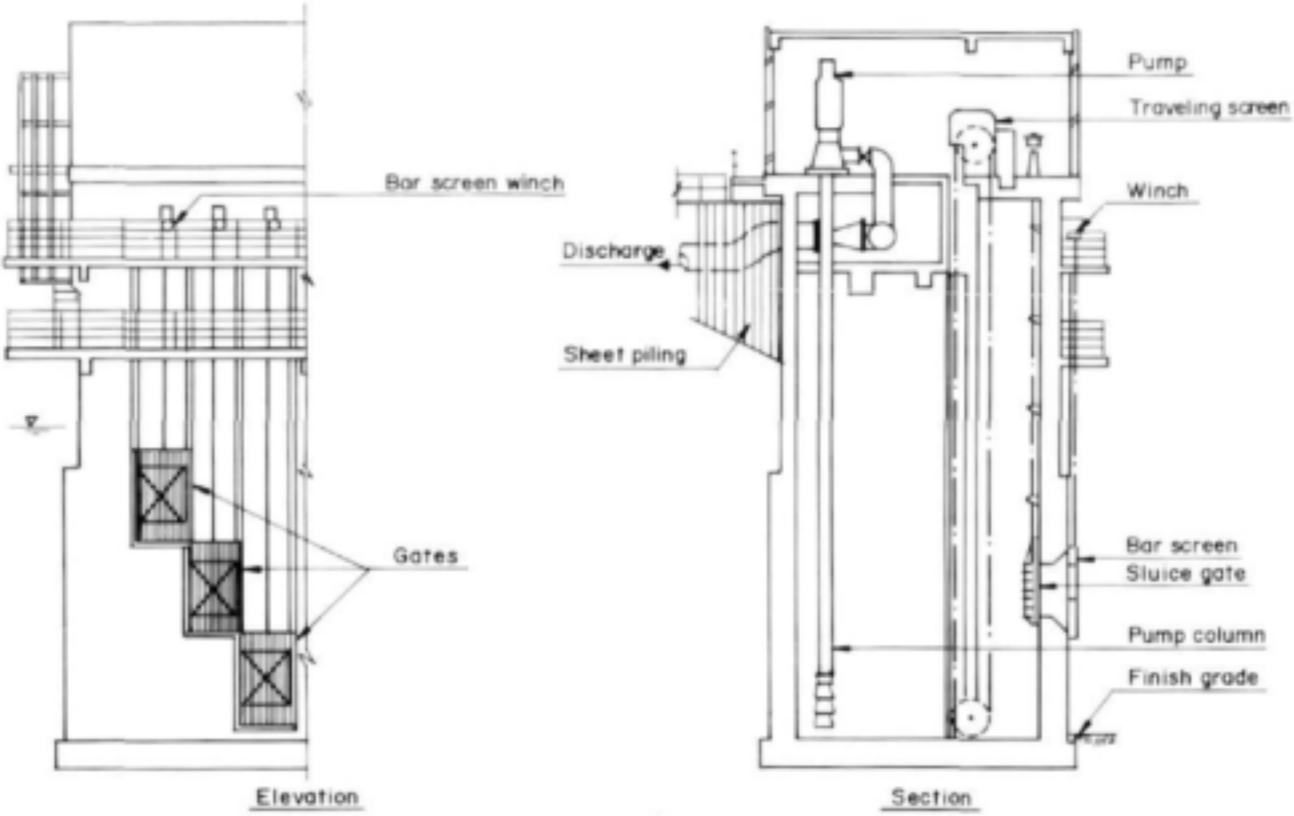


Figure 4.2
Intake tower at a dam

4.2.1 QUALITY ASPECTS

The water quality aspects which are of concern include pollution, eutrophication, algal and water weed infestation, dissolved gases, anaerobic conditions, iron and manganese. Water quality is dealt with more fully in Chapter 2. Some of the problems for intakes and their solutions are discussed here.

In considering alternative intake sites, pollution potentials and eutrophication should be carefully reviewed and a thorough study should be made of available water quality data at the site of the intake. Where such data are not available or is inadequate, a substantial water sampling and testing programme may be warranted, particularly if a high-capacity, costly intake is under consideration.

The presence of algae and weeds would influence siting of the intake to minimise the problem. It would also influence subsequent process selection. The development of excessive algal or weed growth might force abandonment of the intake.

High organic matter content in a river or reservoir bed will give rise to dissolved gases and indicates the need to avoid of lower-level draw-offs. Anaerobic conditions could well give rise to unacceptable concentrations of iron and manganese. Avoidance of these conditions is preferred or else steps have to be taken to provide a necessary process such as oxidation by aeration or chlorination.

If the mainstream flow is sediment-laden and no provision is made for the selection of level of intake, very turbid water can be drawn into the water works. Density currents, deflected upwards from the dam wall, will carry turbid water towards intakes if these are not correctly sited. High turbidity would necessitate higher coagulant and chemicals dosages and thus lead to increased cost of purification. It would also cause increased solids removal in clarifiers and filters and more sludge disposal difficulties.

Steps need to be taken to avoid or ameliorate sedimentation problems. Siting and orientation of the intake and a properly designed sill or deflectors should minimise these problems.

Should there be algal growth in the reservoir and the intake is sited close to the banks of the lake or in shallow water, algae will inevitably enter the water works. High algae concentration will result in increased coagulant and other chemical dosages. It could well result in the creation of taste and odour and even a toxic hazard in the otherwise purified water.

Anaerobic conditions may occur at depths in water below the hypolimnion.

4.2.2 LOCATION AND PRESELECTION

Reservoir intake should be located with a view to an adequate supply of water of the best quality, consistent with reliability, economical construction and minimum effect on aquatic life. The intake water should also not be algae or silt laden.

The preferred location for a river bank intake system is one that provides deep water and a stable channel, and thus good quality water. The outside bank of an established bend in a river generally offers the best channel conditions. The inside bank is likely to be troublesome because of shallow water and sandbar formation.

In order to reduce clay, silt and sand problems, a submerged inlet should be located where these occur the least. Alternatively, clay, silt and sand have to be dealt with outside the river or reservoir by pretreatment.

The intake should preferably be located where the water is deep. A long intake conduit may be needed to accommodate the necessary depth.

4.3 FLOWS

Intake capacity is directly related to inlet submergence, and the designer should be aware of the substantial variations in water levels that are encountered in rivers and impoundment reservoirs. The intake system must have the capacity to meet the maximum water demand during its projected service life.

The required flow must be available during periods of minimum water level of the supply source. Inadequate submergence could result in greatly reduced capacity. In order to obviate clogging, high submergence should be provided for rivers that are subject to floating logs and debris.

A conservative approach should be adopted in establishing the lowest water level for which the intake system is designed. Substantial water level variations are inevitable and the hydraulic performance of the overall intake system must be evaluated under the most adverse lake or reservoir level conditions.

It may be impossible to obtain adequate submergence on some rivers. For such situations, adequate submergence should be created by means of a low, self-scouring weir or similar, reliable intake system.

4.4 DESIGN CONSIDERATIONS AND CRITERIA

A number of design considerations apply whatever the type of intake. These considerations lead on to the criteria that are used. Factors affecting design of intakes include water source characteristics, location, capacity, water quality, aquatic life, water levels, climate, topography, foundation conditions, sediments and economics.

It is difficult to alter or enlarge an intake once it is built. The structure must therefore be designed and built for long-term water needs, e.g. up to 30 years. The incremental cost incurred to provide a larger capacity intake could be relatively small (AWWA, 1990).

The features of intakes, whether they are on rivers or in impoundment reservoirs, can be generalised (AWWA, 1990) as follows :

- location of intake to provide water of the best available quality;
- provision for draw-off at various depths where desirable;
- provision against failure to supply water due to fluctuation in water level or channel instability;
- protection against surges, floods, floating debris and traffic;
- provision of racks and screens to protect pumps and the works;
- provision of space for cleaning and maintenance of equipment;
- provision for removal of pumps and equipment for repairs;
- location and design to minimise damage to aquatic life;
- if chemical treatment is needed, space and facilities must be provided for receipt, storage and dosing of chemicals.

These are the features of intakes in general; detailed aspects are affected by siting in a river or a lake.

4.4.1 RIVER INTAKES

Streams can at times carry heavy loads of suspended clay, silt, sand and gravel. Intakes must be designed in such a way so that they are not clogged by bed-load deposits and sediments. In addition to clogging the intake, solids will cause abrasion of pumps and other mechanical equipment and will lead to severe problems at the water works. In order to obviate or limit sedimentation, a jetty can be built to deflect the main flow of the river and suspended matter past the face of the intake.

Ample protection should be provided against flood damage. The intake structure must be designed against flotation and the thrust of log jams. Careful consideration should be given to floods at the intake site and a substantial margin of safety provided.

Because of watershed and river channel alterations, future floods may exceed those of the past. The designer should consider the possibility that the intake will be exposed to floods in excess of those on record.

For intakes located on deep rivers, gated inlet ports should be provided at several depths for selective withdrawal of water. Clogging by or intake of silt, sand and gravel deposits can cause intake failure. In most rivers, submergence governs, and ports must be placed at as low an elevation as possible. Port inlet velocities should be selected to minimise entrainment of river flotsam, debris and fish.

The lowest intake ports should be 500 mm above the bottom of the channel.

The velocity through the gross area of the ports should be $< 0{,}3$ m/s.

4.4.2 LAKE INTAKES

When intake towers are located in deep impounding reservoirs, they should have gated inlet ports at several levels in order to permit selection of water of the best available quality. Adjustment of depth of draft should result in improvement of water quality by limiting or obviating undesirable intake.

The lowest ports should be 0,5 to 2 m above the bottom to avoid or minimise the intake of sediment. Ports should be provided at 3 to 5 m vertical intervals. The velocity through the gross area of ports should not exceed 0,3 m/s.

4.4.2.1 Shore shafts

A shore shaft can serve as a screen chamber and a raw water pump suction well at the same time. The shaft must provide adequate submergence for the pumps. It could also dissipate surges due to power failure. An outlet in the shaft, large enough to readily discharge the surge back to the supply source will accomplish this.

Shafts should have adequate screens for pump protection. Travelling screens are generally preferable. Cellular construction of the shore shaft is advisable and inlet control gates should be provided so that all or part of the shore shaft can be removed from service for inspection and repair.

A tunnel or a pipeline can be used to connect a submerged inlet works with the shore shaft. Tunnels have a high degree of reliability, but are usually too costly for small water systems. Biological growths on the interior surface of the conduit may reduce its capacity; conservative hydraulic design is therefore advisable.

Intake conduits should be laid on a continuous slope to avoid air accumulation at high points. If the pipe level undulates, air release valves have to be provided at high points. The velocity in a conduit should be >1 m/s in order to prevent deposition. Pump requirements as regards swirl should, however, be taken into account.

If the intake conduit is a pipe, it can be concreted into a trench on the reservoir bottom. About 1 m of cover should be placed over the pipe and a protective top layer of crushed rock can be provided.

4.4.2.2 Racks and screens

These are essential to intake ports, to protect pumps and the purification works units, from river debris.

Racks (also termed bar racks or bar screens), preferably removable, are located at the ports to prevent entrance of large floating objects.

Racks are generally constructed of 10 to 15 mm metal bars, spaced to provide 25 to 100 mm clear openings. The larger openings are suitable for tree stumps.

Screens remove material that is too small to be deflected by the coarse racks. Screens may be of various types - for example, the basket or panel types, which can be raised for cleaning, or the mechanical, travelling types, which are cleaned by water jets. The latter are generally employed at large intakes.

There are numerous variations in the design of backwashed, fixed screens. Hydraulically cleaned (backwashed) stationary screens can be used as panel screens, can be difficult to clean, and travelling screens may well be impractical. Stationary screens, which can be cleaned quickly and easily, are preferable.

Clear openings in screens should be not less than 7 mm and may be as much as 25 mm if they are situated upstream of pumps.

Velocity of flow through the net open area of racks and screens should not exceed 0,7 m/s at maximum design flow and minimum submergence.

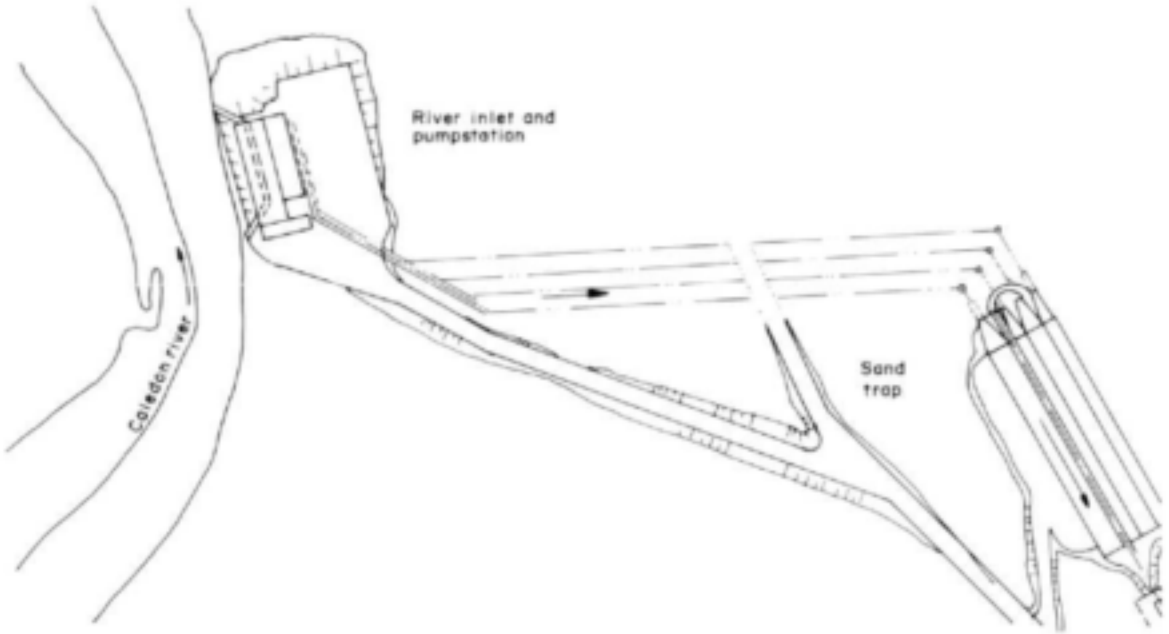


Figure 4.3
River inlet and pump station with sand trap system

4.5 EXAMPLES

The examples given here were selected for a variety of conditions and water quality. Examples were obtained from the Department of Water Affairs and Forestry and consulting engineers. These are typical and the design approach may therefore be useful elsewhere.

4.5.1 HEAVILY SEDIMENT-LADEN WATER

The Welbedacht impounding reservoir on the Caledon River is rapidly silting up. By means of Knellpoort Dam an off-channel storage reservoir has been provided to minimise sediment to the 145 Mℓ/d water purification works. Water is pumped from the river, upstream of the main reservoir, into Knellpoort at a rate of 1 to 10 m^3/s.

A sediment trap intake system was designed to reduce the silt load into the Knellpoort reservoir and on to the water works. The system consists of a river inlet/pump station and a sand trap as shown in Figure 4.3.

The sediment load of the water pumped is 0,6% of the water volume. Sixty to eighty per cent of the sediment settles in a jar within 5½ minutes. This can be regarded as a practical achievable retention time at acceptable cost. It would, however, not be possible to settle the smallest particles economically.

Other characteristics of the sediment are:

60% is 0,07 mm (fine sand)
30% is 0,035 mm (colloids)
Porosity is 30%
Dry density is 2,65.

The sediment channels were designed to remove particles of nominal diameter greater than or equal to 0,07 mm.

From the Stokes equation for laminar flow (9.1), the settling velocity of these particles is 3,83 mm/s.

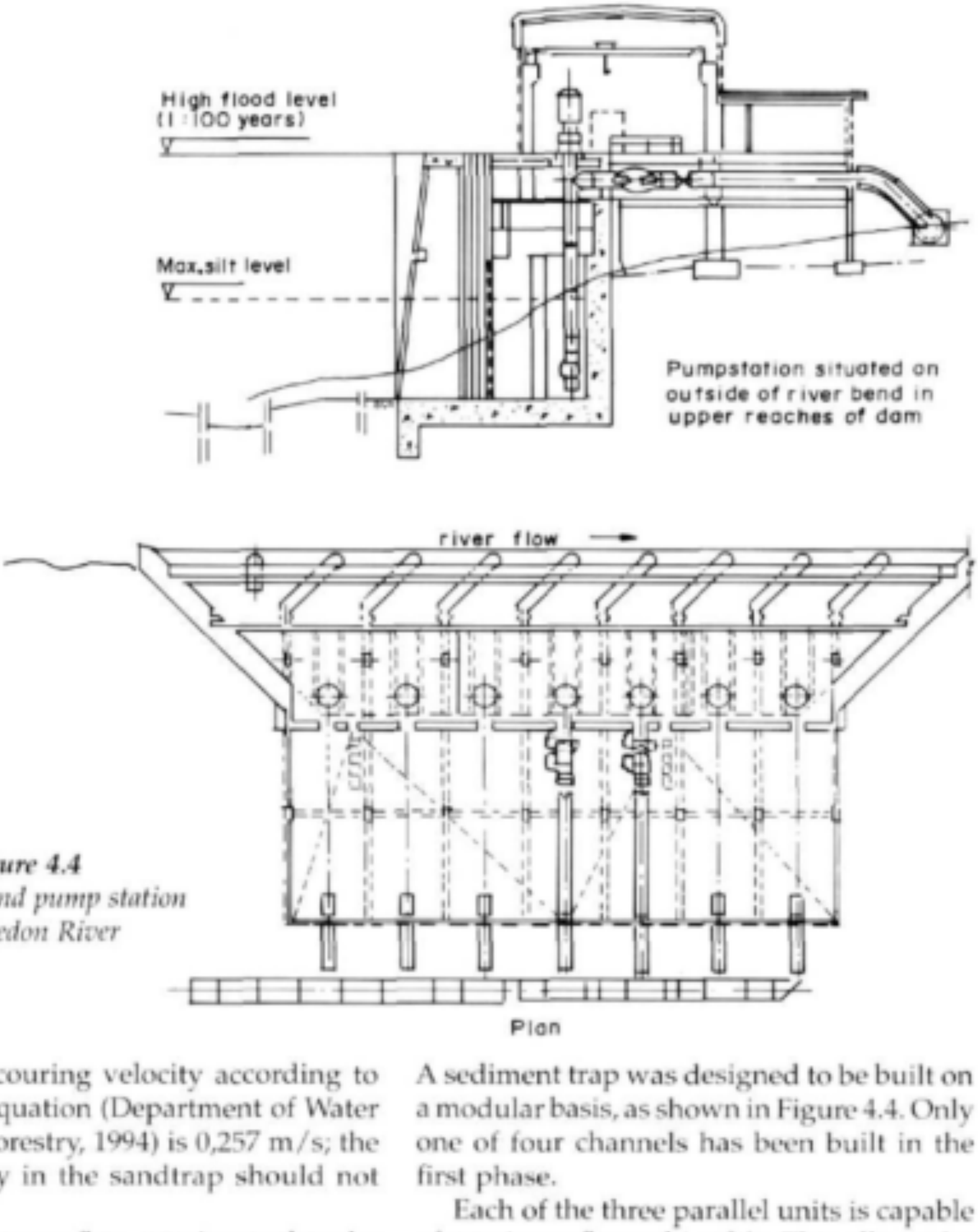


Figure 4.4
River inlet and pump station on Caledon River

The non-scouring velocity according to the Lopatin equation (Department of Water Affairs and Forestry, 1994) is 0,257 m/s; the mean velocity in the sandtrap should not exceed this.

The surface overflow rate is equal to the sedimentation velocity of the design particle;

v = Q/A
  = total inflow/surface area of the sand trap
  = 3,83 mm/s.

A sediment trap was designed to be built on a modular basis, as shown in Figure 4.4. Only one of four channels has been built in the first phase.

Each of the three parallel units is capable of passing a flow of 1 m^3/s. This allows for alternate scouring of one of the canals at a time, without stopping the flow. The other parameters and resulting dimensions are:

Length of settling zone	100 m
Width of settling zone	2,6 m
Depth of settling zone	2,5 m
Desilting slope	1:100

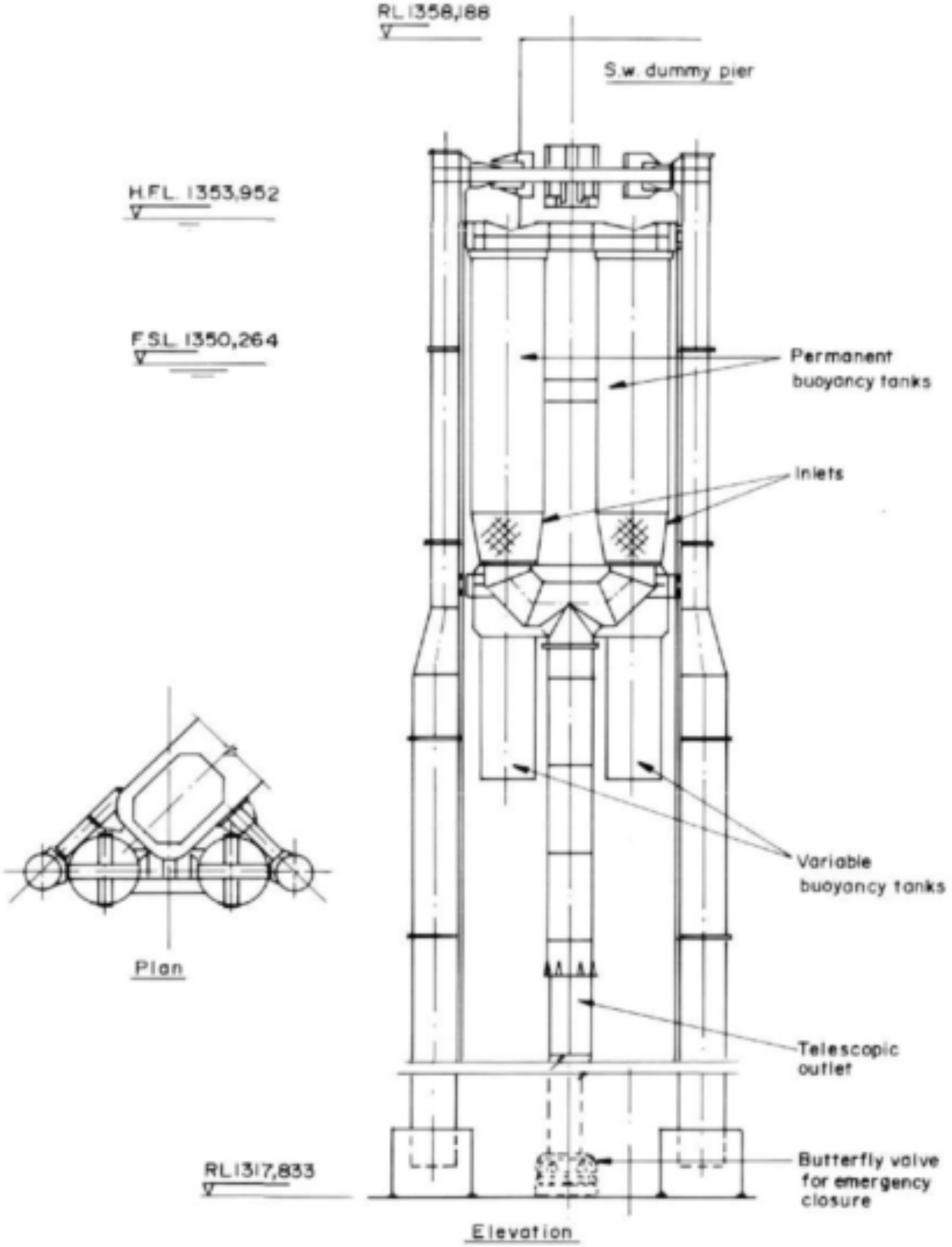


Figure 4.5
Variable level inlet at Ebenezer Dam

At maximum throughflow velocity the retention time is 6,5 minutes.

The sediment disposal outlet consists of a 1 220 mm precast pipeline with a nominal diameter of 900 mm and a 100,6 m concrete chute. The maximum design flow is 2,4 m/s. The sand trap units are desludged one at a time. The mean energy gradient of the sediment disposal pipe is 0,024. The full flow, pipe size was determined from the Hazen-Williams equation using a friction factor of 120, to be 750 mm .

4.5.2 VARIABLE LEVEL INTAKE

The water purification works at Ebenezer Dam was initially fed through bottom draw-offs. Process and operational problems were subsequently experienced with iron, manganese and odours.

The 35 Mℓ/d purification process consists of only four rapid gravity filters as the turbidity of the water ranges from 2,5 to 10 units. The colour of the water is some 5 Hazen units, indicating a low trophic condition.

The metals are present in a dissolved state, due to anaerobic conditions at depths in excess of 7 m. Iron was present at a concentration of 12 mg/ℓ and the manganese concentration was 5 mg/ℓ at the original 32-m intake level. Activated carbon was considered but it was decided to abandon the existing inlet and to construct a new one.

A telescopic outlet pipe system was designed and built as shown in Figure 4.5. The variable level draft system is operated by means of a set of adjustable and a set of permanent buoyancy tanks. Water is abstracted through this outlet at a depth of 5 m below surface water level.

The flow velocity in the draw-off is limited to 3 m/s .

From the equation $h = flv^2/2gd$ a pipe diameter of 500 mm was found to be suitable.

4.5.3 RIVER INTAKE

A river bank intake has been provided on the Vaal River to a 10 Mℓ/d water purification works. The fixed intake consists of a training wall and a gated inlet to the pump station. As shown in Figure 4.6 the inlet has a fixed rack protection.

The orientation of the inlet is 120° downstream to the river flow in order to facilitate self cleansing.

The inlet gate is 800 mm wide x 1 000 mm deep. This yields a flow velocity of 0,2 m/s, which is acceptable.

A mild steel rack is installed ahead of the gate at a rake of 45°. The rack consists of 5 x 40 mm bars held at 50 mm spacing by 12 x 20 mm bars at 150 mm spacing.

A freeboard of 2, 200 m is provided above the inlet gate to conform to the surrounding river bank and environment.

Three pumps including one stand-by, deliver the raw water via a 400 mm pipe to the water works, over a distance of 2 700 m.

5

Pretreatment

G E Rencken

The objective of pretreatment is to reduce or obviate in-plant problems and to optimise the subsequent processes and operations. Raw water quality and the required quality of the product water will determine the need for, nature and amount of pretreatment. As used here, pretreatment precedes coagulation, flocculation, sedimentation, clarification, filtration, disinfection and pH correction.

5.1 RAW WATER QUALITY FACTORS

The following raw water quality factors which will influence the type of pretreatment are considered in this chapter:

- extreme suspended solids concentrations, i.e. very high and very low,
- taste and odour,
- organics,
- iron and manganese,
- algae,
- colour and
- low/high pH.

The pretreatment options for each of these raw water quality factors are shown in Table 5.1.

The pretreatment options for

TABLE 5.1
PRETREATMENT OPTIONS FOR EACH RAW WATER QUALITY FACTOR

Raw water quality factor	Pretreatment option
Very high suspended solids concentration (turbidity)	Storage Pre-sedimentation Roughing filtration
Very low suspended solids concentration (turbidity)	Bentonite (coagulant/ sedimentation aid) dosing
Taste and odour	Impounding reservoir management Adsorption Pre-oxidation Aeration
Organics	Impounding reservoir management Adsorption Pre-oxidation Aeration/stripping
Iron and manganese	Impounding reservoir management Aeration Pre-oxidation
Algae	Impounding reservoir management Micro-straining Dissolved air flotation Pre-oxidation
Colour	Pre-oxidation
Low/high pH	Addition of alkali or acid

each raw water quality factor in Table 5.1 are discussed in this chapter. No further mention is made of pH correction, however, since it is dealt with in Chapter 13.

The details of some unit operations, such as adsorption and oxidation, are discussed in Chapter 16 (**Other Processes**).

5.2 SUSPENDED SOLIDS REMOVAL

A common pretreatment requirement in South Africa is the removal of suspended solids or turbidity upstream of the main water purification works. This is especially so where thunderstorms and flash floods lead to drastic increases in suspended solids. The methods of removal include:

- storage;
- pre-sedimentation;
- roughing filtration;
- use of high molecular weight polyelectrolytes.

The turbidities of the raw water for these methods of suspended solids removal, are indicated in Table 5.2.

TABLE 5.2
TURBIDITY RANGES FOR VARIOUS METHODS OF PRETREATMENT

Pretreatment	Turbidity range NTU
Storage	>1 000
Pre-sedimentation	100 to 1 000
Roughing filtration	20 to 150

Note: These numbers have been adjusted upwards for South African conditions (Van Duuren, 1992)

5.2.1 STORAGE

Impounding reservoirs can be used for presedimentation by storage. The residence time is then generally greater than that in sedimentation tanks, ranging from about one week to a number of months. Storage is the preferred pretreatment for turbid (average annual turbidity over 1 000 NTU) rivers or streams.

Storage serves several purposes, including:

- reduction of turbidity by natural sedimentation;
- attenuation of sudden fluctuations in raw water quality;
- reduction of the number of pathogenic bacteria.

However, storage may cause biotic growth problems if the water is eutrophic and the storage period is substantial. This is the case at Hartebeespoort Dam where either algae or hyacinths cause continuous problems.

The capacity required for the reduction of turbidity by storage is usually exceeded by the capacity required for consumption purposes. There is a reduction of turbidity of the water stored by Vaal Dam, but this was not the objective. After hurricane Demoina, the turbidity of the water in Lake Nsese increased substantially, indicating that the natural storage was inadequate. The Nooitgedacht Dam on the Caledon River is rapidly silting up, again indicating the inadequacy of storage where turbidity is massive (in excess of 5 000 NTU at times).

Despite the existence of Bloemhof Dam on the Vaal River, provision was made for pre-sedimentation in the water works. This was done because the turbidity in the flow through the impounding reservoir could be high (> 1 000 NTU) in times of flood (Van Duuren, 1992).

5.2.2 PRE-SEDIMENTATION

This method of solids removal is required to reduce concentration without coagulants. The particle size would therefore be in excess of 0,01 mm and removal depends on sedimentation rate.

Particle sedimentation rates can be calculated with the Stokes equation (9.1) for laminar flow. Sand, silt and clay, having a specific density of 2,65 and grain size of 10^{-2} mm would have a settling velocity of 6,9 x 10^{-2} mm/s and would require 12,5 hours to settle through a depth of 3 m. By contrast, alum and iron flocs with a relative density of 1,002 and 1 mm diameter will settle at 8,3 x 10^{-1} mm/s or some 3 m/h with obvious implications.

However, it is advisable to carry out tests on the particular suspension which has to be clarified. This is done by means of 2 to 5 m high x 200 mm diameter cylinders and measurement of turbidity removal versus time. The results are represented in graph form for use in design. Clearly the relationship between these results and the sedimentation tank will have to be assessed in terms of the hydraulics of the tank. (Inclined tubes can be used to enhance the hydraulic performance). The efficiency of removal of a poor tank would be some 50%; a good tank will have up to 80% efficiency.

Design criteria (Schultz & Okun, 1990) for pre-sedimentation tanks are presented in Table 5.3.

Pre-sedimentation basins are usually designed for manual sludge removal (inclined tubes will result in a concentration of sludge close to the tank outlet). The design is usually based on sludge removal once a year. Volumetric determination of solids content of the raw water will facilitate calculation of the sludge volume to be designed for.

TABLE 5.3
DESIGN CRITERIA FOR PRE-SEDIMENTATION BASINS

Parameter	Range of values
Detention time	0,5 to 3 h
Upflow velocity	1 to 3 m/h
Depth of basin	1,5 to 2,5 m
Length to width ratio	4:1 to 6:1
Length to depth ratio	5:1 to 20:1

Note: Dimensions metricated and rounded off: upflow velocity used instead of 'surface loading'.

5.2.3 ROUGHING FILTRATION

Roughing filters can be used ahead of slow sand filters because of their effectiveness in removing suspended solids (Schultz & Okun, 1990). They allow penetration of suspended material readily into the filter bed and because of their depth have a large storage capacity.

Roughing filters use larger media than slow or rapid sand filters, as indicated by the comparison below:

• Slow sand filters	0,15 to 0,35 mm
• Rapid sand filters	0,4 to 0,7 mm
• Roughing filters	> 2 mm diameter

The two types of roughing filter are vertical and horizontal flow. Upflow is more desirable in roughing filters because flow is then in the direction of reducing media size and reducing turbidity.

According to Boller (1993), roughing filters ahead of slow sand filters have become very popular in many Third World countries for the following reasons:

- roughing filters do not require addition of flocculants in order to separate solids

to the extent necessary for downstream slow sand filters;
- roughing filters do not require highly qualified personnel to build and operate them.

Although roughing filters have not been used extensively in South Africa, it is a technology which merits further consideration, especially for the supply of drinking water to the less developed regions of South Africa.

5.2.3.1 Vertical flow roughing filters

From the bottom to the top of an upflow roughing filter, the following grading of stone can be used:

- In the bottom 0,75 m of the filter: 10 to 15 mm
- 0,75 to 1,5 m depth of the filter: 7 to 10 mm
- 1,5 to 2,0 m depth of the filter: 4 to 7 mm

Filtration rates in upflow filters are up to 20 m/h. The large pore space enables the high flow rate. Cleansing by back-washing is lower than for rapid filters (20 m/h) and the duration is longer (up to 30 minutes). Fast draining of these filters before backwashing improves cleaning of the filters.

5.2.3.2 Horizontal flow roughing filters

According to Schultz and Okun (1990) these filters are relatively long and filter operation can extend over a period of years between cleansing. The media grading is from coarse (40 mm) at the inlet to fine (4 mm) at the outlet end. The bed depth is 1,0 to 1,5 m. The media sizes can be zoned over a total length of 9 to 12 m. The first zone is the longest in order to store most sediment. The zones are as follows:

- Coarse size fraction in the first zone of 4,5 to 6,0 m length
- Medium size fraction in the middle zone of 3,0 to 4,0 m length
- Fine size fraction over the last zone of 1,5 to 2,0 m length.

The horizontal roughing filter flow rate ranges from 0,5 to 4,0 m/h, based on cross-flow area.

5.2.4 USE OF HIGH MOLECULAR WEIGHT POLYELECTROLYTES

Some authorities in South Africa, such as Goldfields Water Board and Phalaborwa, have been successful in using high-molecular weight polyelectrolytes as coagulants during periods of high turbidities (up to 1000 NTU) (Langenegger, 1995). In contrast to other water authorities mentioned earlier, these authorities have not found it necessary to install significant pre-storage or other presedimentation facilities for suspended solids removal during periods of high turbidities. In addition to the dosing of high molecular weight polyelectrolytes, the clarifiers have been designed to have sufficient "desludging capacity" during periods of high turbidity. The design rise rates for the clarifiers are 1 to 1,5 m/h (Langenegger, 1995).

5.3 RAW WATER WITH VERY LOW SUSPENDED SOLIDS CONCENTRATION

The coagulation/flocculation of raw water with very low suspended solids concentrations (turbidities) is sometimes difficult. During the dry winter months the turbidity of the raw water in Umgeni Water's Henley

Dam is at times as low as 2 NTU. At times it is difficult to achieve the desired final water turbidity at the HD Hill water treatment plant (which abstracts water from Henley Dam), using conventional coagulants/flocculants. The dosing of a dilute bentonite clay slurry (0,1% m/m) at a bentonite dose rate of 1 to 2 mg/ℓ at the head of the water treatment plant prior to coagulation makes it possible to achieve the required final water turbidity of less than 0,5 NTU (according to WHO standards). However, at the Western Transvaal Regional Water Company where the total dissolved solids concentration of the raw water is in excess of 700 mg/ℓ, the addition of bentonite has not been successful (Krüger, 1995).

Bentonite clays may also be used as weighting agents when the floc produced is very light and will not settle well in a clarifier.

5.4 TASTE AND ODOUR REMOVAL

Decaying vegetation and metabolites of microbiota are probably the most universal sources of taste and odour problems in surface water. The organisms most often linked to taste and odour problems are the bacteria *ACTINOMYCETES* and blue-green algae (AWWA, 1990). A comprehensive list of algae-related taste and odour problems is given in AWWA (1990).

The two most common taste- and odour-causing metabolites of *ACTINOMYCETES* and blue-green algae are geosmin and 2-methylisoborneol. These compounds cause an earthy-musty odour in water supplies. Both geosmin and 2-methylisoborneol can have odour threshold concentrations of less than 10 ng/ℓ.

When there is a deficiency of oxygen in deeper water layers in a storage, a highly unpleasant odour of hydrogen sulphide may occur as the result of anaerobic bacterial action on sulphates, as shown by reaction (5.1). This rotten egg odour can be detected at less than 100 ng/ℓ (AWWA, 1990).

$$6e^- + 8H^+ + SO_4^{2-} \leftrightarrow$$
$$H_2S\,(g) + 2H_2O + 2OH \text{ .... } (5.1)$$

Apart from the compounds listed above, it should be noted that there is a multitude of natural and synthetic organic and inorganic compounds, which can lead to taste and odour problems and it is impossible to list them all; for further information the reader is encouraged to refer to AWWA (1990).

5.4.1 IMPOUNDING RESERVOIR MANAGEMENT

There are several techniques of impounding reservoir management which can be applied to combat taste and odour problems.

Destratification of reservoirs can be effectively used to combat both turnover and anaerobic conditions and in some cases to control algae (see Section 5.7.1).

Mixing techniques to accomplish destratification include diffused air aeration and mechanical or hydraulic mixing. Aeration is a process in which air is injected over wide areas of the impounding reservoir to promote mixing. In mechanical or hydraulic mixing, water is pumped from one level of the impounding reservoir to another area of different density. Both aeration and mechanical or hydraulic mixing cause a circulation and a mixing action that results in a homogeneous water column and removes anaerobic conditions in the bottom waters. Mechanical or hydraulic mixing systems generally have been found to be more costly to operate and, therefore, are less widely used than diffused air systems (AWWA, 1990).

Although the application of these impounding reservoir management techniques has been limited in South Africa, considerable success has been achieved overseas in this regard. In a study on the Occoquan Reservoir in Virginia, USA, it was demonstrated that impounding reservoir aeration had (amongst others) the following important benefits (AWWA, 1990):

- elimination of anaerobic conditions and the resultant generation of H_2S;
- reduction in the amount of powdered activated carbon (PAC) required to combat taste and odour problems;
- ability to draw from the bottom intake because of improved quality.

5.4.2 ADSORPTION

Both granular activated carbon (GAC) and powdered activated carbon (PAC) may be used to remove organic taste and odour compounds such as geosmin and 2-methylisoborneol. However, only PAC is used for pretreatment. The main advantage of PAC is that a PAC dosing system may be installed relatively easily at the head of a water purification works and the capital costs are much lower than those required for GAC. Another advantage of PAC is that the dose may be changed as the water quality changes. This is especially important for systems that do not require an adsorbent for much of the year.

Adsorption theory and further technical details for PAC and GAC are described in Chapter 16.

The disadvantages of PAC dosing are high operating costs (if high doses are required for long periods of time) and increased sludge dewatering and sludge disposal costs.

The most important process variables which have to be considered for PAC dosing are:

- particle size of PAC;
- internal surface area of PAC;
- contact time of PAC with water;
- point of application of PAC.

5.4.2.1 Particle size of PAC

The particle size of PAC can greatly influence the rates of adsorption. A compromise has to be found between the higher rate of adsorption for fine carbons and the ability to capture the particles in the sand filters at the water purification works.

5.4.2.2 Internal surface area of PAC

In practice the iodine number serves as an indication of the surface area which is available for the adsorption of small molecules. The iodine number measures the amount of iodine which will adsorb under a specific set of conditions. The iodine number for PAC which is suited to taste and odour removal should be greater than 500 (Freese, 1994).

5.4.2.3 Contact time of PAC with water

Laboratory tests (known as isotherm tests) are necessary to determine the contact time for maximum efficiency. An aliquot of water is mixed with a given dose of activated carbon and stirred or shaken at constant temperature for various time intervals. After the carbon has been filtered off, the residual concentration of the undesired compound is obtained. From a graph of fraction of contaminant removed versus contact time an assumption can be made as to the required contact time. Typical contact times are in the range of 10 to 30 minutes.

TABLE 5.4
ADVANTAGES AND DISADVANTAGES OF DIFFERENT POINTS OF PAC ADDITION

Points of addition	Advantages	Disadvantages
Intake	Long contact time; good mixing	Some substances may adsorb that otherwise would be removed by coagulation, thus increasing the activated carbon usage rate.
Rapid mix	Good mixing during rapid mix and flocculation; reasonable contact time	Possible reduction in rate of adsorption because of interference by coagulants. Contact time may be too short for equilibrium to be reached for some contaminants. Some competition may occur from molecules that otherwise would be removed by coagulation.
Filter inlet	Efficient use of PAC	Possible loss of PAC to the clear well and distribution system.
Slurry contactor preceding rapid mix	Excellent mixing for the design contact time; no interference by coagulants; additional contact time possible during flocculation and sedimentation	A new basin and mixer may have to be installed. Some competition may occur from molecules that otherwise would be removed by coagulants.

5.4.2.4 Point of application of PAC

Important criteria for selecting the point of PAC addition include, (AWWA, 1990):

- the provision of good mixing between the PAC and the water being treated;
- sufficient contact time for contaminant adsorption;
- minimal interference of treatment chemicals with adsorption on PAC;
- no degradation of final water quality.

The main advantages and disadvantages of different points of PAC addition are shown in Table 5.4, which is taken from AWWA (1990).

5.4.2.5 Examples of PAC pretreatment for taste and odour removal

Random incidents of taste and odour have occurred in the raw water supplied from

Nagle Dam to Umgeni Water's Durban Heights water purification works since 1986. These taste and odour problems have been ascribed to the occurrence of geosmin and 2-methylisoborneol.

PAC dosing at the head of the works has been successful in removing both the geosmin and 2-methylisoborneol to non-detectable concentrations. Dosing rates have been in the range of 2 to 17 mg/ℓ.

5.4.3 PRE-OXIDATION

Oxidants which may be used for taste and odour control include chlorine, ozone and chlorine dioxide.

Theoretical and technical aspects of ozone and chlorine dioxide generation and dosing are discussed in detail in Chapter 16.

Chlorine is generally not as effective for taste and odour control as ozone and chlorine dioxide. In fact, chlorine addition often adds new tastes and odours to the water and may also form chlorinated by-products (see Section 5.5).

Of the three oxidants mentioned above, ozone appears to be the most effective at destroying some of the recalcitrant taste and odour compounds, particularly geosmin and 2-methylisoborneol (AWWA, 1990). However, Masschelein (1992) points out that musty tastes associated with geosmin, including 2-methylisoborneol, are not always improved by ozonation. At the Western Transvaal Regional Water Company, a reduction of 20% in the 2-methylisoborneol has been recorded with an ozone dose of approximately 1,7 mg/ℓ (Krüger, 1995).

Significant research has been done in the past few years on so-called advanced oxidation processes. Among the most promising processes are ozone decomposition initiated by ultra-violet (UV) radiation and by hydrogen peroxide. Both ozone/UV and ozone/peroxide have been shown to be more effective than ozone for the destruction of taste and odour compounds such as geosmin and 2-methylisoborneol that resist other oxidants (AWWA, 1990).

5.4.4 AERATION

Theoretical and technical aspects of aeration as well as the different types of aeration devices or unit operations are described in detail in Chapter 16.

Aeration for taste and odour control is used for the removal of hydrogen sulphide from anaerobic waters.

Aeration or stripping can only remove that portion of the gas that is in the gaseous phase. Hydrogen sulphide reacts with water as follows (AWWA, 1990):

$$H_2S\,(g) + H_2O \leftrightarrow H_3O^{+} + HS \quad \ldots\ldots\ldots\ldots (5.2)$$
$$K_1 = 1 \times 10^{-7}\ (25°C)$$

$$HS + H_2O \leftrightarrow H_3O^{+} + S^{2} \quad \ldots\ldots\ldots\ldots (5.3)$$
$$K_2 = 1 \times 10^{-15}\ (25°C)$$

The amount of total sulphide present as H_2S can be calculated by (AWWA, 1990):

$$\%\ H_2S = 100\,\frac{[H^{+}]^{2}}{[H^{+}]^{2}+K_1[H^{+}]+K_1K_2} \quad \ldots\ldots\ldots\ldots (5.4)$$

At pH 8 only 10% of the total sulphide present is H_2S. Therefore, the most that can be removed by aeration is 10% if the aerator is 100% efficient. At pH 7, approximately 50% of the H_2S is removable.

Some of the remaining sulphide may be oxidised by oxygen added to the water during the stripping process:

$$2H_2S\,(g) + O_2 \leftrightarrow 2H_2O + 2S(s) \quad \ldots\ldots\ldots (5.5)$$

This sulphur may be further oxidised to sulphate, however, and then reduced back to sulphide in the distribution system.

5.5 ORGANIC CONSTITUENTS

Organic compounds in water come from three major sources (AWWA, 1990):

- breakdown of naturally occurring organic materials;
- waste water from domestic and industrial activities;
- organic reactions that occur during water treatment.

The first source comprises humic materials, micro-organisms and their metabolites, e.g. geosmin and 2-methylisoborneol mentioned in Section 5.4. Humic organic compounds, such as are prevalent in Cape brown waters, can serve as precursors to the formation of trihalomethanes (THMs) and other organohalogen oxidation products during oxidation and disinfection with chlorine.

Organic compounds derived from domestic and industrial activities are constituents of waste-water discharges, agricultural and urban runoff. They include pesticides and solvents. A substantial list of organic compounds that may be present in water is given in AWWA (1990).

Organic contaminants formed during water treatment include disinfection by-products such as trihalomethanes (THMs) and other chlorinated organics that have been identified only recently.

5.5.1 IMPOUNDING RESERVOIR MANAGEMENT

By applying algae control techniques (see Section 5.7.1) in an impounding reservoir the organic metabolites of algae cells may be controlled or reduced.

5.5.2 ADSORPTION

The adsorption pretreatment technique for the removal of organic compounds from raw water is the dosing of PAC. The principles of PAC dosing as a pretreatment option have already been mentioned in section 5.4.2 for taste and odour removal.

The predominant reason for the use of PAC is to control taste and odour. Trihalomethane precursor removal using PAC (measured in terms of trihalomethane formation potential (THMFP) has not been very high (AWWA, 1990), possibly because removal is limited by the slow kinetics of adsorption and equilibrium capacity. At Contra Costa, California, addition of up to 40 mg/ℓ of PAC showed no effect on THMFP removal, while at Newport News in Vancouver a dose of 18 to 25 mg/ℓ of PAC increased the removal of THMFP by 9 to 20% over that which could be achieved by coagulation alone (AWWA, 1990).

One of the techniques to improve the ability of PAC to adsorb slow-diffusing compounds such as THM precursors, is to add the PAC upstream of a sludge blanket clarifier (AWWA, 1990). This has the potential to improve adsorption efficiency because the carbon can be kept in contact with the water for a longer time.

Pesticides and herbicides are generally removed quite well by PAC dosing (AWWA, 1990).

5.5.3 PRE-OXIDATION

5.5.3.1 Oxidation of THM precursors

THMs and other halogenated organics are produced by the reaction of chlorine with natural and synthetic organics. Both chlorine and natural organics are needed for

THM formation. A useful treatment strategy is to use chlorine dioxide or ozone as a pre-oxidant in place of chlorine to oxidise THM precursors so that chlorine could be used later in the treatment process.

Under certain conditions, however, ozonation can actually increase the THM formation potential, while under other conditions substantial reductions can occur (AWWA, 1990). Because of water quality variations, particularly humic composition and bicarbonate levels (AWWA, 1990), each case must be treated individually and laboratory or pilot-scale trials are highly recommended.

For further information on the oxidation of THM precursors using chlorine dioxide and ozone, the reader is encouraged to read Masschelein (1992) and Langlais *et al.* (1991).

5.5.3.2 Oxidation of phenols and synthetic organic compounds

Strong chemical oxidants are capable of reacting with synthetic organic chemicals present in raw waters. Chemical oxidation of synthetic organic compounds is highly dependent on the nature of the organic compound, the oxidant and other contaminants in the water. Some organic compounds are relatively easy to oxidise while others are very resistant, e.g. phenolic compounds react readily with chlorine, ozone and chlorine dioxide (AWWA, 1990). With chlorine, objectionable tastes and odours can occur because of the formation of chlorinated phenols. For chlorine dioxide and ozone, complete oxidation will yield small polar molecules, but at lower doses partially oxidised species are formed (AWWA, 1990).

Ozone and other common oxidants are theoretically capable of reacting with other synthetic organics, but they are highly selective and the rates of reaction are low. This is especially true when synthetic pollutants present in concentrations of µg/ℓ are competing with natural contaminants with concentrations in mg/ℓ.

The so-called advanced oxidation processes of ozone/UV and ozone/peroxide (see Section 5.4.3) are sometimes more successful than either ozone or chlorine dioxide on their own in oxidising synthetic organic compounds.

The oxidation of synthetic organics using ozone and chlorine dioxide involves complex chemistry and the reader is encouraged to refer to Masschelein (1992) and/or Langlais *et al.* (1991) for further information.

5.5.4 AERATION/STRIPPING

Theoretical and technical aspects of aeration as well as the different types of aeration devices or unit operations are described in detail in Chapter 16.

Traditionally aeration has been used to remove hydrogen sulphide for taste and odour control (as described in Section 5.4.4) as well as to remove iron and manganese (see Section 5.6.2). However, in recent years, aeration has been used to strip volatile organic compounds (VOC), including THMs from water.

Packed adsorption columns are the best air/water contactor or stripping device for the removal of VOC and THMs from water (AWWA, 1990). An excellent summary for the design of such towers is given by AWWA (1990).

Diffused aeration has found some use in the removal of VOC. However, diffused aeration has a much higher power cost for VOC removal than packed adsorption columns (AWWA, 1990). For this reason, it is generally only considered when the diffused aeration process can take place in existing tanks, thereby eliminating much of the capital cost associated with packed adsorption columns.

5.5.4.1 Off-gas treatment

According to AWWA (1990) the most cost-effective method to remove VOC is air stripping. Recently concern has been expressed, however, about the resulting VOC air pollution from the off-gas associated with air stripping operations, mainly by packed adsorption columns. One method of removing VOC from off-gas is by adsorption of the VOC in the off-gas onto GAC.

According to AWWA (1990), the gas phase adsorption of VOC is more efficient than aqueous phase adsorption onto GAC, provided the off-gas is heated prior to adsorption to remove humidity. Also, carbon consumption is less than half that for aqueous adsorption.

5.6 IRON AND MANGANESE REMOVAL

Stratification in storage reservoirs leads to the formation of an anaerobic layer (referred to as the hypolimnion) at the bottom of the impounding reservoir as described in Section 5.4.1. In the anaerobic hypolimnion, iron and manganese are present as soluble ferrous ions (Fe^{2+}) and manganous ions (Mn^{2+}) respectively. The soluble ferrous and manganous ions may also be complexed to various organic and inorganic species.

The same may apply to groundwater.

Ferrous and manganous ions can oxidise and precipitate in the distribution system, possibly causing fouling and discolouration problems.

The ferrous and manganous ions have to be oxidised prior to coagulation/flocculation at the water purification works.

Techniques and technologies for iron and manganese removal are described in detail in Chapter 16. The most important pretreatment options for iron and manganese will therefore be described briefly in this chapter.

5.6.1 IMPOUNDING RESERVOIR MANAGEMENT

The mixing and aeration destratification techniques described in Section 5.4.1 may be used to prevent anaerobic conditions in a storage reservoir and therefore the solubilisation of iron and manganese.

5.6.2 AERATION

The oxidation of iron (II) and manganese (II) by molecular oxygen may be described by the following equations:

$$4Fe^{2+} + O_2\,(g) + 10H_2O \leftrightarrow 4Fe(OH)_3(s) + 8H^+ \qquad (5.6)$$

$$2Mn^{2+} + O_2\,(g) + 2H_2O \leftrightarrow 2MnO_2(s) + 4H^+ \qquad (5.7)$$

The insoluble precipitates, $Fe(OH)_3$ or MnO_2, may subsequently be removed by filtration.

At a pH equal to or greater than about 5, the following rate law is applicable to the oxidation of iron (II) with oxygen (AWWA, 1990):

$$= -k\,(PO_2)\,[OH^-]\,[Fe^{2+}] \qquad (5.8)$$

where:

= rate of iron (II) oxidation (mol/ℓ.min)
k = reaction rate constant
= 8,0 (±2,5) x 10^{13} ℓ^2/min.atm.mol at 20,5°C
PO_2 = partial pressure of oxygen (atm)
$[OH^-]$ = hydroxide ion concentration (mol/ℓ)
$[Fe^{2+}]$ = iron (II) concentration, (mol/ℓ)

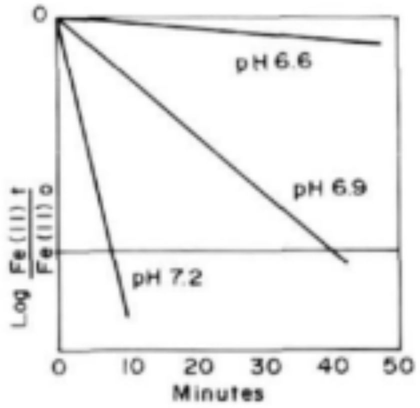


Figure 5.1
Oxidation rate of iron at various pH values

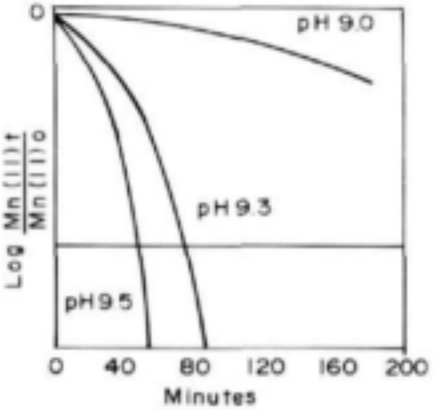


Figure 5.2
Oxidation rate of manganese at various pH values

The oxidation of iron with molecular oxygen is shown in Figure 5.1.

As shown in Figure 5.1, the oxidation of iron (II) is very dependent on pH.

Equation (5.8) and Figure 5.1 are based on uncomplexed iron. Various humic compounds and similar materials can act to complex the iron and to slow down the kinetics of oxidation (Montgomery, 1985). Strongly complexed iron is not readily oxidised with oxygen and more powerful oxidants are then required (see Section 5.6.3).

The oxidation of uncomplexed manganese (II) with molecular oxygen is shown in Figure 5.2. The reaction is very slow and in fact impractical, below pH 9,5. Generally, a stronger oxidant than oxygen is required to remove manganese (AWWA, 1990) (see Section 5.6.3).

5.6.3 PRE-OXIDATION

Because of the slow reaction kinetics with molecular oxygen and the fact that iron (II) and manganese(II) are often strongly complexed, chemical oxidants are often added at the beginning of the treatment process to oxidise iron and manganese.

Common oxidants which may be used are chlorine, potassium permanganate, ozone and chlorine dioxide.

5.6.3.1 Pre-oxidation with chlorine

The oxidation of uncomplexed iron (II) and manganese (II) using chlorine is shown in Equations (5.9) and (5.10) respectively:

$$2Fe^{2+} + Cl_2\,(g) + 6H_2O \leftrightarrow$$
$$2Fe(OH)_3\,(s) + 2Cl^- + 6H^+ \quad (5.9)$$

$$Mn^{2+} + Cl_2\,(g) + 2H_2O \leftrightarrow$$
$$MnO_2\,(s) + 2Cl^- + 4H^+ \quad (5.10)$$

According to AWWA (1990) very little has been reported in the literature with regard to the oxidation kinetics of iron (II) and manganese (II) with chlorine, because the oxidation of both metals is quite rapid at pH 7 and higher. Thus, kinetic considerations have little influence on either design or operation.

In general, chlorine is now less favoured as a pre-oxidant, because of the potential formation of THMs.

5.6.3.2 Pre-oxidation with potassium permanganate

The oxidation of uncomplexed iron (II) and manganese (II) using potassium permanganate is shown in Equations (5.11) and (5.12) respectively:

$$3Fe^{2+} + KMnO_4 + 7H_2O \leftrightarrow$$
$$3Fe(OH)_3\,(s) + MnO_2\,(s) + K^+ + H^+ \ldots\ldots (5.11)$$

$$3Mn^{2+} + 2KMnO_4 + 2H_2O \leftrightarrow$$
$$5MnO_2\,(s) + 2K^+ + 4H^+ \ldots\ldots (5.12)$$

According to AWWA (1990), as for chlorine, very little has been reported in the literature on the kinetics of oxidation of iron (II) and manganese (II) with potassium permanganate, since the oxidation of both metals is quite rapid at pH 7 and higher. During periods of high algae concentrations in the raw water at the Western Transvaal Regional Water Company, it was found that potassium permanganate was more specific or selective than either chlorine or ozone, for the oxidation of manganous ions (Krüger, 1995).

5.6.3.3 Pre-oxidation with ozone

The chemical reaction of ferrous ions with ozone may be written as (Masschelein, 1992):

$$Fe^{2+} + O_3\,(g) + H_2O \leftrightarrow Fe^{3+} + O_2 + 2OH^- \ldots (5.13)$$

$$Fe^{3+} + 3H_2O \leftrightarrow Fe(OH)_3\,(s) + 3H^+ \ldots\ldots (5.14)$$

The oxidation of manganous ions by ozone in the pH range 5 to 7 is described by the following chemical reaction (Masschelein, 1992):

$$Mn^{2+} + O_3\,(g) + H_2O \leftrightarrow$$
$$Mn^{4+} + O_2\,(g) + 2OH^- \ldots\ldots (5.15)$$

$$Mn^{4+} + 4OH^- \leftrightarrow MnO_2\,(s) + 2H_2O \ldots\ldots (5.16)$$

However, over-oxidation should be avoided otherwise soluble permanganate may be formed (Masschelein, 1992):

$$2Mn^{2+} + 5O_3\,(g) + 3H_2O \leftrightarrow$$
$$2MnO_4^- + 5O_2\,(g) + 6H^+ \ldots\ldots (5.17)$$

This may give a pink colour to the water. Ozone has been used successfully for manganous ion oxidation at the Western Transvaal Regional Water Company. Pietersen *et al.* (1993) found that ozone was superior to chlorine for the oxidation and removal of manganese from eutrophic middle-Vaal River water at the Western Transvaal Regional Water Company. Pre-oxidation with chlorine and ozone also resulted in enhanced manganous ion oxidation on the filter sand (which was covered with a black MnO_2 layer) at the plant.

5.6.3.4 Pre-oxidation with chlorine dioxide

The oxidation of ferrous ions using chlorine dioxide is described by the following equation:

$$ClO_2 + Fe^{2+} + 3H_2O \leftrightarrow$$
$$Fe(OH)_3(s) + ClO_2^- + 3H^+ \ldots\ldots (5.18)$$

Contrary to ozone, the oxidation of manganous ions using chlorine dioxide does not proceed further than the Mn^{4+} state (Masschelein, 1992):

$$2ClO_2 + Mn^{2+} + 4OH^- \leftrightarrow$$
$$MnO_2\,(s) + 2ClO_2^- + 2H_2O \ldots\ldots (5.19)$$

5.7 ALGAE REMOVAL

5.7.1 IMPOUNDING RESERVOIR MANAGEMENT

The most common indicator of eutrophication in a storage reservoir is the presence of algae. Operational problems which may occur as a result of the presence of algae in storage dams include taste and odour problems, high concentrations of THM precursors and filter clogging at the water purification works (AWWA, 1990).

Algae in a storage reservoir may be controlled by the dosing of copper sulphate (AWWA, 1990). However, caution is required because copper sulphate dosing can have a detrimental effect on fish and other biological life.

Destratification using either mixing or diffused air aeration (see Section 5.4.1) may in some cases also be used to control algae.

5.7.2 MICRO-STRAINING

If the algal concentration in the raw water is not too high, microstraining may be used to remove the bulk of the algae prior to coagulation/flocculation.

A typical microstraining system is shown in Figure 5.3. A typical unit consists of a motor-driven rotating drum mounted horizontally in a pit or vat. The rigid drum support structure has either a stainless steel or plastic woven screen fastened to it. Mesh size is normally in the 10 to 40 µm range. (Degremont, 1979).

Feed passes from the inside to the outside of the drum depositing solids on the inner surface. Water jets on top of the screen dislodge collected solids into a waste hopper. The peripheral drum speed is usually adjustable. Filtration rates are in the range of 30 to 90 m^3/m^2.h while the head loss through the screen is typically 75 to 150 mm

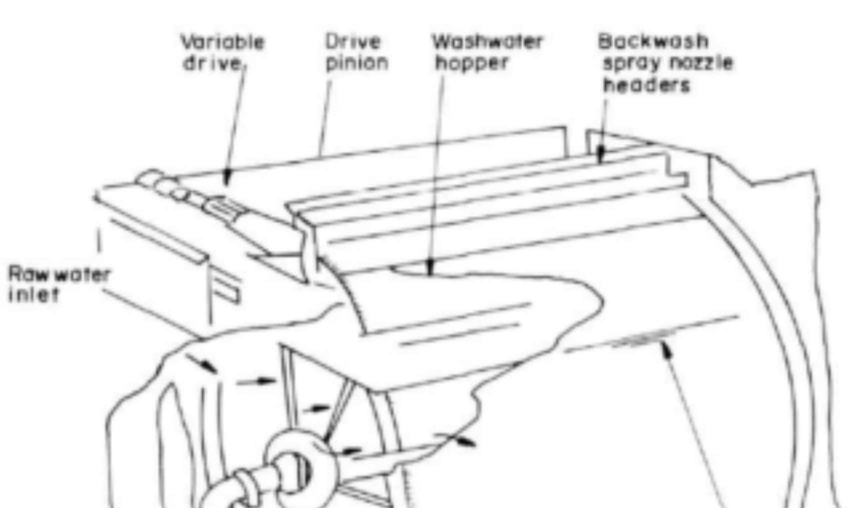


Figure 5.3
Schematic diagram of a microfilter (Nalco, 1979)

of water (Nalco, 1979).

According to Degremont (1979), microstraining is particularly effective for certain types of green algae (e.g. *Pediastrum* and *Scenedesmus*) and removal efficiencies of up to 95% are possible for these types of algae. However, tests done at the Western Transvaal Regional Water Company in 1982/83 indicated that for a mesh size of 10 µm, 80% of the algae passed through the microstrainer. When smaller mesh sizes were used, the microstrainer blocked very quickly (Krüger, 1995), so that microstraining was not considered a viable option for algae removal. From the conflicting reports above, it therefore seems that microstraining may only be viable for relatively low concentrations of certain types of algae. Tests using various mesh sizes are necessary to determine whether microstraining is a viable option for a particular application.

5.7.3 DISSOLVED AIR FLOTATION

Algae removal using dissolved air flotation is described in detail in Chapter 11 and will not be repeated here.

5.7.4 PRE-OXIDATION

Conflicting results on the effect of pre-oxidation on the removal of algae have been reported in the literature.

Sukenik *et al.* (1987) investigated the effects of chlorine, ozone and chlorine dioxide on *Scenedesmus* cultures. Algae cell and chlorophyll concentrations decreased and the concentration of dissolved organic substances increased with increasing ozone dosage.

TABLE 5.5
COMPOSITION OF ALGAE SPECIES IN INANDA DAM WATER AT TIME OF TEST

Species	Number of cells	Composition
Microcystis	389 268	99,8
Nitzschia	590.00	0,2

TABLE 5.6
COMPOSITION OF ALGAE SPECIES IN SHONGWENI DAM WATER AT TIME OF TEST

Species	Number of cells (/mℓ)	Composition (%)
Anabaena	154 823	98,08
Microcystis	2 949	1,86
Chlorella	88.00	0,06

Pretreatment with chlorine dioxide or ozone significantly enhanced algae flocculation with alum, while higher doses of chlorine increased the required dosage of alum by approximately 15%.

Several authors have reported the ability of ozone to improve the efficiency of flocculation and algae removal, especially during periods of intense algae blooms (Langlais *et al.*, 1991). However, Hoyer *et al.* (1987) observed that ozonation impaired the flocculation of samples containing cultures of *Dictyosphaerium*, even at very low doses of ozone. For cultures of *Fragilaria* and *Pseudanabaena*, an improvement in flocculation was noticed for relatively low doses of ozone. However, at higher ozone doses, flocculation was impaired.

Rencken (1994) observed divergent trends for the pre-ozonation of algae-laden water from the Inanda and Shongweni dams in Umgeni Water's area of supply.

The effect of ozone dose on coagulant demand for Inanda and Shongweni dams is shown in Figures 5.4 and 5.5, respectively. The coagulant demand in Figures 5.4 and 5.5 is the coagulant dose required to achieve a turbidity of less than 0,5 NTU in the final water. A polyaluminium chloride polyelectrolyte was used for Inanda Dam water, while a normal cationic polyelectrolyte was used for Shongweni Dam water. The typical compositions of algae species in Inanda and Shongweni Dam waters are shown in Tables 5.5 and 5.6, respectively.

As shown in Figure 5.4, for Inanda Dam water, the coagulant demand increased sharply (even at low ozone doses) with ozone dose until a peak was reached, after which the coagulant demand increased with increasing ozone dose.

As shown in Figure 5.5 for Shongweni Dam water, the coagulant demand decreased with increasing ozone dose for relatively low doses of ozone, while for higher ozone dose rates the opposite was true.

The effect of ozone dose on algae counts for *Microcystis* is shown in Table 5.7 for Inanda Dam water. Similar

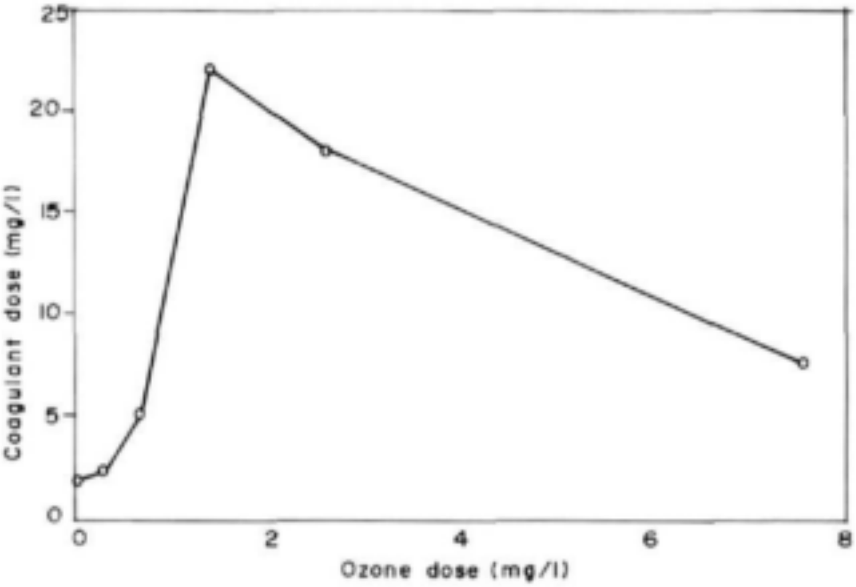


Figure 5.4
Effect of ozone dose on coagulant demand for a batch of Inanda Dam water

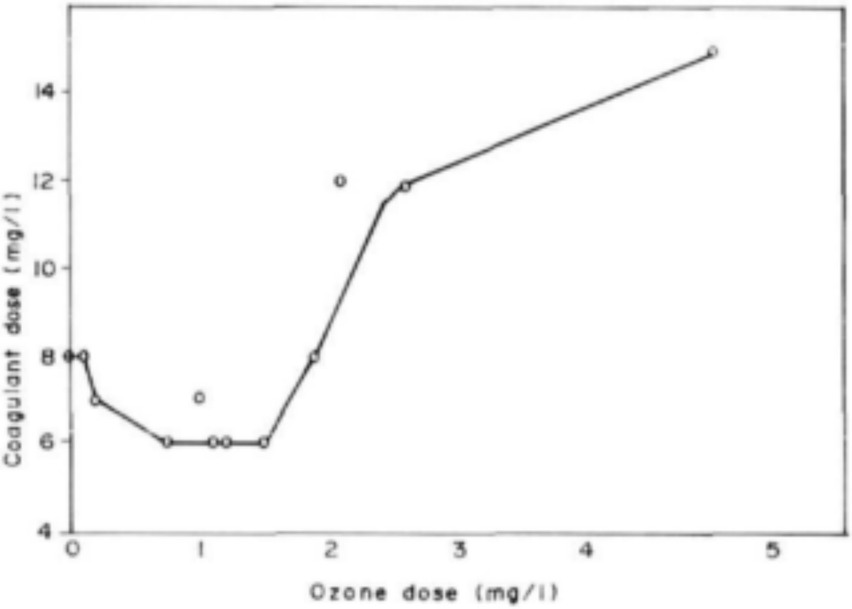


Figure 5.5
Effect of ozone dose on coagulant demand for a batch of Shongweni Dam water

TABLE 5.7
THE EFFECT OF OZONE DOSE ON ALGAE COUNTS (INANDA DAM WATER)

Ozone dose (mg/ℓ)	Number of cells of *Microcystis* (/mℓ)
0,0	389 268
3,2	237 962
5,0	165 881
6,1	92 894
7,6	39 910

trends were observed for Shongweni Dam water.

From the above discussion, it is evident that laboratory- or pilot-scale tests are required before a decision can be made as to whether a pre-oxidant facilitates algal removal for a specific type of raw water and composition of algae species.

5.8 COLOUR REMOVAL

5.8.1 PRE-OXIDATION

Colour in water is caused by the presence of organic and inorganic substances, usually of natural origin, which absorb visible light. In most cases colour is caused by natural organics (humic substances) that have complexed metals bound into their structures (AWWA, 1990).

Pre-oxidation in combination with chemical coagulation/flocculation is usually the pretreatment method of choice for removing colour from water. Of the common oxidants, chlorine, ozone and chlorine dioxide are the most effective.

5.9 FURTHER APPLICATIONS OF PRE-OXIDATION

5.9.1 CONTROL OF BIOLOGICAL GROWTH IN A WATER PURIFICATION WORKS

Water purification works that have sedimentation tanks and filters open to sunlight are especially susceptible to growth of algae and other forms of micro-organisms in sedimentation tanks (clarifiers) and on sand filters.

Pre-chlorination has commonly been used for the control of biological growth, but is now less favoured because of the potential formation of THMs. Ozone and chlorine dioxide are effective substitutes for chlorine, but each has disadvantages. It is usually difficult to maintain an ozone residual throughout a water purification works after pre-ozonation, for example. As a result biological growth may still occur in some areas of the plant. Application of ozone at the head of the works and midway through the treatment process, may be necessary to overcome this problem. It is also possible that ozone will react with natural organics to form organic by-products, which have a lower molecular mass and are more biodegradable than the precursors. As a result, biological growth may actually be enhanced by pre-oxidation.

5.9.2 PRE-OZONATION FOR IMPROVED COAGULATION/FLOCCULATION OF COLLOIDS

Ozone may behave as a coagulant aid during the treatment of water. Numerous European investigators have observed that when ozone is added to raw water, particles are destabilised, small particles are converted to larger ones (referred to in the literature as micro-flocculation) and less coagulant is required to achieve the desired final water quality (Chang & Singer, 1991).

However, according to Chang & Singer (1991), despite the numerous reports on the benefits of ozone as a coagulant aid, there are inconsistencies in the results reported.

Chang & Singer (1991) conducted some excellent research into the impact of ozonation on particle stability. The subject is complex and therefore only the most important conclusions are presented here. The reader who requires more information is encouraged to refer to Chang & Singer's paper.

The main conclusions of Chang & Singer's (1991) research were:

- Water hardness and total organic carbon (TOC) concentration have a major impact on the stability of particulate material. The aggregation rate of suspended particles is very sensitive to the hardness-to-TOC ratio of the unozonated raw water.
- Optimal ozone-induced coagulation occurs in waters with hardness-to-TOC ratios > 25 mg $CaCO_3$/mg C and ozone doses of about 0,4 to 0,8 mg O_3/mg C.

6

Coagulation, Coagulants and Chemicals

F A van Duuren

Colloidal particles in water are electro-negatively charged and in stable suspension. Aggregation of these particles results from the addition of positively charged ions to the suspension resulting in destabilisation of the particles i.e. coagulation. The importance of this step can be deduced from the relevant part of the statement: "If man can categorise impurities which are effectively removed by (coagulation), then a water purification process can be designed rationally" (Tambo, 1990).

Coagulation was differentiated as perikinesis where particle collisions are effected by diffusion and orthokinesis where collision is effected by shear flow (Von Smoluchowski, 1916; 1917).

A coagulant is a chemical species which is used to cause coagulation of colloidal particles. Flocculation is used further on to refer to the orthokinetic, hydrodynamic operation which further aggregates the destabilised particles, primarily by collision, bridging and attachment. A flocculant is by definition a compound which aids flocculation.

Coagulation and flocculation have been identified as a two-step operation, step one being the charge neutralisation process (i.e. destabilisation) by for example hydrolysed polynuclear metal ions and step two the bridging of the destabilised particles by precipitated hydrolysed metal species (Tambo, 1990).

In order for coagulation to be effective, the water to be treated should be tested chemically and assessed as regards the processes required. It may well be found that the water is of such high quality that it requires only disinfection to ensure potability. At very low turbidity and colour, slow sand filtration could be sufficient. At higher turbidity and colour, such as usually pertains for the surface waters of South Africa, coagulation and flocculation are essential. Process selection is dealt with in detail in Chapter 3.

In order to understand its key role, coagulation can be viewed in context with dosing, mixing, flocculation and phase separation. The theory of coagulation, testing and practical application is considered here.

A large range of chemicals is used in water purification and treatment. The chemicals generally used on a water works include coagulants, coagulant aids, flocculants, pH correctors, corrosion inhibitors and disinfectants.

6.1 COAGULATION

Coagulation is a key process of water purification; it is very closely followed by the equally important and essential flocculation operation (Fiessinger, 1978; Van Duuren, 1979; 1990). In order to relate coagulation and flocculation to the preceding and following operations and processes a tabulation of these is given in Table 6.1.

The sequence and nature of the various unit processes (U P) and unit operations (U O)

TABLE 6.1
THE ROLES OF COAGULATION AND FLOCCULATION IN WATER PURIFICATION

Stage	Terminology		Process or operation		Nature
1 **Reagent formation** (reagent is coagulant, flocculant or coagulant aid, if effect starts at 2.4)	Dilution		1.1	Preparation: Dissolution, ionisation, polymerisation	U P
	Flash mixing		1.2	Introducing: Dispersion, diffusion Reagent-particle contact	U P
	Hydrolysis		1.3	Reaction with water: ionisation with water: ionisation, hydrolysis, polymerisation Formation of metal-hydroxo complexes with Al and Fe salts	U P
2 **Particle destabilisation** (mainly electrochemical but some motion)	Coagulation	Coagulation	2.1	Compression of the electrical double layer by non-hydrolysing counter-ions	U P
			2.2	Reduction of surface potential through chemisorption hydrolysing metal ions or surface active substance chemical reactions	U P
		Micro-flocculation	2.3	Enmeshment in a precipitate (sweep flocculation)	U O
			2.4	Interparticle bridging through specific adsorption of coagulant or flocculant of polymeric species. Mutual aggregation	U O
3 **Particle transport** (collision)	Flocculation	Perikinetic	3.1	Brownian motion (thermal diffusion). For particles of size - 1 µm	U P
		Orthokinetic	3.2	Fluid (velocity gradients G)	U O
			3.3	Differential motion of particles; e.g. settling, flotation	
4 **Phase separation**			4.1	Sedimentation, clarification flotation, filtration, thickening, sludge disposal	U O

of water purification are shown in Table 6.1. The terminology used in the table is reflected in the glossary to this text. Chemical dosing, mixing and the phase separation unit operations are dealt with more fully in subsequent chapters.

A number of factors are involved in the coagulation of particles in natural water. The properties of colloids, their stability, the kinetics of coagulation and the physico-chemical factors involved are of relevance. Coagulants are considered in terms of coagulation and the utility and availability of various compounds used for the purpose.

6.1.1 PROPERTIES OF COLLOIDS

Hydrophilic sols are formed by starches and proteins; the result resembles a true solution. Their molecules are, however, large enough to be of colloidal dimensions and exhibit colloidal properties. Hydrophobic sols generally have a much larger size than hydrophilic sols. The distinction between the two types of colloid is not definite - a series of substances exists with properties intermediate between the two extremes.

Clay minerals are an example of an intermediate type; they are readily coagulated by an electrolyte, but also result in stable sols on contact with water.

Bacteria also behave like colloids; although some are larger than the arbitrary size range usually accepted for colloids. They are subject to Brownian motion, exhibit light scattering to some degree, but do not result in granules when coagulated. They can be classified as hydrophilic colloids. Viruses are also negatively charged and highly adsorptive; they can therefore also be coagulated.

Size is one of the most important parameters of colloids. Molecular dispersions have particles smaller than 10^{-9} m; the size range of colloidal particles is 10^{-9} to 10^{-6} m. The smaller the particles of a given mass are, the greater the surface area for a given mass.

The surface effects of colloids determine their degree of stability in dispersion; stability is due to the mutual repulsion between particles. Repulsion is indicative of the like charge. Most colloids have an electro-negative charge; the magnitude of the charge can be measured electrophoretically (Van Duuren, 1967).

Particle interaction due to electrical charge can clearly be effective only if the particle size is below a certain level; above this level the force of gravity becomes dominant. The mobility of particles can be measured in terms of their zeta potential. This gave rise to the concept that coagulation can be assessed in terms of zeta potential. This is unfortunately not the case, to some extent because measurement is subjective. Streaming current can be used as a laboratory method to assess coagulation; it is measured by means of a synchronous motor and an ammeter and is therefore not subjective (Bratby, 1980).

The most common colloids to be coagulated are clay particles and bacteria; these are about 1 micrometer in diameter. Viruses and colour colloids are one hundredth that size and algae are ten times larger. Algae vary over a wide range of sizes, shapes and coagulation properties.

An image of colloidal particle size can be obtained from their number in a given volume. The number of clay particles in a 1 mg/ℓ suspension was estimated to be 10^5 to 10^6 per mℓ (Van Duuren, 1967; Tambo, 1990). One mℓ of polluted water can contain some 30 million bacteria.

A variety of microorganisms and particles which are found in water is shown in Figure 6.1; the schistosoma is not of colloidal size and is unaffected by coagulation.

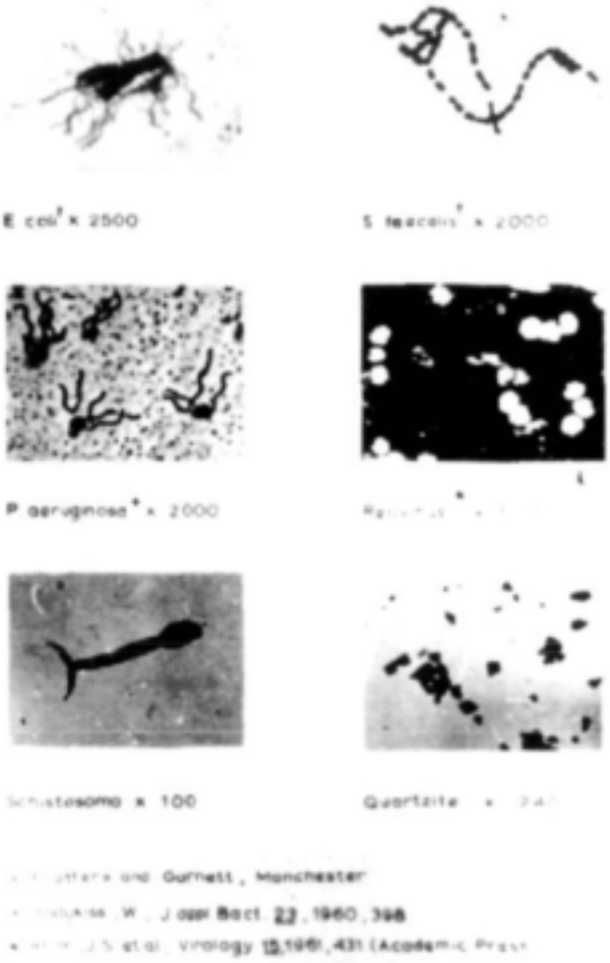


Figure 6.1
Examples of microoganisms and particles in raw water

6.1.2 Stability of colloids

Two broad theories exist to explain the basic mechanism of colloid stability and coagulation.

The physical theory emphasises the concept of an electrical double layer which surrounds a colloid and also the significance of mainly physical factors such as ionisation, ion adsorption and zeta potential.

The chemical theory assumes that colloids are aggregates of defined chemical structure, that the primary charge of colloid particles arises from complex ionogenic groups present on the surface of the dispersed particles, and that the destabilisation of colloids is due to chemical interactions such as complex formation.

The two theories are both of importance. The physical theory primarily deals with the particulate matter which is to be removed. The chemical theory also deals with the water medium in which the particles are suspended and the coagulants used. The physical theory is dealt with further in the kinetics of coagulation. A special aspect of the chemical theory is the interaction of electrolyte concentration and the zeta potential which in turn is related to the thickness of a double layer.

An increase in electrolyte concentration decreases the thickness of the double (electrical) layer thus allowing the particles to come closer together, and allows forces of attraction to come into operation. This means that the addition of a coagulant releases ions or electrolytes and thus promotes coagulation.

The effect of ions in solution on the zeta potential of a particle is illustrated in Figure 6.2 (Glasstone, 1948).

6.1.3 Kinetics of coagulation

The rate of coagulation depends on the collision frequency of the sol particles and their energy with respect to an energy barrier. Colloidal particles undergo Brownian motion and thereby diffuse and collide; they are in kinetic-thermal equilibrium with the suspending fluid. In the absence of an energy barrier, the rapid coagulation rate will be determined solely by the collision frequency.

Von Smoluchowski (1916) arrived, by dimensional analysis, at equations of the kinetics of coagulation. He found that the equation for perikinetic collision frequency is:

$$p = 4 \pi D R n_o \text{ .... } 6.1$$

where:

D = diffusion constant

R = radius of the attraction sphere

n_o = initial number of primary, spherical particles per unit volume

He also found that the orthokinetic collision frequency (f) may be represented by:

$$f = 4/3 R^3 n_o G \text{ .... } 6.2$$

where G is the velocity gradient.

Rapid coagulation and the flocculation of particles are illustrated in Figure 6.3; Von Smoluchowski developed the figure from experiments and mathematical considerations. In this figure n_o is the initial number of particles; n_1 is the number of particles after time t; T is the time required for the number of particles to be halved.

A comparison of coagulation and flocculation collision frequencies demonstrates the relative significance of these in terms of particle size; from Equations 6.1 and 6.2 :

$$\frac{\text{Flocculation collision frequency}}{\text{Coagulation collision frequency}} = \frac{f}{p}$$

$$= \frac{4 R^3 . n^0 . g}{4 \pi . D . R . n^0} \text{ .... } 6.3$$

From the Einstein equation for the diffusion coefficient D for molecules in liquid:

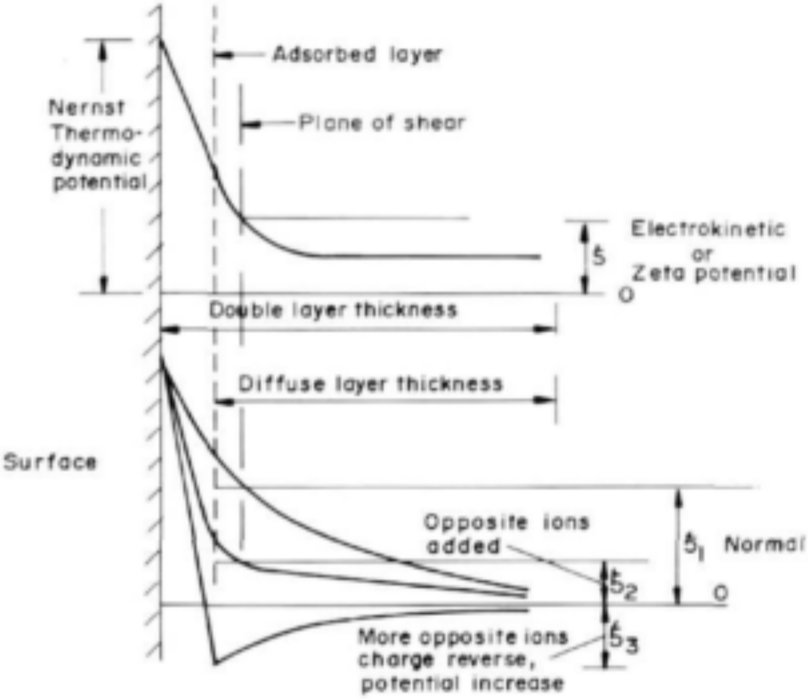


Figure 6.2

Effect of ions in solution on zeta potential

$$D . R = \frac{2 . K_B \text{ (temperature)}}{3 . \pi . \mu} \text{ .... } 6.4$$

where:

K_B(temperature) = Boltzman constant
$= 1,38.10^{-17}$ k cal/m^2s K
and μ = absolute viscosity
$= 1,01. 10^{-3}$ g mass/m s at 20°C.

Hence:

$$\frac{f}{p} = \frac{1,01 . 10^{-3} R^3 G}{2 . 293 . 1,38 . 10^{-17}}$$

i.e.

$$f/p = 1,25 . 10^{11} R^3 . G \text{ .... } 6.5$$

When perikinetic and orthokinetic flocculation proceed at the same rate, f = p and for $G = 10 s^{-1}$, R is nearly 1 micron.

When R is 10 micron, f/p = 1 250; even for this very small floc particle, orthokinetic flocculation proceeds at a much faster rate than coagulation. It is therefore clear that

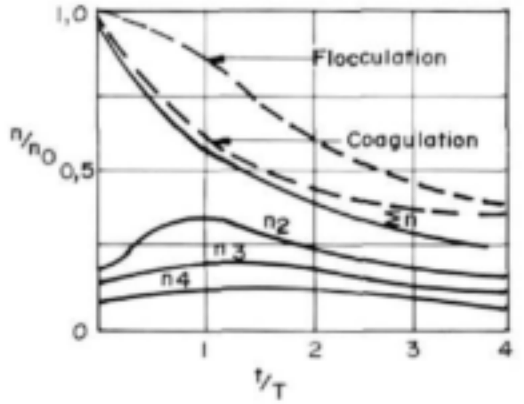


Figure 6.3
Coagulation and flocculation of particles

destabilisation is an essential precursor to aggregation but that orthokinetic flocculation is essential to increasing floc size in preparation for phase separation. Unless this distinction is understood and applied in practice, phase separation will be adversely affected or even impossible.

6.1.4 PHYSICO-CHEMICAL FACTORS

Theriault, Clark and Miller established the basis of our understanding of the chemical factors involved in coagulation for waters containing turbidity or colour colloids (Packham, 1962). They found that a certain amount of cation should be present for effective coagulation. They also found that an anion of strong coagulating power should be present. Lastly they found that pH must be carefully adjusted.

The Shulze-Hardy rule of coagulating power and valency indicated that the salts of the trivalent Al and Fe were the most effective mineral coagulants. In terms of this rule the coagulating power of cations are in the ratio of 1:10:1000 as the valence of the ions increases from 1:2:3. According to this rule, sodium is a very weak coagulant.

The hydrolysis products of aluminium and ferric salts are more effective than the corresponding trivalent cations in reducing or neutralising the zeta potential of colloids. Certain anions such as SO_4 have a more marked effect on coagulation than that of cations and the effect increases with ionic charge.

The significance of pH for coagulation with most coagulants has been extensively confirmed. The most effective coagulation of clay colloids with aluminium sulphate occurs over the pH range 6,8 to 7,8 (Packham, 1960); typical pH is 7,0 (AWWA). The ferric coagulation pH range is 5,0 to 8,5; typically 7,5 (AWWA). It should be noted that alum salts are amphoteric, i.e. soluble in both low and high pH environments.

Effective coagulation of colour colloids occurs at pH levels 5,5 to 6,8 (Stumm and Morgan, 1962). Initial alkalinity also plays a significant role for colour colloids; it affects the pH range of favourable removal.

The cation exchange capacity of the primary particles plays an insignificant part in coagulation. This property may well be masked by adsorption of coagulant ions onto the surface of these particles. This also applies to the superficial properties of particles such as structure and layering (Van Duuren, 1967).

The same dosage of coagulant (alum) does not equally effectively remove different particles; this is illustrated in Figure 6.4.

It will be seen from Figure 6.4 that the removal of particles from a suspension by coagulation with alum differs considerably for the same dosages. The results were obtained by experimentation under controlled chemical conditions as regards solute and pH.

6.1.5 FORMING MICROFLOCS

The objective of obtaining an understanding of coagulation from theory is to enable effective microfloc to be formed. By definition, coagulation is understood to mean the destabilisation and initial aggregation of colloids in suspension. An attempt has been made (Benefield *et al.*, 1982) to summarise the wide-ranging coagulation theory into a unified, practicable concept. According to these authors, the four discernible mechanisms which result in the forming of flocs from colloids are:

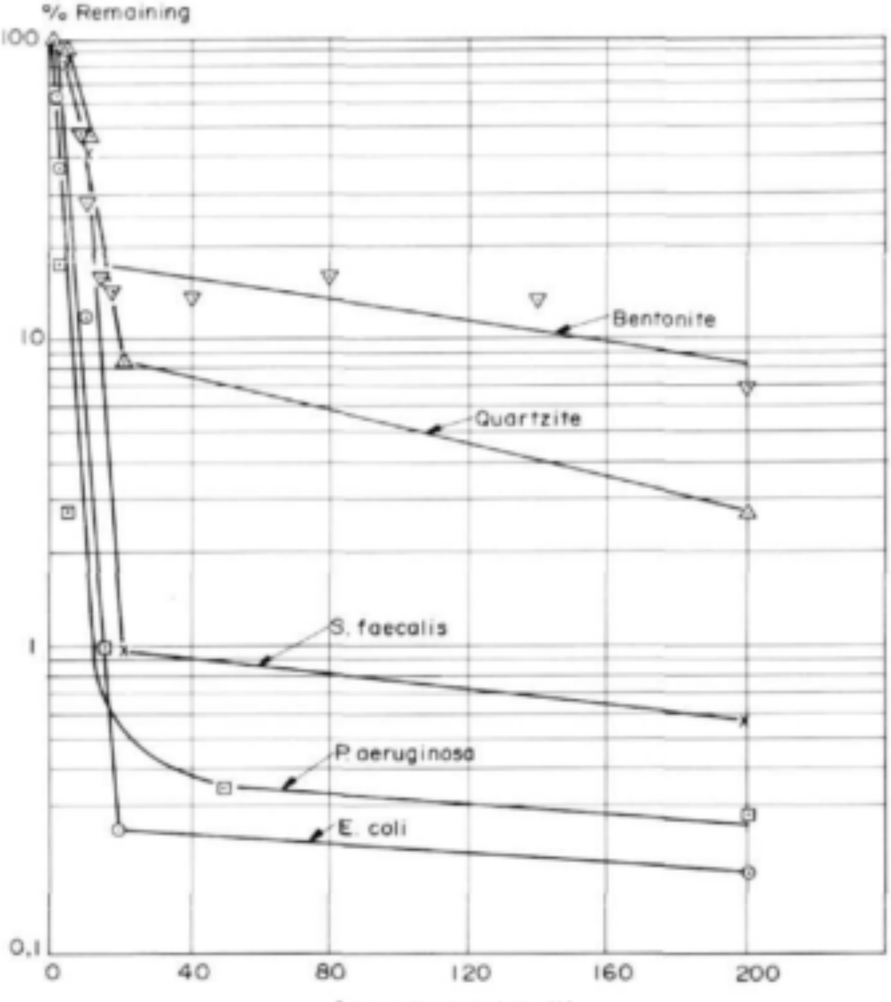


Figure 6.4
Removal of particles by coagulation and flocculation

- Reduction of surface potential by double-layer compression, i.e. the Schulze-Hardy rule. The coagulating power of a salt is determined by the valency of one of its ions.
- Reduction of surface potential by adsorption and charge neutralisation. This differs from double-layer compression in three ways. Firstly sorbable species are capable of destabilising colloids at much lower dosages than non-sorbable ions. Secondly the destabilisation by adsorption is stoichiometric. Thirdly it is possible to overdose a system and cause restabilisation as a result of reversal of charge on the colloids.
- Destabilisation of colloids by enmeshment in a precipitate. Some metal salts rapidly form precipitates in water; colloids are nuclei for these or may become enmeshed as the precipitate settles. This is termed sweep floc coagulation; it does not depend on neutralisation of surface charge.
- Destabilisation by adsorption and interparticle bridging. This is achieved by means of materials with large molecular size and multiple electrical charges along

a chain of carbon atoms. Excessive polymer, or intense or prolonged agitation may destroy bridges and result in restabilisation.

The characteristics or composition of the microflocs formed, are determined by the nature and concentration of particles to be coagulated, the kind and concentration of metal or organic polymer coagulants used, the ion concentration, pH and temperature of the water and the duration and intensity of agitation.

6.2 COAGULANTS AND COAGULANT AIDS

A very large range of coagulants is available. These can be categorised as mineral and organic. Even the mineral coagulants are now available in modified form e.g. poly-aluminium chloride. As regards anionics and cationics the health aspect may well play a part in the selection or acceptability of a coagulant.

6.2.1 COAGULANTS

Metal coagulants, such as aluminium and ferric salts, have been used for many years in the coagulation of colloid suspensions. Since the beginning of the 1960s, structures and behaviours of hydrolysed products of these salts, have been determined extensively (Tambo, 1990).

6.2.2 COAGULANT AIDS

Coagulant or flocculant aids are used to strengthen floc structure and to make large and well settleable floc. Many non-ionic or weakly anionic polymers are used for this purpose. The first of this kind of flocculant is a silica sol used at the Chicago filtration works (Baylis, 1937). Now known as activated silica, it has a molecular weight of 10^4 and is negatively charged. This flocculant aid has been used for many years by the Rand Water Board.

A more recent floc strengthener is non-ionic or weakly anionic polymer such as polyacrylamides or polyacrylic acid. These have molecular weights of about 10^6; they result in very tough and even pellet flocs.

An inorganic, cationic coagulant, which is based upon polymerised silica sol of molecular weight of 10^6 and accompanying metal ions in the sol, has also been developed. This innovative cationic polymer can be used for both colour and clay colloid coagulation over a wide concentration range.

6.2.3 APPLICATION

In the case of coagulation of colour colloids, competitive reaction of aluminium humate production and hydrolysis of aluminium during the initial stage of the coagulation operation can occur (Snodgrass *et al.*, 1984).

When humic substances and clay colloids co-exist, coagulation proceeds effectively in the weakly acidic region, which is similar to the coagulation of coloured water. In this case the aluminium-humic complex formation proceeds first and the resulting products neutralise the charge and serve as bridging precipitates for the clay particles.

For clay suspensions of a relatively higher concentration cationic polymers of molecular weight of 10^5 effectively act as the charge neutraliser and bridging agent at the same time. The number concentration of this size of polymer in a 1 mg/ℓ polymer solution is in the order of 10^{13}. The number is relatively smaller than that of colour colloids to be coagulated. Cationic polymers are therefore not used effectively for colour particle coagulation (Tambo, 1990).

Weakly charged anionic polymers or nonionic polymers, with molecular weights of 10^6 to 10^7, are used to strengthen clay or colour flocs resulting from metal coagulants. The number of polymer particles with a molecular weight of about 10^7 is of the order 10^{10}/mℓ. The dosage needed, is controlled by the number of microflocs to be bridged (Tambo,1990).

6.2.4 DOSAGES

The determination of dosage of coagulant and of chemicals to aid coagulation is still best done by means of the jar test. This is also referred to in Chapter 21, which deals with the operation and control of water works.

The prime objective in determining the dosage of coagulant and chemicals is effectiveness and economy. Effectiveness applies to colour and turbidity which has to be removed or diminished and economy applies to the minimum and yet effective dosage. However, there are also optimum and operational dosages (Polasek, 1992). All of these can be defined as follows:

- The effective dosage is that at which the greatest quantity of impurities per unit mass of coagulant is aggregated.
- The optimum dosage of coagulant is that at which the best water quality is produced, i.ę. the lowest colour and turbidity is attained.
- The economic dosage is the lowest dosage at which water of an acceptable quality is produced.
- The operational dosage is the lowest dosage at which the quality of the purified water complies with purified water quality criteria.

Calculations based on theory can also be used. These are, however, less reliable than jar tests (Polasek, 1992) and still require local verification. The parameters on which the various formulae are based include colour, turbidity, alkalinity, hardness, sulphates, chlorides and oxidisability. Corrosivity results from chlorides and sulphates; this is dealt with in Chapter 13.

6.3 THE JAR TEST

Jar tests do provide a readily reproducible set of parameters repeatedly (Packham, 1962). The jar test device has been evaluated in terms of a defined velocity gradient device (Van Duuren, 1967) and it was concluded that it can be used as a laboratory flocculator. Definitive coagulation tests can be performed in water works laboratories with a jar test device (Van Duuren, 1967; Bratby, 1980). It should be noted that jar tests are performed on a batch basis whereas a continuous flow pertains in most water works.

A guide is clearly needed to the selection of coagulants and dosages to be studied. Water has been classified, for coagulation by jar test procedures, into four categories, based on turbidity and alkalinity concentrations (Benefield *et al.*, 1982). This is shown in Table 6.2.

A jar test machine is shown in Figure 6.5 (similar to Phipps & Bird). The device is fitted with a revolution counter, a simultaneous dosing attachment and six flat paddles, driven by a single motor. Rectangular paddles, 7,5 x 2,5 cm, are generally used. Tall-form (9 cm diameter) one-litre jars with depth markings should be used to contain the water sample to be tested.

For the purpose of coagulation tests the colour, turbidity, pH and alkalinity of the raw water sample (50 ℓ) should first be determined. The sequential procedure is then as follows:

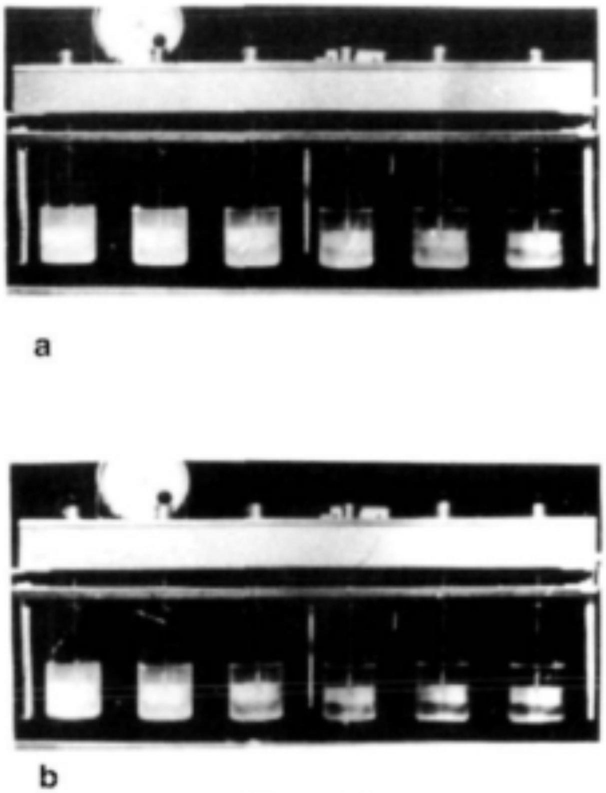


Figure 6.5
Jar test machine

- Measure 500 mℓ portions of the sample into each of the six tall jars and place in the machine.
- Start the motor.
- Dose with the coagulant to be tested at varying doses, preceded by pH (alkalinity) adjustment if necessary. For convenience, the coagulant stock solution can be prepared so that 1 mℓ added to 500 mℓ water is equivalent to 10 mg/ℓ.
- Agitate at 100 r/min for 2 minutes (rapid mix) and then at 40 r/min (flocculation) for a further 8 minutes.
- While flocculation is taking place, observe the first appearance of visible floc in each of the samples; also note the appearance, size and quantity of the floc at the end of stirring.
- Observe and note the settling rate of the suspension if possible.
- Let all the portions settle in the jars for 15 to 30 minutes. Observe the extent of clarification and the character of the floc remaining in the suspension. Observe the amount of sediment at the end of the period of settling.
- Carefully decant or withdraw a sample of clarified supernatant from each jar by means of a pipette.
- Determine the colour, turbidity, pH and alkalinity of the samples.
- If in a series the best final colour is higher than 20 and the turbidity higher than 10 units, repeat the procedure.

6.3.1 TYPICAL EXAMPLES

A number of typical examples of jar tests on a variety of waters and with various coagulants are given in Tables 6.3.1 to 6.3.3.

It is evident from the jar test results that the most effective coagulant and relevant dosage can be assessed in the laboratory. The results may not be directly applicable on a water works to the extent especially that water quality is changeable and can change over short periods of time. However, the more test results are obtained over an extensive period of time, the greater the degree of confidence attainable in works process design and subsequent operation. It may then well be possible to determine the coagulant and dosage to be used from works records and water quality at any given time.

6.4 CHEMICALS

It is essential to know in the first instance what the process uses of chemicals are in water purification and treatment. A tabulation

Table 6.2
Classification of Waters and Dosages of Coagulants for Jar Tests

Type of water	Aluminium sulphate	Coagulant ferric salts	Polymers
Type 1: High turbidity, high alkalinity (easiest to coagulate)	Effective for pH 5 to 7. Addition of alkalinity and coagulant aid not required.	Effective for pH 6 to 7. Addition of alkalinity and coagulant aid not required.	Cationic polymers very effective. Anionic and non-ionic may be effective. High molecular weight materials best.
Type 2: High turbidity, low alkalinity	Effective for pH 5 to 7. May need to add alkalinity if pH drops during test.	Effective for pH 6 to 7. May need to add alkalinity if pH drops during test.	Cationic polymers very effective. Anionic and non-ionic may be effective. High molecular weight materials best.
Type 3: Low turbidity, high alkalinity	Effective in relatively large dosages, which promote precipitation of $Al(OH)_3$. Coagulant aid may be needed to weight floc and improve settling.	Effective in relatively large dosages, which promote precipitation of $Fe(OH)_3$. Add coagulant aid to weight floc and improve settling.	Cannot work alone due to low turbidity. Add particulates (e.g. bentonite) ahead of polymer.
Type 4: Low turbidity, low alkalinity	Effective only for sweep floc formation, but resulting dosage will neutralise alkalinity. Add alkalinity to produce type 3 of particulates to produce type 2 water.	Same as alum.	Will not work alone due to low turbidity. Add particulates (e.g. bentonite) ahead of polymer.
High turbidity > 100 units High alkalinity > 250 mg/ℓ as $CaCO_3$		Low turbidity < 10 units Low alkalinity < 50 mg/ℓ as $CaCO_3$	

Table 6.3.1
Turbid, Low-Colour, High-Alkalinity Water

Jar	Alum dosage mg/ℓ	Floc resulting	Settled water			
			Colour Pt Co	Turbidity units	pH	Alkalinity mg/ℓ
Raw water	0	-	10	250	7,6	125
1	10	none	10	200	7,4	120
2	20	poor	8	155	7,1	115
3	30	good	5	50	6,8	110
4	40	very good	5	8	6,6	105
5	50	good	5	10	6,4	100
6	60	poor	7	30	6,2	95

Note that the optimum dosage is some 40 mg/ℓ and the pH 6,6. The test was repeated for dosages between 35 and 50 mg/ℓ.

TABLE 6.3.2
COLOURED, LOW-TURBIDITY AND LOW-ALKALINITY WATER

Jar	Alum/lime dosage mg/ℓ	Floc resulting	Settled water			
			Colour Pt Co	Turbidity units	pH	Alkalinity mg/ℓ
Raw water	0 0	-	80	10	5,6	15
1	20 0	poor	50	10	4,6	6
2	20 3	fair	30	8	5,0	10
3	20 6	good	10	5	5,4	13
4	30 8	good	10	5	5,4	15
5	30 10	fair	25	8	5,8	15
6	30 12	poor	70	1010	6,2	18

Note that the best results were obtained in jar 3 at dosage 20 mg/ℓ alum, lime 6 mg/ℓ and pH 5,4. The test was repeated for alum at dosages of 20 to 30 mg/ℓ and for lime at 4 to 9 mg/ℓ.

TABLE 6.3.3
LOW-TURBIDITY, LOW-COLOUR, MEDIUM-ALKALINITY WATER

Jar	Poly/alum chloride mg/ℓ	Floc resulting	Settled water			
			Colour Pt Co	Turbidity units	pH	Alkalinity mg/ℓ
Raw water	0	-	10	9	7,6	55
1	5	poor	10	5,6	7,5	50
2	10	poor	8	2,7	7,3	45
3	15	fair	7	1,8	7,1	39
4	20	good	5	1,8	6,8	35
5	25	fair	7	1,3	6,5	31
6	30	poor	10	1,6	6,2	28

Note that for poly aluminium chloride, jar 4 had the best results, with a coagulant dosage of 20 resulting in a colour of 5 units, turbidity 1.8 and pH 6,8. Further tests were carried out for dosages between 15 and 25 mg/ℓ.

is therefore given of process use. It is then necessary to know the properties and characteristics of the chemicals used. Availability, procurement, storage, handling, safe use and dosing of these chemicals are of particular interest to designers, supervisors and operators of water works.

There are some compounds which are new to potable water purification and treatment. These include polyelectrolyte organic coagulant aids, complexed compounds such as poly aluminium chloride and chlorine dioxide. These and also fluoride compounds have to be cleared for use. Some works have

TABLE 6.4
PROCESS CHEMICALS

Process	Chemicals
Coagulation	Aluminium sulphate Ferric chloride Ferric sulphate Ferrous sulphate Sodium aluminate
Flocculation and sedimentation aid	Activated silica Bentonite Polyelectrolytes
Defluoridation	Activated alumina Aluminium sulphate
Disinfection	Anhydrous ammonia Calcium hypochlorite Chlorinated lime Chlorine Chlorine dioxide Ozone Sodium hypochlorite
Dechlorination	Activated carbon Sulphur dioxide
Fluoridation	Sodium fluoride
Oxidation, aeration	Chlorine, ozone Hydrogen peroxide Potassium permanganate
pH adjustment	Calcium hydroxide Carbon dioxide Hydrochloric acid Sodium carbonate Sodium hydroxide
Softening	Calcium hydroxide Ion-exchange resins Hydrochloric acid Sodium carbonate Sodium chloride Sulphuric acid Zeolite
Stabilisation and corrosion control	Calcium hydroxide Sodium hexa metaphosphate Sodium hydroxide Sodium silicate
Taste and odour control	Activated carbon Chlorine and ozone Hydrogen peroxide Potassium permanganate

been equipped to dose activated carbon in case of need; this can readily be done with the same equipment as is used for lime dosing. Sodium hydroxide and sodium carbonate is now more commonly used for pH adjustment. Chlorine dioxide has been considered in some works and potassium permanganate has been used for oxidation purposes particularly when iron and manganese are present.

The physical aspects of the chemicals, including handling are dealt with in this chapter; the chemical and biological aspects are dealt with more specifically in the other sections of this chapter, and in Chapter 14 which deals with disinfection.

6.4.1 PROCESS USE OF CHEMICALS

The process use of chemicals can be categorised as shown in Table 6.4.

6.4.2 PROPERTIES AND CHARACTERISTICS

The properties and characteristics of the chemicals used in water purification and treatment are categorised in Table 6.5. The common, generic names of these chemicals are also given. Shipping containers or packaging, handling materials, appearance, available form, solubility, solution dosing strength, commercial strength and bulk density are also given.

6.4.3 INVENTORIES AND DELIVERY

An inventory of the chemicals on a works is necessary as it involves storage, type of container, period of delivery, delays, payment and possible emergencies. A basic decision

TABLE 6.5
PROPERTIES AND CHARACTERISTICS OF WATER WORKS CHEMICALS

Chemical name and formula (trade name)	**Packaging/ suitable handling materials**	**Appearance/ available form**	**Solubility/ solution dosing strength (kg/ℓ)**	**Commercial strength/ bulk density (%, kg/ℓ)**	**Remarks**
Activated alumina Al_2O_3	bags / iron, lead, steel	white/ granules	insoluble	100%	SABS
Activated carbon C	bags/ rubber silicon	black/powder, granules	insoluble		SABS
Aluminium sulphate $Al_2(SO_4)_3$ $.18\ H_2O$ (alum)	bags (50, 100 kg) / silicon, asphalt	yellow-white/ lumps, granules, powder, liquid	0,42/10%	15-22% dry; 8% solution 0,94	pH of 1% solution 3,4 SABS
Ammonia NH_3 NH_4OH (ammonia water)	cylinders/ glass, iron nickel, steel	colourless/ gas, liquid	complete dilute as needed	25% as NH_3 0,52	SABS Cl_2 stabiliser
Bentonite Al_2O_3 $4SiO_2nH_2O$ (Clariflo)	bags/iron steel	white/ powder	insoluble	0,96-1,04	floc weighting particles
Calcium hydroxide $Ca(OH)_2$ (slaked lime)	bags/ bulk asphalt, cement, iron, rubber	white/ powder light, dense	0,014	85-99% $Ca(OH)_2$ 70% CaO 0,6-0,7	slurry or hopper agitation
Calcium hypochlorite $Ca(OCl)_2$ (HTH)	cans, drums (10-100 kg) glass, rubber, wood	white/ granule powder tablet	18% @ 25 °C up to 10%	70% Cl_2 0,8	SABS 295
Carbon dioxide CO_2	cylinder 78 kg	colourless/ gas	complete	0,173	corrosive SABS
Chlorine Cl_2 (chlorine)	cylinder (68, 900 925 kg) dry: iron copper wet: silver glass, rubber PVC, polythene	green-yellow/ gas	0,007 15°C	99,5% + Cl_2	poisonous oxidant SABS

Chemical name and formula (trade name)	**Packaging/ suitable handling materials**	**Appearance/ available form**	**Solubility/ solution dosing strength (kg/ℓ)**	**Commercial strength/ bulk density (%, kg/ℓ)**	**Remarks**
Chlorine dioxide CO_2	produced	yellow-green/ gas	0.025 1-2,5%	0,8	very unstable
Ferric chloride $FeCl_3$ $FeCl_3.6H_2O$	drums fibreglass rubber	liquid	complete		very corrosive SABS 0157
Ferrous sulphate $FeSO_4$ $7H_2O$ (copperas)	bags, bulk asphalt wood	light green/ crystal	0,384	1,13	optimum pH 8,5-11,0 SABS
Ozone O_3	aluminium glass	bluish/ gas	3 mg/ℓ 20 mg/ℓ in air		Produced *in situ* SABS
Potassium permanganate $KMnO_4$	drums iron, steel wood	purple crystal	0,063 up to 5%	100% 1,38	Explosive SABS
Sodium aluminate $NaAlO_2$ Al_2O_3 (soda alum)	bags iron, steel plastic	brown/ powder liquid rubber	30% 0,168	70-80%	hopper agitation feeder SABS
Sodium fluoride NaF (fluoride)	bags iron, steel	Nile blue/ powder	4% 4%	90-95% NaF	pH 6,6 4% solution SABS
Sodium carbonate Na_2CO_3 (soda ash)	bags iron powder steel rubber	white/ crystalline	0,30-0,33 up to 5%	99,4%	hopper agitation feeder SABS
Sodium hexa meta phosphate $Na(PO_3)_6$ (Calgon)	bags steel rubber plastic	white/ powder	0,2 - 0,9 up to 5%	66% P_2O_5 1,1 to 1,5	pH 6,0-8,3 for 0,25% solution
Sodium hydroxide NaOH (caustic soda)	drums cast iron rubber	white/ flakes	0,24 to 0,48	98,9% NaOH 1,09	pH of 1% solution 12,9 SABS

Chemical name and formula (trade name)	Packaging/ suitable handling materials	Appearance/ available form	Solubility/ solution dosing strength (kg/ℓ)	Commercial strength/ bulk density (%, kg/ℓ)	Remarks
Sodium silicate Na_2SiO_3 (water glass)	drums cast iron rubber	opaque/ viscous liquid	complete up to 10%		pH of 1% solution 12,3 SABS
Sulphur dioxide SO_2	cylinders aluminium brass	gas	20% as required	99%	Irritating corrosive
Sulphuric acid H_2SO_4	drums cast iron	colourless viscous steel	complete as required liquid	93%	Dangerous activator Regenerant SABS

has to be taken as regards the number of days of supply desired. The quantum depends on a number of factors such as communications, availability of supplies, transport and remoteness. Excess inventory is undesirable for economical and physical reasons.

Chemicals such as calcium hypochlorite and potassium permanganate are not readily stored because of the danger of explosion. Other chemicals may be hygroscopic and cannot be stored for any length of time. The store life of a number of chemicals, especially in solution form, is limited. The type of material and its packaging also have an influence on storage and handling.

Some provision has to be made for vertical and horizontal movement of chemicals upon delivery to the works. A loading platform is usually required at the entrance to the chemicals store and to the chlorination room. Hoists which can receive bulk chemicals, usually run from the platform into the store room. Delivery trucks should ideally be equipped with pumps to convey liquids and even powdered chemicals over some distance and height.

Transportation is usually by road and trucks; rail delivery can be used for bulk supplies. The safety aspect of transportation and handling needs special care.

Unloading requires a hoist, an inclined conveyor belt or a lift. The unloading machinery should be designed for the maximum anticipated load. The safe unloading of chlorine requires a variety of mechanical devices, including a hoist, cradles and skids. Ropes, chains and even lifting magnets are unsafe; handtrucks are preferable. Proper lighting is essential for night operations.

More mechanical equipment is needed for solid chemicals than for liquid chemicals. Solids can be unloaded pneumatically, by screw conveyor, or by lifts. Bucket conveyors can be used when excessive dust is not a problem.

Liquid alum, acids, sodium hydroxide and corrosion inhibitors can readily be transported from containers by pumping. The supplier should be required to provide the necessary instruction and equipment for safe unloading and handling of liquid chemicals. Steps must be taken to ensure that there is sufficient capacity to receive liquid chemicals when delivery takes place. A high-level, audible alarm would assist in warning the operating staff and transporter that a tank has reached full capacity.

Dust control may be required when delivery is taken of dry chemicals. A bag-type filter may be best suited to removing dust over the loading areas. The chemical trapped in the filter bag can be recovered if special care is taken to use it for only one chemical at a time.

6.4.4 HANDLING

It is usually necessary to handle chemicals between the point of storage and dosing. Vertical transportation of chemicals from the store room to the dosing facility, is shown in Figure 6.6. Granular chemicals can be hoisted by means of a skip type of elevator as shown in this figure. An arrangement for a lime silo is shown in Figure 6.7. This equipment can obviate dust at the feeder, but the silo requires careful charging or a cover.

An example of wet storage of chemicals is shown in Figure 6.8. This configuration contains all the elements required to handle chemicals from the store to the point of the dosing equipment.

6.4.5 BULK STORAGE

Storage can be provided in- or out of doors; clearly however, chemicals should be protected adequately against wind and weather. Subsurface storage should be avoided. The type of bulk storage will depend on the chemical to be stored. Chemicals storage areas should be kept clean, well-ventilated and free from dust, moisture or vapours.

Compressed gases should preferably be stored under-roof, on suitable flooring such as concrete. Indoor storage areas should be designed in such a way that personnel can escape quickly in an emergency. Where gases are stored or used, at least two exits should be provided and kept clear at all times.

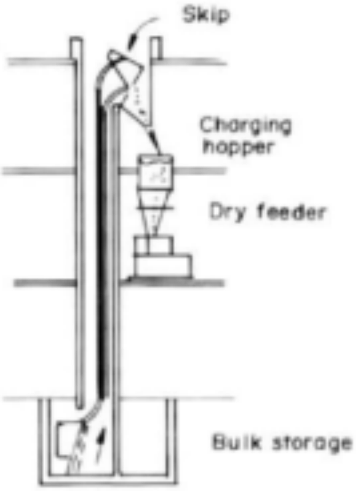


Figure 6.6
Chemicals elevator

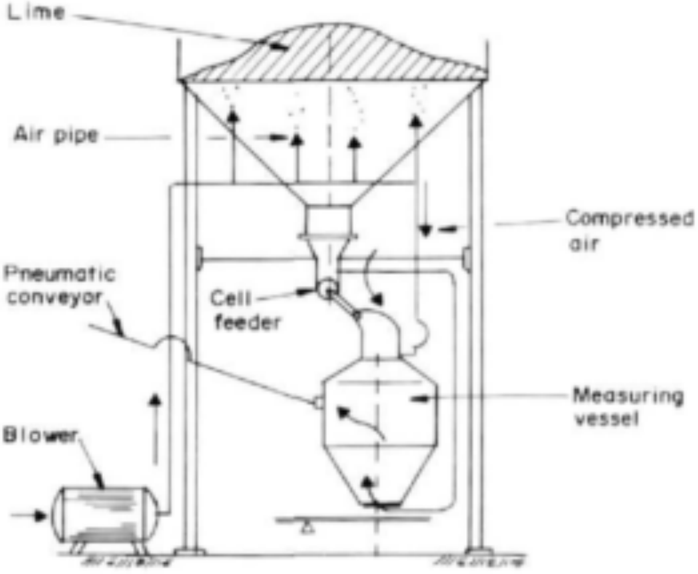


Figure 6.7
Lime silo

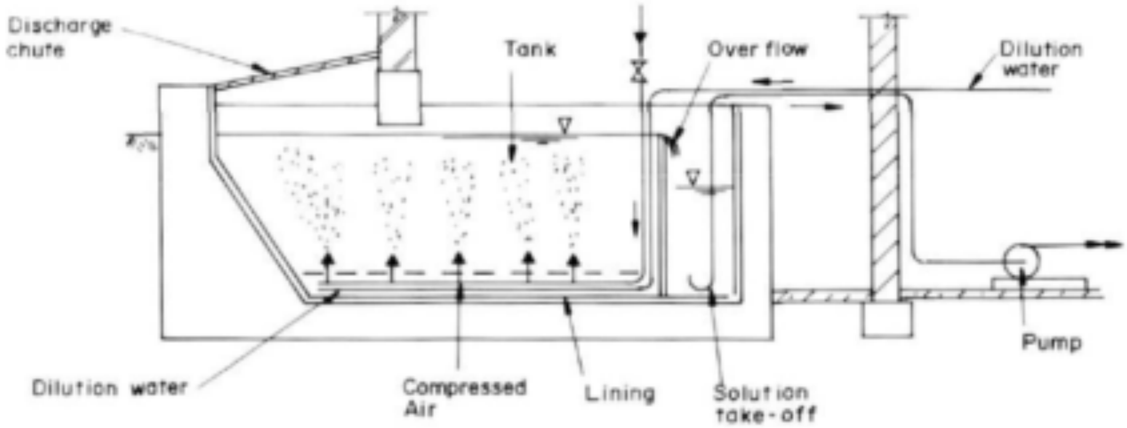


Figure 6.8
Wet storage of chemicals

Silos, bins or tanks can be used for bulk storage of chemicals. The number and size of storage facility should be consistent with the size of shipments received and the rate of use.

Storage tanks, for liquid chemicals, are made of suitable materials. As indicated in Table 6.5, mild steel, rubber, fibre glass, PVC and polythene are commonly used. The choice of material in contact with the chemical in storage will depend on chemical resistance, pressure and temperature. Flotation of (empty) tanks must be guarded against. Covered tanks require access manholes, lighting and ventilation.

The storage capacity of all chemicals must be related to the expected or experienced dosages.

6.4.6 PROTECTIVE MEASURES

From the array of chemicals available, it will be clear that protective measures are necessary for safety. The use of particular chemicals will determine the protective measures provided and taken (Maschellein, 1992). A written programme should be developed and reviewed with all personnel. Defensive plant operation and control will keep staff in a state of preparedness for an emergency.

Protective clothing and eye protection should be obtained and issued to personnel. Respirators and masks should be available, immediately outside dangerous areas such as a chlorinator room. Safety showers and eye baths should be provided where necessary.

Detailed attention should be given to ventilation of space such as in chemical and pump rooms.

Chemicals safety practice runs should be held periodically; this is dealt with more fully in Chapter 21.

6.5 WORKS EXAMPLE

An example of design considerations and procedure for chemical storage and dosing equipment is given here for a medium-sized water works.

A 10 Mℓ/d nominal capacity water works was designed complete with a river water intake. The river water has a turbidity of 100 NTU over extensive periods of time and as high as 1 500 NTU for up to five days; this water can also have high algal growth for periods of up to three months. Jar tests indicated that 60 mg/ℓ of aluminium sulphate (alum), or 20 mg/ℓ of $FeCl_3$ or 5 mg/ℓ of polyelectrolyte are satisfactory at different times, depending on turbidity or algal content.

The water required little or no pH correction, before or after coagulation, as it is well buffered. However, there is a possibility that pH will have to be corrected for short periods of time. It may also be necessary to apply activated carbon in the event of an industrial effluent spill upstream.

The works is fairly distant from the source of supply of coagulants and delivery time is up to three weeks from the date of an order being placed. It is therefore prudent to allow a storage period of 90 days and to ensure that orders are placed when there is still one month's supply in store.

Assuming a direct relationship between jar test results and full-scale, works operation for a 60 mg/ℓ dosage of alum, the following applies:

Mass of alum to be stored
= flow x dosage x days storage
= $10 \times 10^6 \times 60 \times 10^{-6} \times 90$
= 54 000 kg

At a bulk density of 750 kg/m^3 this is equivalent to 72 m^3.

Stored on pallets over 2 m high, a nominal floor area of 36 m^2 is required for the alum.

The $FeCl_3$ dosage is a third of that of the alum. However, it is delivered in liquid form in 200 ℓ drums and an allowance of 12 m^2 can be made for this purpose. Some 6 m^2 can be allowed for storage of the polyelectrolyte, 2 m^2 for activated carbon and 2 m^2 for pH adjustment and other chemicals.

A total of 58 m^2 nominal area is therefore needed. Passage ways, doors, lift and movement requires an additional 50% of area for a total of 87 m^2 floor area of the chemical store room.

The size of solution tanks can be based on sufficient supply for a day at a time. For the alum at 10% solution:

Volume of tank required
= Flow x dosage x period x 1/dilution x 1/bulk density
= $10 \times 10^6 \times 60 \times 10^{-6} \times 1/0{,}1 \times 1/750$
= 8 m^3

The volume of tank needed for $FeCl_3$, also at 10% solution strength, is 2,7 m^3.

For polyelectrolyte, at 5% solution strength, 1,3 m^3 capacity of containers is required.

7

Dosing and Mixing

F A van Duuren

It is necessary to add chemicals to water at different stages of purification or treatment, for various purposes. Dosing is the addition of chemicals to water whereas mixing implies blending or homogenisation of the chemicals and water.

Effective chemical dosing and mixing is important as it determines the efficiency of all the subsequent unit operations and therefore of water purification and treatment. The objective is to obtain a uniform reaction of the chemical with all of the suspension or water to be treated.

For coagulation and water treatment, the correct amount of chemical has to be measured off, injected into the main stream and blended rapidly with it. Uniform distribution of coagulant and other chemicals is essential because a relatively small chemical dose is proportioned to the entire flow. If this does not happen effectively, some or all of the suspension or water will be overdosed and some would pass on without being dosed; both are ineffective, uneconomical and even detrimental.

The chemicals may be added to water in dry or solution form. Dosing and blending involves both feeding and mixing of small volumes of solution, suspension or solids into large volumes of flowing water. Dry addition is used when flows are large and solution volumes would become too large. The ratio of dosage to flow is of the order 1 : 1 000 to 1: 100 000.

The theoretical aspects, the criteria for and types of chemical dosing and mixing equipment are outlined here. The theoretical aspects of mixing are of particular interest; a practical approach is, however, outlined. Chemical dosing methods and devices for dosing and mixing are also dealt with. Some methods, equipment, reaction and contact tanks are also dealt with in the chapters on disinfection and other treatment of water.

Ideal mixing pertains when there is an instantaneous, completely mixed flow. This is essential for coagulation and in the first stage of disinfection. Flow dispersion is also dealt with in Chapter 9.

7.1 THEORY OF MIXING

The problem of mixing two or more substances in disparate proportions, is not simple. Mixing is carried out to promote intimate contact between the liquids and solids involved in order to facilitate the most effective chemical reaction and physical effects.

As mixing is obtained mainly through turbulent mass flow, a potential gradient that will impart velocity to the mass being mixed is required. The flow of a velocity stream or jet within a mixing tank, entrains surrounding water through transfer of its initial total energy into kinetic energy. The velocity flow having thus entrained the surrounding water by momentum transfer and aided by the eddies formed is finally dissipated as heat of friction in the liquid.

The major aspects to be considered in the mixing of a solution or suspension and water is dispersion by diffusion, turbulence and energy requirements. Related aspects such as density, viscosity and time are also of importance.

7.1.1 DISPERSION BY DIFFUSION

Diffusion is the interchange of fluids between two neighbouring zones. It necessarily involves every local characteristic which the fluids or flow may possess.

If fluid is carried laterally from a low-velocity zone into a high-velocity zone and *vice versa*, the velocity and thus the momentum and kinetic energy of the low-velocity zone will be increased while that of the high-velocity zone will be decreased. The rate of increase or decrease evidently depends upon two variables, i.e. the carrying capacity of the cross-flow and the extent of the difference of the characteristics of the two zones.

Where N represents the rate of lateral transport of the fluid, the basic equation of diffusion is of the form:

$$N = - D \cdot dc/dy \quad \text{.......... } 7.1$$

where:

D = diffusion coefficient
c = characteristic intensity or concentration with respect to
y = distance

The negative sign indicates that transport of properties is in the direction of decreasing concentration.

The same dispersion could result from a low-velocity cross-flow which persists over a considerable lateral distance and from a high-velocity cross-flow which persists for only a short lateral distance. The coefficient D thus physically represents the combination of velocity and distance which together characterise the nature of the dispersion process.

The characteristic intensity or concentration may be measured in terms of the difference in density between an emerging jet of fluid and the surrounding fluid. The spread of a jet into a liquid is the physical limit of its effectiveness. A submerged jet, discharging vertically downwards, into a large body of liquid, makes an angle of 6° to 10° with the vertical (Hubbel,1962). When a jet discharges horizontally into a large reservoir of liquid, the angle of expansion is some 12° to 14° from the horizontal (Rouse, 1950).

7.1.2 TURBULENCE EFFECTS

Since mixing is performed by mass flow and turbulence, and in open basins mass flow is augmented by entrainment, it is clear that velocity alone is not the means of mixing. Turbulent flow in pipe lines or conduits involves instantaneous velocity fluctuations which through eddy formation, result in mixing. Rapid variations in velocity are of more importance than the actual magnitude of the nominal velocity since it is the rate of change of instantaneous velocity that creates momentum transport and eddy flow.

With adequate turbulence, the duration of mixing can be short, i.e. 1 to 5 seconds; this aspect is interrelated with power consumption. There is, however, some indication that the time required for mixing is a function of the flow. This may be seen from a consideration of the effect of having staged mixing. In a batch system all fluids and particles to be mixed, are in a tank for the same length of time. Since there are different resident times for the different fluids and particles in a continuous flow tank, the quality of the product from a continuous reactor is inferior to that from a batch reactor (Perry & Green, 1984).

The rate at which the energy of turbulence is dissipated is directly proportional to the rate of fluctuation and inversely proportional to the scale of the eddies. It should therefore be possible to control the effect of turbulence by proper variation of the eddy characteristics.

Eddies of small size govern the rate of dissipation whereas the largest practicable size is demanded for conditions requiring the longest eddy duration.

It would appear in general, that a mechanism causing numerous small eddies would be preferable to a single action which causes only a few large eddies. Fully turbulent flow is the ideal for dispersion.

7.1.3 ENERGY REQUIREMENTS

Rapid mixing can be attained only at the expense of higher power consumption. In order to attain the fully mixed state, higher power input is required for the necessary increased turbulence. Power consumption per unit volume increases rapidly as the size of the system increases.

Power requirements for fluid motion were derived from models and a theoretical formulation was developed. The formulation includes diameter D and speed N of the agitator. A plot of the power number and impeller Reynolds number, N_{Re}, yields a useful correlation of power data (Perry, 1984).

For power P and density ρ the power number is:

$$N_P = P . g / N^3 . D^5 . \rho \quad \text{.......... } 7.2$$

for viscosity μ the impeller Reynolds number is:

$$N_{Re} = N . D^2 . \rho / \mu \quad \text{.......... } 7.3$$

For a Reynolds number up to 300, power P is:

$$P = K' . \mu . N^2 . D^3 \quad \text{.......... } 7.4$$

Even for Reynolds number up to 300, inertia effects are clearly dominant and viscosity plays only a small role. Inertia plays an even more significant role at higher Reynolds number.

In the turbulent flow region ($N_{Re} > 10^5$), the power used is:

$$P = K . N^3 . D^5 \quad \text{.......... } 7.5$$

For a propeller, with three blades, K varies from 0.32 to 1.0; for a shrouded turbine K is 1.08 to 1.12 .

7.1.4 ENERGY DESIGN GUIDE

Velocity gradient is used as a design guide to the energy required for mixing. Instantaneous velocity gradient (G') is definable for laminar flow as the change in velocity across a liquid layer. However, because turbulent flow nearly always pertains, the mean velocity gradient (G) is used in practice.

In flocculation, G is the relative number of contacts of particles per unit volume per second (see Equations 8.4 and 8.5). Particle contacts are, however, not a factor in mixing; velocity gradient is therefore used arbitrarily (Montgomery, 1985), but because it is practical, it is useful. In their text, the American Water Works Association (AWWA, 1990), uses G as the energy term for mixing and for flocculation, but ignores the quadratic relationship; dimensional analysis shows this to be wrong.

For mixer design purposes, velocity gradient, G (s^{-1})is:

$$G = [P / \mu V]^{1/2} \quad \text{.......... } 7.6$$

where:

P = applied energy in W ($kg/m^2.s^3$)
μ = viscosity (1,52 to 1,15 x 10^{-3} kg/m.s @ 5 to 15°C)
V = volume (m^3)

The derivation of Equation 7.6 is outlined in the next chapter (8.1.4) and its use in flocculation is substantiated to that extent; it is used for calculations of mixing in water. The velocity gradient required for effective mixing, ranges from 600 to 1 000 s^{-1}.

7.1.5 VOLUME DESIGN GUIDE

Determination of the volume applicable to any particular mixer has to be based on judgement. Where mixing takes place in a pipe or conduit with a flow velocity > 0,5 m/s, V may be taken to be equivalent to 2 second of flow. For a baffled flash mixer as shown in Figure 7.10, the total volume of the basin should be used.

7.2 CHEMICAL FEEDING METHODS AND DEVICES

An early method of treating water with chemicals was by batch process. This consists of the addition of a weighed quantity of chemical to a known volume of water in a tank which is then stirred; the bulk of the solids is allowed to settle and the clarified water is finally filtered. Modern practice is to feed chemicals, whether soluble, insoluble, solid, liquid or gaseous to water by means of hydraulic or mechanical feeders.

Feeders may be divided into three types: dry, wet, and gas. Dry feeders dispense dry substances of either soluble or insoluble nature. Wet feeders are devised to feed solutions only or else either solutions or suspensions. Gas feeders feed soluble gases.

They may be divided further into constant rate types and proportioning types. Constant rate feeders are adjusted for any particular flow or dosage and this remains fixed until such time as it becomes necessary to readjust it. Proportioning types of chemical feeder dispense chemicals in proportion to varying flow rates of water. They have, however, to be adjusted for the concentration of chemical to be fed.

7.2.1 DRY FEEDERS

Feeding dry chemicals is more economical than feeding wet chemicals because of smaller masses and volumes. Savings are therefore effected in transportation and the sizes of plant and equipment; this is important in all works. Some chemicals, such as lime, soda ash and polymers come in dry form and should therefore be handled thus.

Dry, chemical feeders are especially suitable where space requirements would otherwise be excessive. They require about 1/10th to 1/20th the space necessary for a wet feeder. This is because a wet feeder applies chemicals at a concentration of 5% to 10% and hence, one m^3 of solution contains 40 kg to 80 kg of solids as opposed to dry densities of 400 to 800 kg/m^3.

Dry feeders are either of volumetric or gravimetric type. There is no great difference between the capacities of these two types, but the gravimetric type is generally more accurate and they is used where the amounts of chemicals fed are large.

In the case of the volumetric feeder, chemicals are fed via a hopper which may be equipped with a vibrator and thence to a belt, table or a screw. In any of these feeders a positive feed rate device should be incorporated. Their accuracy is of the order of 3% to 5%.

The gravimetric type of feeder contains a balance, and feed rate is adjustable to an accuracy of up to 1%.

It is desirable that all types of dry feeder be used in conjunction with an efficient type of solutioniser. A small vortex cone device can be used for this purpose.

7.2.2 LIQUID FEEDERS

There is a great variety of liquid feeders. They differ in degree of sophistication from the simple pot type to the complex orifice-water meter-contactor-feed control unit.

The pot type of feeder, shown in Figure 7.1, is limited to crystal or lump chemicals where the amounts fed are small, the accuracy required is not critical but low capital cost is important. An orifice plate may be used in the water line to create a pressure differential and water may then be shunted from upstream of the plate, via the bottom and through the pot to downstream of the plate. Flow control may be achieved by means of a valve ahead of the pot. It is possible, with the aid of somewhat more intricate pipe work and an additional orifice plate to devise an alum solution feeder based on the pot feeder principle.

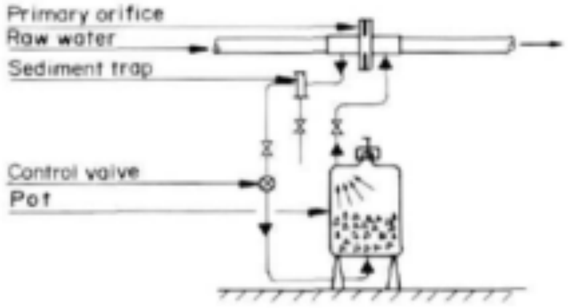


Figure 7.1
Pot type feeder

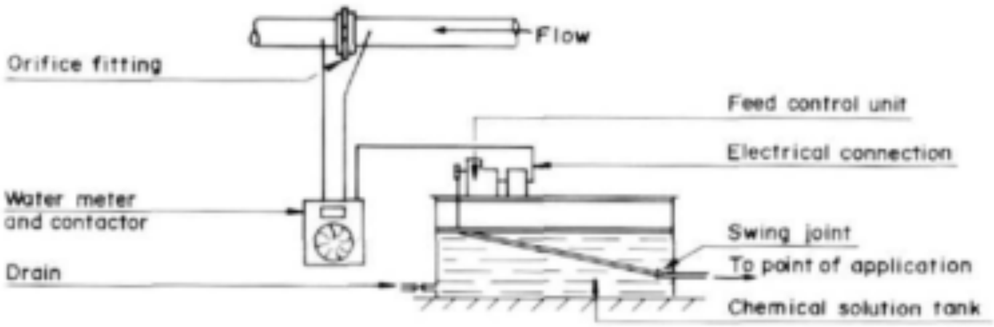


Figure 7.2
Electromechanical feeder for solutions

Reciprocating pump feeders are frequently used because of their positive displacement characteristic. The pump is usually provided with a manually operated stroke adjustment for setting the rate of flow. Automatic control may be operated through the stroke rate of the pump.

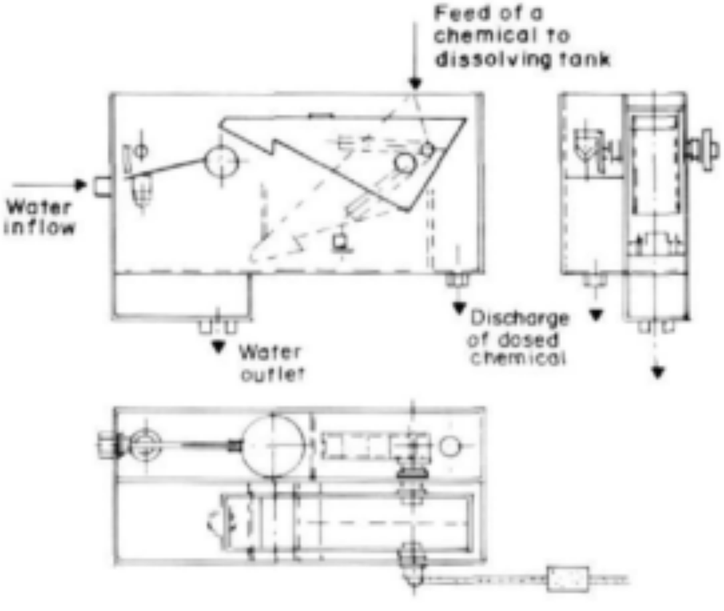


Figure 7.3
Tipping doser dosing alkaline solutions

A linkage of hydraulic, electrical and mechanical devices can be made in such a fashion that dosage of a given solution is applied at a rate determined by the flow of the water. Such a device is shown in Figure 7.2. The device consists of an orifice plate linked with a flow meter and contactor which actuates a feed control unit. This unit lowers a pipe inlet in the solution tank at a rate determined by the flow of water in the main pipe and a proportional chemical dosage is thus obtained.

An hydraulically operated device is shown in Figure 7.3. This tipping doser is driven by water on a counter balance principle. The flow rate into the tipping tank and the solution strength determine the dosage rate of this device.

The solution strength generally applied is less than 10%. The accuracy of wet feeders is determined by the flow measurement control which may have an accuracy of within 2%.

7.2.3 GAS FEEDERS

There are basically two types of gas feeder. These are direct feed as a gas or after it has been dissolved and is in solution form. The gas may be drawn from point of manufacture as in the case of carbon dioxide, which is derived from a stack or some type of carbon burner, otherwise it is drawn from a steel cylinder which is filled in the factory with liquefied gas. Means are provided, in both the direct and solution types of feeder, for controlling the feed rate either by varying the pressure differential across a fixed orifice or by varying the orifice size.

In the case of direct feed, the gas is applied from supply through some type of diffuser into the water. Flow measurement is by means of a manometer (pressure differential) or a rotameter (variable area) with provision for pressure compensation to ensure metering accuracy.

Solution feed equipment provides for dissolving the gas in a small volume of water under pressure, and for the application of the solution against pressure to the water to be treated. The gas/liquid solution is usually manufactured under vacuum throughout the metering and dosing equipment.

In order to obviate evaporation of gases, it is desirable to use an enclosed, flow-restriction device. Flow of the dosed water should be directly into a properly designed reaction tank.

The accuracy of a gas feeder is determined by that of the measuring device incorporated. For a rotameter this accuracy is ± 2%.

7.3 MIXING DEVICES

The major aspect of this unit operation is the creation of turbulent, eddy motion. Mixing can be carried out in two types of device. In the one, the hydraulic energy of the water flowing over and through static devices is utilised; in the other, the necessary energy is supplied by an external power source. The external power source can be either a motor-driven turbine, propeller or air bubbles.

The simplest static, hydraulic mixing device exists in water works where a pipe emerges into a channel. Propeller devices are used to transmit energy from an electric motor into the water, at or near the chemical dosing point. Energy can also be transmitted to water by air bubbles; this method of mixing is useful in flotation where the induced buoyancy is advantageous.

7.3.1 HYDRAULIC MIXING DEVICES

These hydraulic blenders are simple and are effective at design flow. There is a variety of static devices which can be used for this purpose. They operate under gravity and mixing is achieved at the expense of pressure head. Fixed walls or plates are used within a pipe, tube or chamber, to cause the necessary head loss and turbulence.

Having a fixed configuration, this type of mixer is inherently limited to a particular flow range. If the flow rate is too high, mixing may be inefficient due to short-circuiting. At low flows the chemical coagulation stage is passed and an undesirable amount of floc formation may be achieved. Unless specifically designed for, such floc settle or is broken up before it arrives in the flocculator or clarification tank.

A flow velocity greater than 2 m/s and a retention time < 1 s is generally desirable in these devices and these constitute the major design parameters.

Weirs, flumes, notches, orifices, pipes, valves and fittings can be used on most water works. These may well serve more than one purpose and are therefore particularly useful on smaller works. The most commonly used hydraulic and static dispersion devices are:

- inlet pipes and channels with turbulence at weirs,
- chambers or channels with baffles,
- pipes with flow restrictions such as orifices or flumes,
- impinging streams such as caused by energy mix nozzles.

7.3.1.1 Weir mixer

Mixing can be achieved where high turbulence exists at a weir. Feed is then done by means of a perforated pipe or a half-round channel with notches, mounted upstream and above the weir. This arrangement is used on plants of less than 150 Mℓ/d (AWWA, 1990).

For a flow Q i.e. V/t and replacing P by ρ g h V/t, in Equation 7.6, the velocity gradient created at the weir is:

$$G = [\, \rho\, g\, h \,/\, \mu\, t \,]^{1/2} \quad\text{.............................}\quad 7.7$$

For a weir mixer with downstream baffle, an effective head loss of 0,3 m, at a water temperature of 10°C i.e. μ of 1,31 kg/m s and a G value of 500 s^{-1}, the mixing time will be 9 s. If t were measured in practice the value of G can be calculated.

7.3.1.2 Baffled channels

Turbulence is increased in channel or pipe flow by means of baffles. This turbulence can be distributed and arranged by means of slots or perforations (Polasek, 1979). This type of unit is used on small works and where potential energy is available and electricity is expensive or unreliable. A slot mixer is shown in Figure 7.4.

For an n number slot arrangement, the total head loss h is given by

$$h = n\, C_d\, v_s^2 \,/\, 2g \quad\text{...................................}\quad 7.8$$

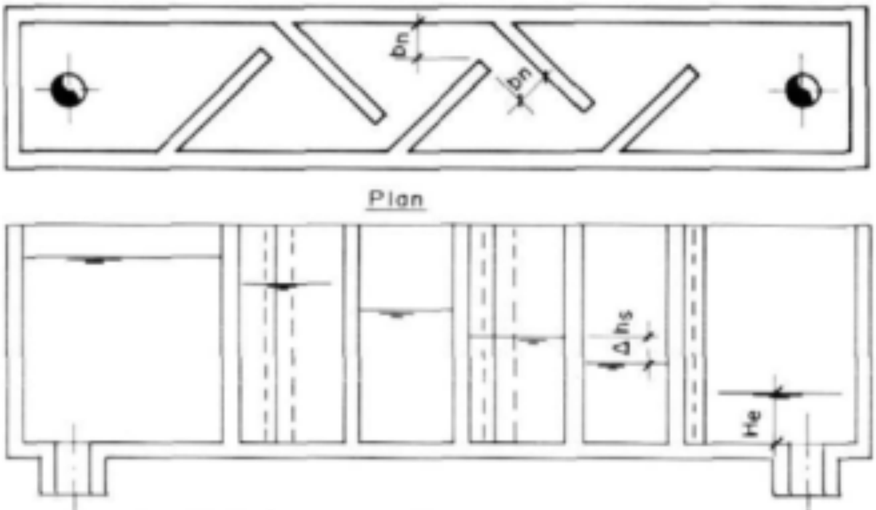


Figure 7.4
Slot mixer

where:

n = number of slots counted from the exit side of the mixer

C_d = coefficient of drag of 2 to 2,5 for baffles at 45° to flow and 3 to 3,5 for reverse flow

v_s = velocity in the slots (m/s)

In the open gravity arrangement shown in Figure 7.4, the slots are vertical. For a water level drop h_s at each slot to maintain constant velocity v_s the width of the slot has to increase in the direction of flow to give the same effective area A_s. The width b of n slots is:

$$b_n = A_s / (h_e + n\, h_s) \qquad 7.9$$

A perforated baffle mixer is shown in Figure 7.5. Two or three perforated baffles can be used when one chemical is dosed and three or four baffles are recommended when two chemicals are sequentially used (Polasek, 1979).

This mixer/flocculator can be used in upgrading of water works where space is limited.

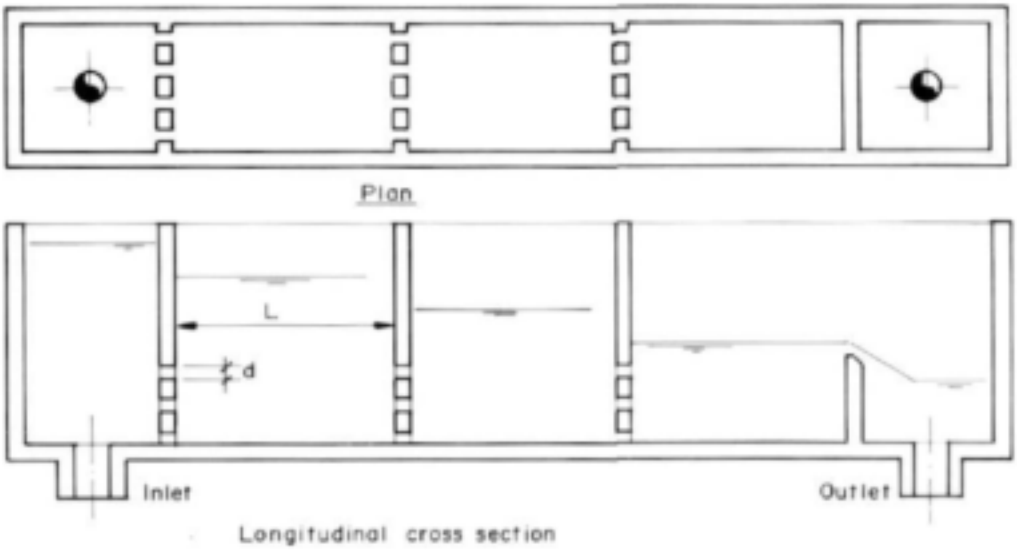


Figure 7.5
Perforated baffle mixer

The unit is sized to give a nominal flow velocity v_n of 0,4 to 0,6 m/s and a velocity v_h of 1,0 m/s in the holes.

Effective mixing is obtained for an L/d ratio of 5 to 7, where L is the distance between baffles and d, the diameter of the holes, is 20 to 100 mm. The head loss produced by flow through the perforated baffles is calculated as for a flow through a flooded opening in a vertical wall. For N baffles, the total head loss h is:

$$h = N\, v_h^2 / 2\, c^2\, g \qquad 7.10$$

where:

c = coefficient of discharge through the hole = 0,62 to 0,70

7.3.1.3 Flow restrictors

The orifice arrangement is simple and common but not the most effective hydraulic flash mixer because of the integrity of the stream flow. Mixing is achieved as a result of turbulence and eddies formed behind the orifice plate; the chemical is introduced as shown in Figure 7.6.

The head loss for a sharp-edged orifice plate is:

$$h = v^2/2\, g\, [1 - 0{,}707\, (1 - d_0^2/d^2) - d_0^2/d^2]^2 \qquad 7.11$$

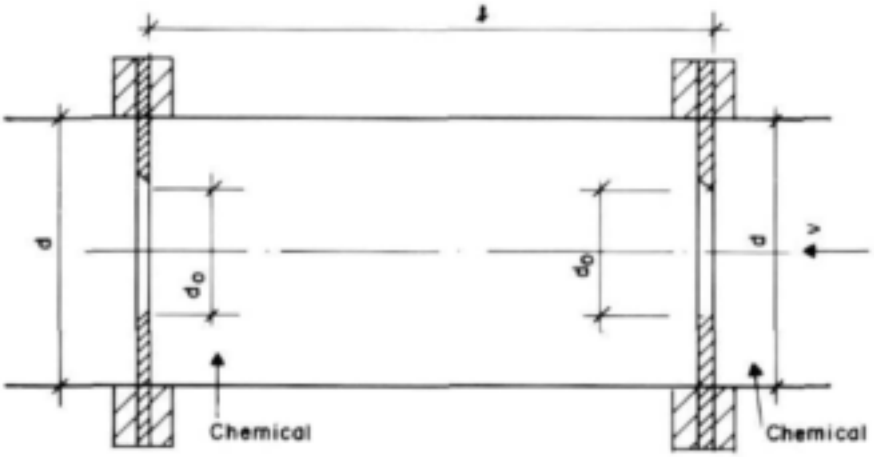


Figure 7.6
Orifice flash mixer

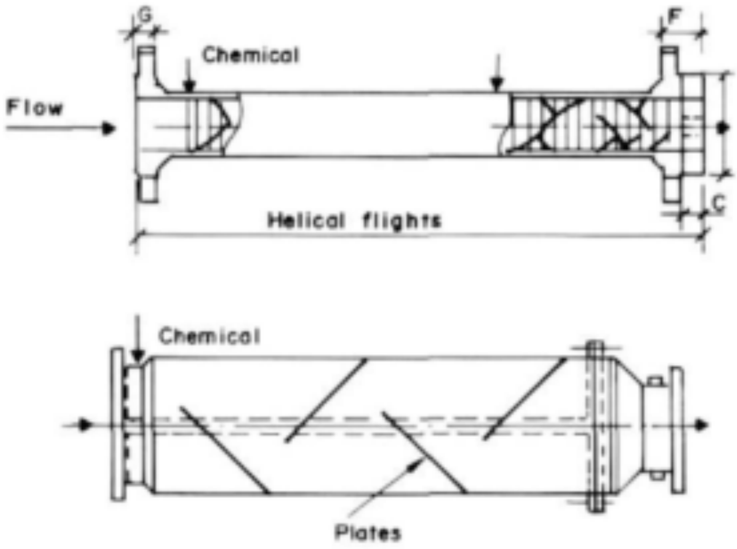


Figure 7.7
In-line static flash mixers

where:

v = velocity of water in the pipe (m/s)
d_o = diameter of orifice opening (m)
d = pipe diameter (m)

The mixing effect is improved by using two or more identical orifices. This is necessary if chemicals have to be added sequentially. The total head loss is the sum of the head losses at the orifices. A ℓ/d_o ratio of 5 to 7 is used for effective mixing (Polasek, 1979).

The problem of effective mixing can be overcome by providing a tortuous flow path as indicated in the helical mixer of Figure 7.7. It should be noted that the flight is repeatedly dislocated through 90° after each 360° of helix. The efficiency of mixing and energy loss in such a device has to be determined experimentally by the manufacturer.

7.3.1.4 Impinging streams

The jet stream flash mixer shown in Figure 7.8 consists of a cylinder housing two tangentially mounted jets and a baffle system. The head loss is determined as the loss in the flow being discharged through a flooded opening according to Equation 7.10 where v_h is the velocity of the jet and $n = 1$.

7.3.2 MECHANICAL MIXING DEVICES

Dispersion devices incorporating an external source of power are used on larger works. The design of these units can thus be performed with greater accuracy and yet flexibility. An advantage of mechanical mixing is that it can be adjusted to suit flow. Disper-

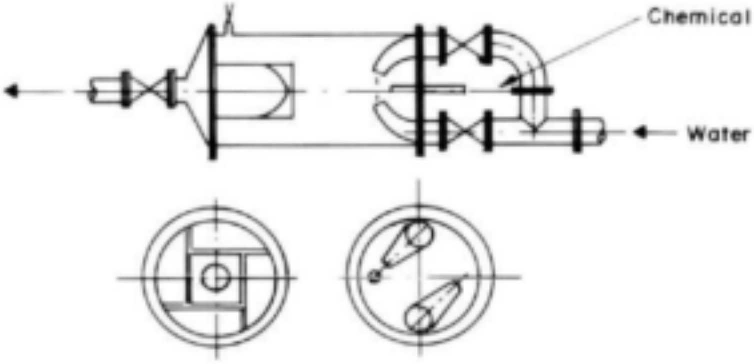


Figure 7.8
Jet stream flash mixer

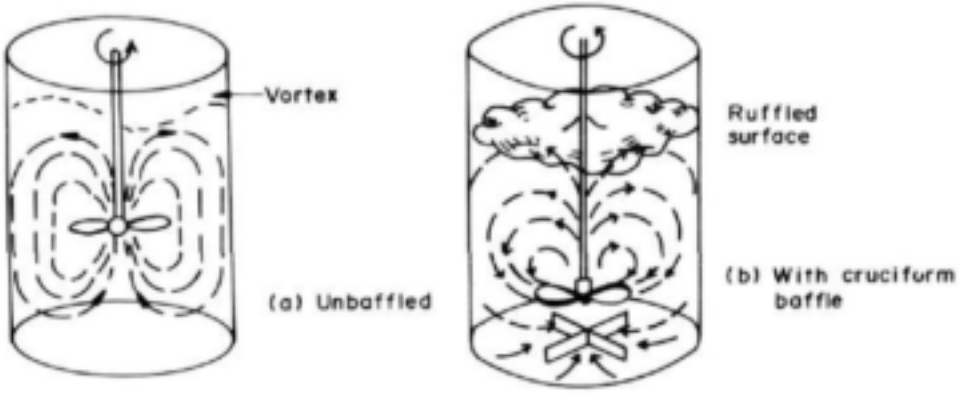


Figure 7.9
Propeller mixer flow patterns

sion devices require less power in operation than static devices. These devices rely, for their action, on entrainment and turbulent diffusion. Air-agitation devices have higher power consumption and might even be undesirable.

Mechanical devices may be grouped as follows:

- open pump in a tank, i.e. the suspended propeller or impeller,
- enclosed pump discharging into a pipeline or flume,
- air injection into a reactor.

7.3.2.1 Open pump

Open pump installations must be specifically designed for the purpose. They include motor, pump and tank. The reactor tank requires 2 to 60 seconds retention capacity. Motor speeds of 600 to 1 500 r/min are used, whilst shaft speeds are some 100 r/min. There is a large variety of impellers ranging from the flat bladed turbine to the curved Devereux propeller.

The flow pattern for propellers is shown in Figure 7.9. The tank may have circular or square cross-sections. The ratio of impeller or propeller diameter to tank is 0,2 for large tanks to 0,4 for small tanks.

If the tank cross section is square, a minimal amount of baffling is required; if the cross section is circular, it is necessary to provide baffling as shown.

If vortex motion is allowed, the propeller shaft length should not exceed 2 m otherwise a foot bearing is necessary.

A typical flash mixing installation is shown in Figure 7.10. It will be noted that the inlet is positioned in such a way that the water flows axially past the propeller and out at the top. Provision is also made in this instance to curb the tendency for vortex formation by means of a baffle. The point of addition of chemical is very important and should be carefully noted.

Example

For $G = 750\ s^{-1}$ and viscosity at 15°C $= 1{,}15 \times 10^{-3}$ kg/m.s for a mixer in a tank of 1,2 x 1,2 x 1,75 m deep, power required will be:

from Equation 7.6:

$$P = G^2 \mu V$$

therefore:

$$P = 750 \times 750 \times 1{,}15 \times 10^{-3} \times 1{,}2 \times 1{,}2 \times 1{,}75 = 1\,630 \text{ watt}$$

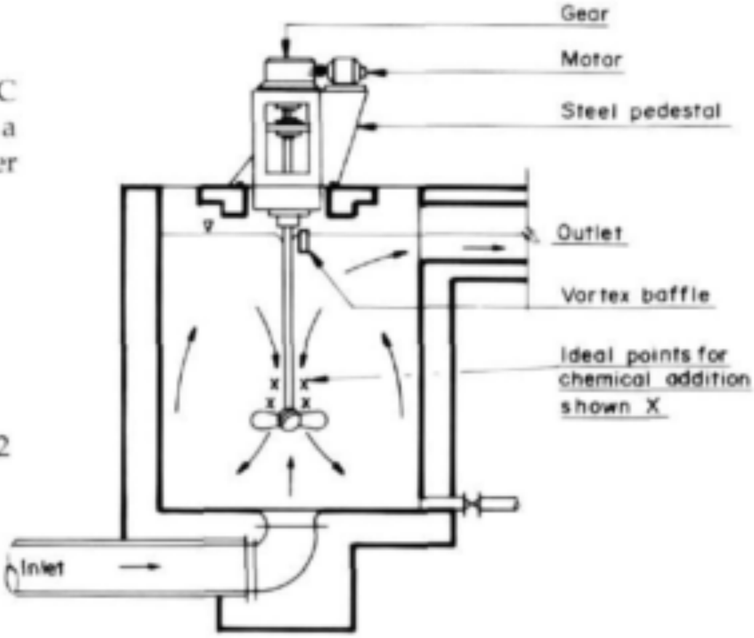


Figure 7.10
Flash mixing tank installation

7.3.2.2 Enclosed pump

This in-line blender is efficient because it can be custom designed and is adjustable to flow. The enclosed type of pump has a mechanical, usually propeller, mixer. It can provide a constant G over a wide range of flow. Low-lift pumps may be used for this purpose but they lack the flexibility of other methods. Enclosed pumps can also be used for mixing if they are at the same time required for other purposes.

An enclosed pump mixer is shown in Figure 7.11 (AWWA, 1990). It will be clear that the chemicals are injected counter flow wise into the works inlet pipe, thus ensuring effective mixing. The energy input can be varied by throttling the flow; the pumped volume of water should not exceed 100 times the volume of alum solution.

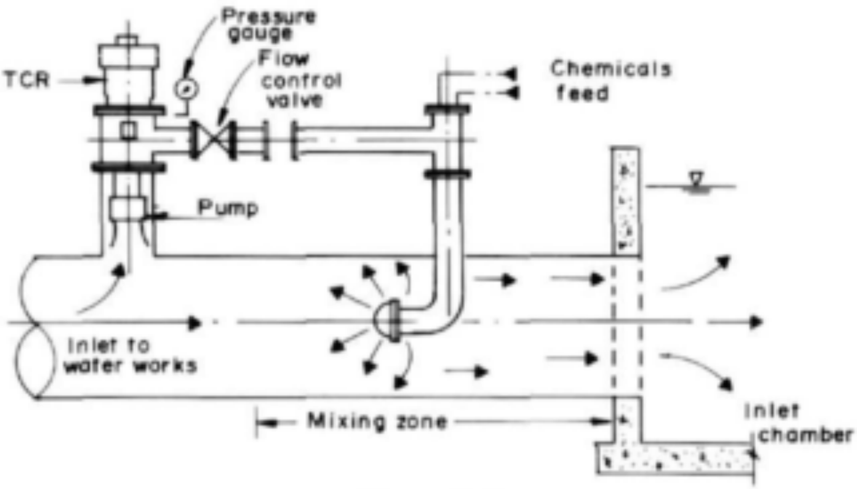


Figure 7.11
Enclosed pump mixer

In-line blenders require a flow period of < 2 s at up to 5 m/s before entry into a flocculator. These devices are proprietary and can be specified to have any required G value.

TABLE 7.1
DESIGN CRITERIA FOR DOSING AND MIXING

Method of dosing	**Comments and criteria**
Dosing Dry feeders	Volumetric or gravimetric types; small space required; accuracy of belt, table or screw is 3% to 5%; gravimetric feed, with balance has up to 1% accuracy; used in conjunction with solutioniser
Wet feeders	Accuracy determined by flow measurement control at <2%
Pot feeder	Used for coarse and low-solubility chemicals on small works
Decanting feeder	Arm decants at rate proportional to flow
Tipping doser	Hydraulically operated on counter-balance principle
Valve	Modulated control capability; external signal can control flow of chemical under constant head
Reciprocating pump	Positive displacement by adjustable stroke; can be automated for flow rate and solution strength
Mixing	Judgement necessary to determine volume of zone where energy is dissipated. At pipe flow velocity 0,5 m/s volume can be taken as 2 s of maximum flow; mixing should take place in 1 to 3 s (15 to 30 s previously). (Otherwise the total volume of the basin should be taken.)
Hydraulic devices	Simple, effective, non-mechanical, but energy varies with flow. Sufficient head and space required; can be small. Efficient due to high degree of turbulence and little or no short-circuiting, if properly designed. Velocity gradients of 600 to 1000 s^{-1} are used for alum and polymer mixing (300 s^{-1} used in the past).
Weirs	Used on smaller works
Flumes	Undesirable where entrainment is a problem
Notches	Useful on small works; includes flow measurement
Orifices	Includes flow measurement
Pipes	Multiple injection points can be useful; minimum pipe length of 30 diameters is necessary
Valves	Serves dual purpose
Mechanical equipment	Used commonly on larger works. Requires maintenance. Mounted in tanks or cylinders smaller than 3,8 m^3 or less than 1,8 m in diameter. Custom designed and built or manufactured. Power need is 0,03 to 0,13 kW per Mℓ/d.
Open pump	Are efficient; effective at all flows. Motors are less than 2,2 kW with speeds 600 to 1500 r/min. Shaft speed 80 to 100 r/min. Tanks are square or round (baffled); impeller to tank ratio 0,2 to 0,4; propeller tip speed maximum is 5 m/s to limit shear of long chain polymers.

Method of dosing	**Comments and criteria**
Enclosed pump	This propeller mixer is effective at all flows. G value can be specified. Chemicals injected counter flow for very effective mixing. Energy input varied by throttling flow. Pumped volume of water not > 100 x volume of alum solution. Flow period < 2 s at up to 5 m/s before entry into a flocculator.
Air	Simple, good if aeration is required; may cause scum. Energy input efficient because it is readily adjusted.

7.3.2.3 Air mixers

Air mixing can be simple and reliable and is advantageous where aeration is required. It is readily incorporated into existing units especially deep conduits or vertical pipes. It is not widely used because of the introduction of buoyancy and possible scum formation (AWWA, 1990). It is clearly useful in flotation and is used extensively for that purpose.

The energy required (kW) can be computed as the volume of water displaced by free air discharge per unit time (q), multiplied by the depth below free surface (h) as follows:

$$P = q\,h\,/\,100\ \text{kW} \qquad 7.12$$

7.4 DESIGN CRITERIA

The objective of dosing and mixing is to inject the correct amount of chemical and to distribute it rapidly and uniformly in the water to be treated. Experimental results must be understood and judgement is necessary for the application of the theory because of scale effects and the large array of devices.

The choice of device or mechanism should be decided by space, cost, economics and power considerations. For example, if sufficient head is available on a small works a simple dosing mechanism and a static mixing device should be used, otherwise a mechanical mixer is preferable.

Methods, devices and equipment of dosing and mixing with general comments and design criteria are given in Table 7.1.

8

Flocculation

F A van Duuren

Flocculation is the hydrodynamic unit operation which results in the formation of floc, more readily settleable than the finely divided particles which compose it. It is therefore essential to phase separation. This unit operation is extensively used in the purification of water for domestic and industrial purposes.

Coagulation precedes flocculation. When the suspension enters a flocculator all the individual particles should have been covered by or attached to coagulants and possibly have formed microflocs. Flocculation takes place as the result of collision of particles and microflocs to form macroflocs.

Effective flocculation requires particle numbers and collision opportunity. It was shown conclusively that without motion, colloidal particles will remain in suspension for years despite the addition of an effective coagulant to a suspension (Stamberger, 1962).

For the purpose of flocculation, collisions are caused by velocity gradients in a flocculator. When destabilised primary particles collide, they generate 2-fold particles. During the course of flocculation, floc particles of numerous sizes appear as a result of collisions of primary particles and 2-fold particles which produce 3-fold particles, and collisions of 2-fold particles and 3-fold articles which produce 4-, 5- and 6-fold particles and so on.

Flocculation is viewed here in terms of the theory, floc characteristics, design considerations, types of flocculators and practical examples.

8.1 THEORY OF FLOCCULATION

Particles must collide in order to flocculate. To facilitate collision, there must be motion and a sufficient concentration of particles. The rate at which collision can occur during flocculation is thus a function of the number of particles involved, their size and the rate of motion.

Collisions take place as a result of Brownian motion, flow past a sphere, differential sedimentation, velocity gradient and turbulence (Robinson, 1964).

8.1.1 BROWNIAN MOTION

Brownian motion occurs in a colloidal suspension because of thermodynamic effects, resulting in random motion of like electrically charged particles. This motion is visible when a shining light is observed through a stationary colloidal suspension to cause the particles to be visible as tiny bright spots; this is due to the Tyndall effect. The equation of kinetics for coagulation of dispersed colloidal particles into small agglomerates is thoroughly established.

Brownian motion is represented by the Von Smoluchowski equation

$$1/n_2 - 1/n_1 = k\,(t_2 - t_1) \qquad 8.1$$

where:

n_1, n_2 = number concentration of particles at times t_1 and t_2,

k = constant

As indicated in Chapter 6 (Equation 6.5) flocculation collision frequency exceeds collision due to coagulation very substantially in the formation of floc larger than 10 micrometre. Most floc particles are larger and Brownian motion therefore has a negligible effect on flocculation.

8.1.2 FLOW PAST A SPHERE

Collisions due to flow of a suspension past a sphere may be expressed in the form of the equation:

$$N = \pi/4 \,.\, v_a \,.\, d_2^{\,2} \,.\, n_2 \,[{}^3/_2 - d_2 \,/\, 2(d_1 + d_2)] \ldots\ldots 8.2$$

where:

N = number of collisions occurring per unit volume in unit time
n_1, n_2 = number concentration of particles of diameter d_1 and d_2
v_a = approach velocity of the fluid to sphere.

Flow past a sphere is especially applicable for particles of greatly different sizes; this occurs in solids contact clarifiers.

8.1.3 DIFFERENTIAL SEDIMENTATION

Collision takes place due to differential settling as microflocs differ in weight, size and shape. Fast settling particles collide with slower ones in their flow space under quiescent conditions. Rising particles collide with stationary or falling particles in coincident flow spaces. This can be expressed by the equation:

$$N = n_1 \,.\, n_2 \,.\, \pi/4 \,.\, (d_1 + d_2)^2 (v_2 - v_1) \ldots.\ 8.3$$

where the particles settle at velocities v_1 and v_2.

Differential sedimentation takes place in sedimentation tanks and especially in clarifiers.

8.1.4 VELOCITY GRADIENT

The stirring of water, mechanically or otherwise, imparts velocity to it and to the colloids or suspended matter in it. Differences in velocity across the path of flow create velocity gradients. Most flocs formed in water are fragile. As they grow in size, the velocity gradient across them increases and they tend to be broken up. This mechanism of flocculation is, therefore, of particular interest in water clarification.

Dependence of flocculation rate, N, on the velocity gradient, normal to streamlines, may be expressed as:

$$N = 1/6\, n_1 \,.\, n_2 \,.\, G\, (d_1 + d_2)^3 \ldots\ldots\ldots\ldots\ldots\ 8.4$$

where

N = number of collisions occurring per unit volume, in unit time, and is proportional to - dn/dt
n_1, n_2 = concentration of particles of diameter d_1, d_2 in the system
G = velocity gradient in a shearing fluid
= difference in velocity dv in a given distance dy normal to the direction of flow.

Viscosity does not play a role in flocculation rate; it does, however, affect energy expenditure.

In a study of viscosity, it was stated (Robinson, 1939) that maintenance of flow of a Newtonian liquid requires expenditure of energy at a rate proportional to the volume of the liquid and to the square of the velocity gradient maintained in it. The constant of proportionality is the coefficient of viscosity. Expressed mathematically, power P is proportional to volume V and to velocity gradient G squared, hence:

P is proportional to $V G^2$ or $P = \mu V G^2$ and thus the well-known equation:

$$G = [P / \mu V]^{½} \qquad 8.5$$

where:

P = energy input
μ = viscosity
V = volume

Equation 8.5 was subsequently introduced into the water field (Camp & Stein, 1943) for use in flocculation of particles in a suspension. As indicated in Chapter 7 the same Equation 7.6 is also used as a mixing energy design guide; its use in flocculation is soundly based.

Considering Equations 8.4 and 8.5 together, it is seen that the flocculation rate, as expressed by the number of contacts per unit time, increases with the number of particles, the power input and especially the size of the particles but is inversely proportional to viscosity and volume. There is, however, a limit to floc growth. The total available binding energy decreases with the neutralisation of charge and diminished bridging sites until the floc reaches a size at which the shearing action is greater than the tendency for aggregation.

Break-up of floc occurs by shear when too high a velocity gradient pertains. The maximum radius of a floc r_m is governed by a factor k which is a measure of the motion transmitted by the liquid to the particle and of the toughness of the floc and hence coagulation. This may be expressed as:

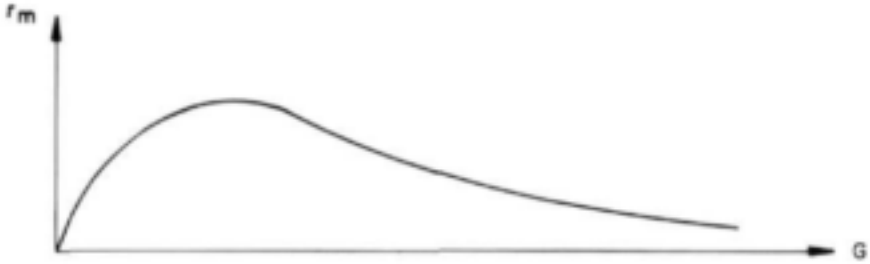


Figure 8.1
Floc size and velocity gradient G

$$r_m = k . v_r / \mu . G \qquad 8.6$$

where v_r is the velocity at the outer edge of the floc, μ is the viscosity and G is the velocity gradient (Fig. 8.1).

A further distinction was made (Argaman & Kaufman, 1970) which takes into account the nature of the particle. This refinement introduced an agglomeration and a breakdown rate. Flocculation time is expressed in terms of the ratio of these two rates. A dimensionless constant (k_a) is used for agglomeration and a time-related constant (k_b) is used for breakdown.

From experiments performed (Bratby, 1980) it appears that for a paddle flocculator, velocity gradient (G) can be estimated in terms of the rotation rate of the paddles. The value of k_a for turbidity is $2{,}5 \times 10^{-4}$ and it is $3{,}09 \times 10^{-4} k_b$ for colour, yielding equilibrium flocculation times of 2 to 15 minutes. Practical experience indicates retention times up to 30 minutes in a channel flocculator and < 5 minutes in a paddle flocculator.

8.1.5 TURBULENCE EFFECTS

Collisions will take place due to random transportation by local fluctuations under turbulent conditions. The random behaviour of particles under the influence of turbulence is analogous to gas molecules in a reacting system. In the liquid system, the local velocity gradient G between streamlines is the analogue of the gas reaction rate factor.

Maximum collision rates are achieved under uniformly turbulent conditions and the number of unflocculated particles are at a minimum. The floc produced under turbulent conditions has a small size, small size range and a high density.

Turbulence is the general state in flocculators whether hydraulic or mechanical energy is used. Reynolds numbers in excess of 10 000 is generally applicable.

8.1.6 PELLET FLOCCULATION

Pellet flocculation is an innovative extension of conventional contact flocculation. The pelleting operation of a concentrated suspension was first proposed as a promising process to dewater coagulated sludges. A horizontal drum dewatering apparatus was used for the purpose with polymer addition (Yusa & Gaudin, 1964).

By introduction of a suitable polyelectrolyte prior to the introduction of a suspension into an agitated floc blanket, high-density pellet like flocs are generated.

The moderate intensity of agitation provided by rotating paddles prevents the random local growth of primary particles on a surface. Flocculation of the adhered elementary particles on a surface results in a good spherical pellet, with low void volume. As a result, regular layered deposition of the elementary particle occurs, and high-density pellets are generated.

8.2 FLOC CHARACTERISTICS

The characteristics of a grown floc is determined by three major factors i.e. the primary particles, coagulation and flocculation. Particle concentration is important; if it is too low it can be increased by the addition of for instance bentonite, activated carbon or recycled sludge.

In the case of aluminium-clay flocs, relatively small numbers of clay in comparison with aluminium and OH ions, cause the reaction of aluminium hydroxide generation to predominate at the first stage of coagulation. The weakly positively charged, semi-precipitated aluminium hydroxide destabilises and bridges clay particles. In the higher clay concentration, the ratio of aluminium or aquo-aluminium complex which reacts directly with functional groups on the clay becomes slightly higher than the lower concentration suspension and the efficiency of coagulant dosage ratio is improved.

The charge-neutralised aluminium-humic complexes are bridged by precipitated aluminium hydroxide to form settleable flocs. The size of the humic substances to be coagulated is much smaller than that of the effective aluminium aquo-complex. Normal coagulation or even mutual coagulation does not proceed effectively and poor flocculation results.

The physical aspects of floc particles that are important in flocculation and separation processes are the density and strength of a floc.

8.2.1 FLOC DENSITY

Density of floc is an important factor as it controls settling velocity in a sedimentation tank and affects head loss in a filter.

Floc density has been found experimentally to be a function of floc size (Tambo & Watanabe, 1979). This can be expressed as

$$\rho_e = a / (d_f / L_c)^{k}f \quad \text{.................................} \quad 8.7$$

where:

ρ_e = effective floc density = $\rho_s - \rho_w$; for ρ_s being the density of the solids and ρ_w the density of water,
d_f = floc diameter (cm) and
a and k_f are constants
L_c = dimension constant

The above-mentioned relationship between floc size and density is universally observed with flocs generated in a conventional flocculator with random collisions of many small flocs.

Densities of a variety of particles and floc (Masschelein, 1992) are given in Table 8.1.

Table 8.1
Densities of Particles and Floc

Particle and nature of floc		Density (kg/dm³)
Softening precipitate		1,15
Crystalline calcium carbonate		1,20
Activated carbon powder		1,2 to 1,4
Bentonite		2,0 to 2,2
Silica sand		2,65
Alum or ferric floc:	Clay	1,005 to 1,010
	Colour	1,001 to 1,004
	Algae	1,001 to 1,002
Alum or Fe plus polyacrylamide	Clay	1,019 to 1,038
	Colour	1,002 to 1,005
Cationic, coagulant aid and G = 300 s^{-1}		1,050
Alum and benotine floc		1,018 to 1,104
Organic waste and polyelectrolyte floc		1,001

8.2.2 Floc strength

The strength or binding force of a floc controls the maximum attainable size of the agglomerate under an agitation condition or shear field. The strength of a floc particle has high correlation with its adhesion to a filter medium particle.

Floc strength can be defined directly or indirectly in many ways. However, for practical purposes the floc diameter of an equilibrium particle size distribution of floc being stirred is used as a measure of floc strength. Floc size varies from 0,01 to about 3 mm.

8.3 Design Considerations

This unit operation requires careful consideration in conjunction with coagulation and the subsequent phase separation. Flocculation does not have to be done in a separate unit but it is an essential component of the system of water purification processes.

For flocculation to take place particles must either come within the sphere of influence of the perikinetic force which causes Brownian motion or they should be subjected to the orthokinetic forces which will cause collision. The nature and limitations of these mechanisms play a role in design and the criteria used.

8.3.1 Brownian motion

Flocculation due to Brownian motion is limited to the particles in a colloidal system, in fogs and aerosols. When gravity becomes significant, flocculation takes place as a result of floc collisions only.

8.3.2 Flow patterns

The nature of the flow in a flocculator is of importance to the extent that short-circuiting should be minimised or obviated in order to optimise particle collision opportunity. Clearly plug flow is desirable especially for reducing value of Go whereas a completely mixed reactor would facilitate uniformity of the floc size. Flow patterns are dealt with to some extent in Chapter 9.

8.3.3 DIFFERENTIAL SEDIMENTATION

Differential sedimentation occurs generally but is of limited application unless favourable conditions are created. It occurs where concentration of solids is sufficiently high, especially where flocculation is incorporated in clarifiers or in sedimentation, bringing microflocs in contact with larger flocs already formed.

8.3.4 OPTIMUM VELOCITY GRADIENT

This will yield the maximum or optimum size of floc. At higher than optimum velocity gradient, a smaller floc of smaller size range and greater density will result at the expense of power. At lower velocity gradient large flocs result.

Good flocculation is achieved in a suspension subjected to uniform turbulence and optimum shear rate. The magnitude of shear should decrease smoothly as floc size increases, otherwise the floc is broken. Numerous small eddies are preferable to a few large, localised ones in order to ensure the maximum number of collisions. The initial stages of flocculation produce high toughness floc around which more material can be added. Taper flocculation has generally been considered to be ideal.

8.3.5 FLOCCULATION INTENSITY INDEX

The mean kinetic index of flocculation intensity can be described by the parameter $G = (E/\mu)^{1/2}$ in which E is the total energy dissipated per unit time per unit volume of fluid, and μ is the viscosity. Mean velocity gradient, G (s^{-1}), is generally used as the flocculation intensity index for turbulent flocculator design.

8.3.6 FLOCCULATION, SEDIMENTATION AND FLOTATION

In order to settle, particles must be dense and large; this is clearly seen from Stokes' equation (9.1).

Sedimentation is significantly affected by flocculation. Optimum velocity gradients can be determined for given alum dosages and particle concentrations (Van Duuren, 1967). Optimum velocity gradient is lower for a small alum dosage than for a higher dosage; this is probably due to increase in floc density at higher G and to low water content of the floc.

For concentrations of clay particles up to 500 mg/ℓ and alum dosages up to 50 mg/ℓ, optimum G is between 10 and 30 s^{-1}, corresponding to a Gt of 12 000 to 36 000 and optimum GtC_O of 1 to 6.

Particles must be buoyant for flotation purposes; they can therefore have low density.

8.3.7 FLOCCULATION AND FILTRABILITY

The relationship between flocculation and filtration can be expressed in terms of the product of velocity gradient, period of flocculation and initial turbidity (C_O) versus a dimensionless filtration index. This index, based on head loss, initial turbidity, flow rate, period of run, initial and final turbidities is described in Chapter 12.

The flocculation product $G\,t\,C_0$, yields optimum filtration at a value of 0,5 to 1,0 for a range of dosages of alum (Van Duuren, 1967). This is a useful parameter with regard to flocculation and the other unit operations (Ives, 1992; Tambo, 1990).

8.4 TYPES OF FLOCCULATORS

The main types of flocculators including the laboratory jar and paddle are baffled channels, mechanical agitators, solids-contact devices, filters and flotation units. These are discussed against the given theoretical background and in terms of design parameters.

8.4.1 LABORATORY FLOCCULATORS

Various laboratory flocculators have been evolved. A defined velocity gradient flocculator, the Couette device, was developed and used extensively for controlled laboratory work (Van Duuren, 1967). The laminar flow Couette principle is still used in standard viscometers. Possibly the best known laboratory flocculator is the jar test machine.

Jar test machines were evaluated in terms of a defined velocity gradient device (Van Duuren, 1967) it and it was concluded that it can be used as a laboratory flocculator. Flocculation can be carried out in a laboratory, in a beaker, or by means of a paddle stirrer. The flow pattern in the jar depends on the stirrer and on baffles used, if any.

The purpose of a laboratory jar test is primarily to subject any number of samples of water to the same mechanical action in each test. It is therefore necessary to standardise the jar test machine, equipment and procedure for coagulation/flocculation purposes. The jar test procedure for coagulation is described in detail in Chapter 6.

Because of the good correlation between the Couette device and the jar test machine and the simplicity of the latter, the jar test machine can be used for flocculation tests in water works laboratories. The problem of determining the velocity gradient applied in a jar test does, however, remain.

The two laboratory flocculation devices are shown in Figure 8.2.

8.4.2 CHANNEL FLOCCULATORS

Baffled channels of over-and-under or to-and-fro type are still used; these are illustrated in Figure 8.3. Baffled channels suffer from greatly varying velocity gradients and flow discontinuity. This type of flocculator has been improved by tapering the compartment size from small to large. Baffled channel flocculators are usually used in small to medium sized water works (< 50 Mℓ/d).

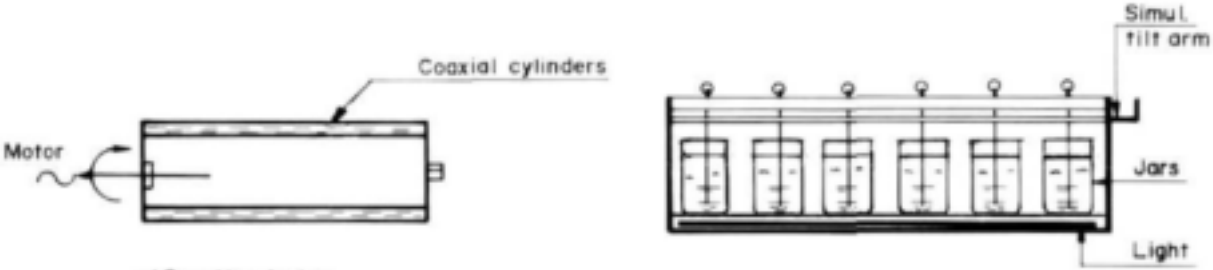
For a channel which has a head loss h, water flow Q, liquid density ρ and viscosity μ,

$$P = Q \,.\, \rho \,.\, g\,h \quad\text{.......................................}\quad 8.8$$

From Equations 8.5 and 8.8, for μ/ρ i.e. kinematic viscosity ν and for retention time t (V/Q), the mean velocity gradient is:

$$G = \left[\frac{g \,.\, h}{\nu \,.\, t} \right]^{1/2} \quad\text{.......................................}\quad 8.9$$

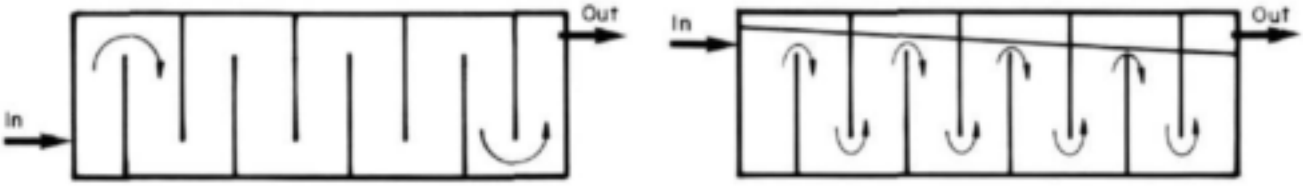
Velocity gradient calculation is rather theoretical for a baffled channel flocculator because of the highly non-uniform shear rate. Eddies at the ends of baffles cause high turbulence and disrupt floc which is then swept into suspension again, possibly to settle out



a) Couette device

b) Jar test machine

Figure 8.2
Laboratory flocculators



a) To-and-fro flow

b) Over-and-under flow

Figure 8.3
Baffled channels flocculators

between baffles due to quiescence.

Particular care must be taken with regard to the connection between the channel and the sedimentation tank to prevent floc destruction or premature floc settling. All the useful work done in creating floc may well be lost because of lack of attention to these aspects.

The problem of poor velocity gradient distribution of the baffled tank has been overcome by means of a tapered width flocculator. Flow pattern tests were performed in a model of such a flocculator, shown in Figure 8.4 (Schmit, 1976).

A tapered, spiral channel flocculator, has been used by Rand Water (RW) for many years. The capacity of that part of the water works served by the spiral flocculator is 450 Mℓ/d.

The inlet to the RW flocculator has a velocity gradient of some 600 s^{-1}, it is 6,6 m in diameter, has a water depth of 4 m and a volume of 139 m^3. The coagulants used in this unit are injected some 5 m from the central inlet. Activated silica (1 to 3 mg/ℓ) is injected 75 s upstream and lime (55 to 70 mg/ℓ) 60 s upstream of the central inlet.

The width of the flocculator canal varies from 1,85 m at the inlet to 2,7 m at the outlet. It has a water depth of 4 m and the water volume is 3 183 m^3.

The velocity gradients were calculated from a head loss equation to be 135 s^{-1} at the inlet, 42 s^{-1} at the outlet with an average of 60 s^{-1} for the whole canal. The product G T is 41 300, giving a retention time of 11,5 minutes. Thus the G T C value is 2 for the average turbidity of 50 mg/ℓ which pertained in 1991.

8.4.3 SOLIDS CONTACT FLOCCULATORS

In solids contact clarifiers, well-grown floc are circulated in a turbulent flow field or sus-

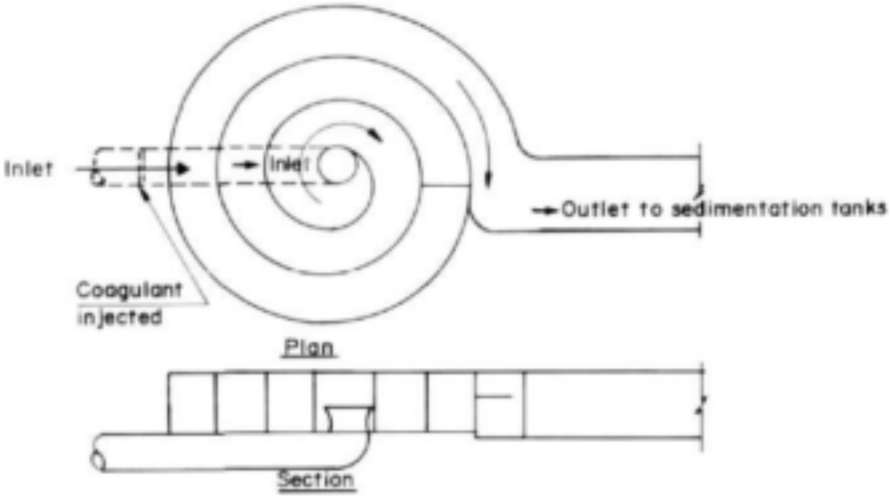


Figure 8.4
Spiral channel flocculator

pended in an upflow stream. Collisions of grown floc in the tank and incoming microfloc therefore occur. Because of the presence of the grown floc at substantial concentration, resulting in high G T C values, the required duration of flocculation can be shortened remarkably.

The size distribution of the grown flocs generally reaches the final equilibrium size distribution distributed between the maximum attainable floc diameter and one half of the maximum diameter (Tambo & Watanabe, 1979b).

In this type of flocculator, the mechanism involved is the collision of microfloc with floc already formed, but still in motion, or collision with stationary particles. Motion of the microfloc and particles to be flocculated is due entirely to the flow of the water which sweeps them along and into contact with the macroflocs. This flocculation mechanism occurs also in filters.

A variety of these flocculators is used in clarifiers. A number of these are shown in Chapter 10; most of them incorporate a mechanical paddle system. A purely hydraulic type of flocculator can also be used effectively. An example of such a flocculator is shown in Figure 8.5.

In the tank shown, the coagulated particle suspension flows tangentially into the hydraulic flocculator. This is contained between the tank and a widening conical section. The spiral flow is stabilised by means of guide vanes before it flows upwards again into a widening flocculator zone. Floc is decanted from the top of the blanket into a consolidation or sludge zone. The continuous widening of flocculator means that this is a tapered system.

The mechanism of flocculation in this unit is clearly differential sedimentation and flow past a sphere. The opportunity for collision or contact is considerable because of the uniformity of flow attainable in a hydraulically, properly designed tank.

For a head loss h and a kinematic viscosity v, a mean velocity gradient G and desired retention time t can be inserted into Equation 8.9 to obtain the head loss:

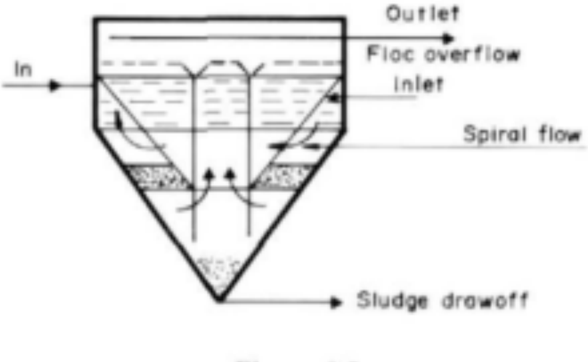
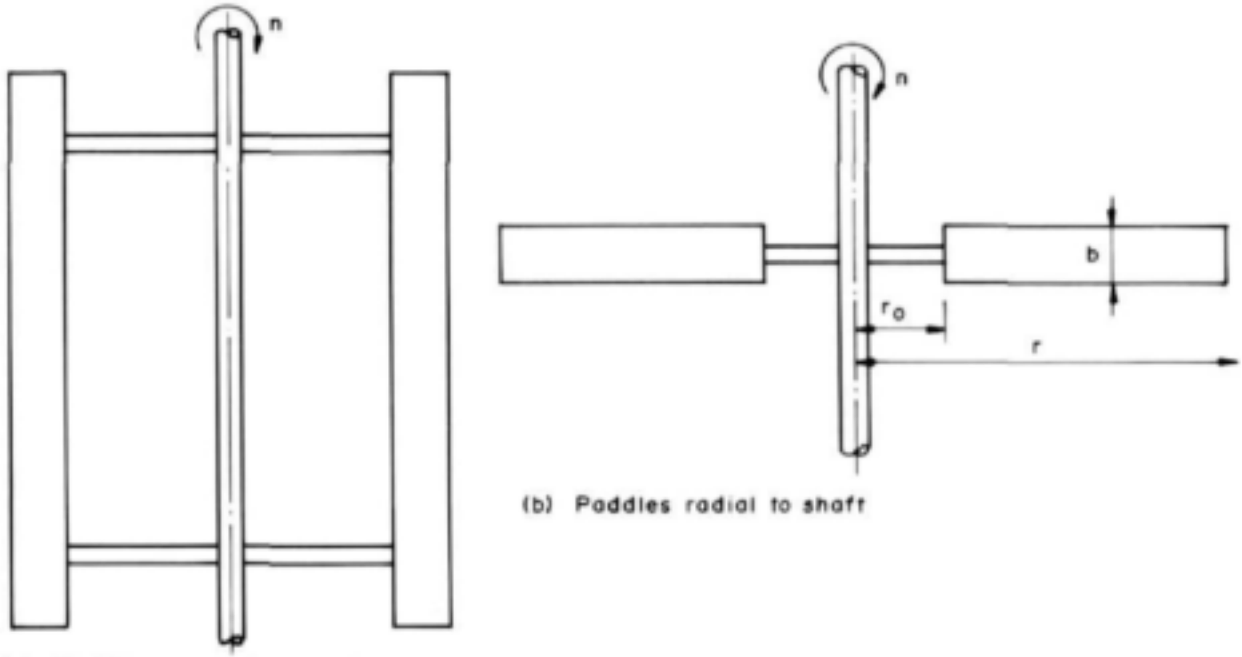


Figure 8.5
Floc blanket flocculator



(a) Paddles parallel to shaft

(b) Paddles radial to shaft

Figure 8.6
Revolving agitator flocculators

$$h = G^2 . v . t / g \qquad 8.10$$

For a known suspended solids concentration, the retention time t can be obtained for an optimum G T C of 1 to 2.

8.4.4 MECHANICAL FLOCCULATORS

Mechanical devices are often used for flocculation purposes. These devices include paddle systems of numerous shapes and design. Propellers and turbines are used for hydrodynamic purposes only. Air diffusers or inducers are used to cause hydraulic agitation and aeration.

8.4.4.1 Paddle systems

The sizes and speeds of motor and drive and the sizes and shapes of paddle facilitate a wide range of velocity gradient for an installation. Taper flocculation can also be achieved in mechanical flocculators by means of staged paddle systems with progressively decreasing velocities.

Paddles may be mounted with their axes vertically or horizontally as shown in Figure 8.6. Whether the axis of the stirrer is to be mounted vertically or horizontally is also influenced by a consideration of such aspects as glands and bearings involved in the mechanical equipment. Underwater glands and bearings require careful design operation and maintenance.

An aspect of particular importance with a rotating paddle system is the flow direction relative to the axis. Flow may be perpendicular to or along the axis of the flocculator.

If the water flows through a flocculator at right angles to the axis of the stirrer, some recirculation takes place. This is undesirable as by-passing takes place. Control can be achieved by means of vanes or slots in the sides of the tank, but this is only applicable

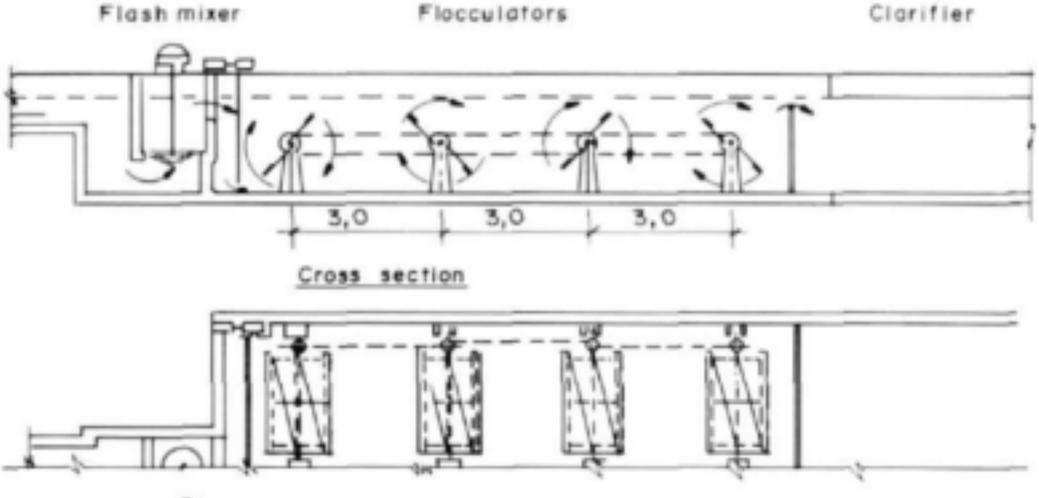


Figure 8.7
Paddle flocculator system

to fairly small tanks.

If flow is parallel to the stirrer axis little recirculation occurs and the whole stream is evenly affected; this is, therefore, the preferable flow direction. For flow normal to the paddles, fixed pickets and contra rotation of successive sets of paddles will reduce or obviate short circuiting.

A carefully designed paddle flocculator system, preceded by a flash mixer is shown in Figure 8.7.

Paddle shape is important as it will affect the flow pattern. The main consideration is, however, the achievement of the maximum number of particle collisions for the lowest power consumption. Streamlined blades would give rise to high local shear and yield few eddies whereas flat blades result in intense eddying.

The power input for a paddle flocculator system determines the velocity gradient and, therefore, the characteristics of the floc. For minimum power consumption, the optimum velocity gradient should be attained at all points in the suspension. This is not easily achieved with a paddle system, but a large number of paddles of small area are preferable to a few paddles of large area.

The useful power input is a function of the drag of the paddles, i.e.

$$P = C_D \, . \, A \, . \, \rho \, . \, v^2 /2 \quad \ldots\ldots\ldots\ldots 8.11$$

and from Equation 8.5, velocity gradient is given:

$$G = \left[\frac{C_D \, . \, A \, . \, v^3}{2 \, . \, \nu \, . V} \right]^{1/2} \quad \ldots\ldots\ldots\ldots 8.12$$

where:

C_D = coefficient of drag for paddles moved face-on to the suspension
A = area of paddles
v = velocity of paddles relative to liquid
V = volume of flocculator
ν = kinematic viscosity

In practice, the peripheral speeds of paddles range from 0,1 to 1,0 m/s.

For an eight blade (0,25 x 3,0 m) paddle flocculator (effective radius 1,5 m) rotating at 4,5 r/min (0,8 relative velocity) in a 150 m³ tank; given that ν is 1,31 x 10^{-6} and C_D is 1,4 for the length to width ratio of 12 of the paddles, the velocity gradient is:

$$G = \left[\frac{1{,}4 \,.\, 8 \,.\, 0{,}25 \,.\, 3 \,.\, (0{,}8 \,.\, 2 \,.\, \pi \,.\, 1{,}5 \,.\, 4{,}5/60)^{3}}{2 \,.\, 1{,}31 \,.\, 10^{-6} \,.\, 150} \right]^{1/2}$$

$$= 40\ s^{-1}$$

A second set of the same paddles rotating at 2,5 r/min in a subsequent tank of the same volume results in a velocity gradient of 25,7 s^{-1}. For a flow of 900 m^3/h the total retention time in the flocculators will be 20 minutes.

For a length to width ratio of 2 to 4, Cd is 1,19, for 4 to 10 it is 1,29 and in excess of 18, Cd is 2,0 (Polasek, 1979).

8.4.4.2 Propellers and turbines

Propeller and turbines are used to some extent in water clarification plant especially of the solids contact type where flocculation is enhanced by continuous recirculation of floc. In this device, a balance is attained between heavier floc which settles out and lighter floc which is re-entrained. The heavy floc may be removed as sludge at the bottom of the tank preferably by using an overflow device where some of the heavy floc is allowed to decant into a separate compartment.

8.4.4.3 Tube flocculators

Pipes may also be used as flocculators; they operate by virtue of head loss and turbulent flow. At a sufficiently high Reynolds number (1000), turbulence is generally ensured in tubes. A mean velocity gradient can be calculated from head loss Equation 8.8 and a pipe head loss equation.

8.4.5 CLARIFIERS

There is a large variety of paddle flocculators in clarifiers and sedimentation tanks. The Dorr-Clariflocculator is a well-known radial flow sedimentation tank with a flocculator in a central inflow compartment.

An example of a mechanical paddle flocculator in the centre of a clarifier is shown in Figure 8.8. It is clear that this configuration should result in good flow continuity between flocculator, clarifier and sludge compartment. Examples of clarifiers are given in Chapter 10.

Figure 8.8
Mechanical paddle flocculator

8.4.6 FILTERS AS FLOCCULATORS

Flocculation takes place when microflocs collide with and penetrate filter media. It occurs as a result of collisions with the solid surfaces of the media particles in the fixed bed. Power dissipation (P) in a filter is given by the equation:

$$P/V = \rho\, g\, h\, v\ /\ p\, l \ \ldots\ldots\ldots\ldots\ 8.13$$

where:

v = velocity of approach
l = filter bed depth
p = porosity of filter bed

Velocity gradient can therefore readily be determined for a rapid gravity filter, from Equations 8.5 and 8.12.

A unit in which this means of flocculation can be exploited is a contact filter. Flow takes place upwards through this filter, the water having received only chemical dosing. This concept has recently re-emerged in the form of the pebble matrix filter (Ives, 1990). This type of filter usually operates without chemicals, for highly turbid waters (20 000 units).

Another type of unit which operates successfully as a flocculator is the contact filter.

Figure 8.9 gives an illustration of a contact filter. The filter consists of a media bed of some 2,5 to 3 m depth, the upper 1 m being sand of 0.6 mm to 0.3 mm in size.

Raw water is fed into the bottom of the unit, together with coagulant. The coarse media at the bottom provide substantial collision opportunity. The successive finer layers provide more uniform flow and act as flocculator/filter. A disadvantage of the unit is upward flow backwashing.

Contact filters are effective for turbidities up to 250 units. They have been in use, for water purification, in Russia and again receive attention.

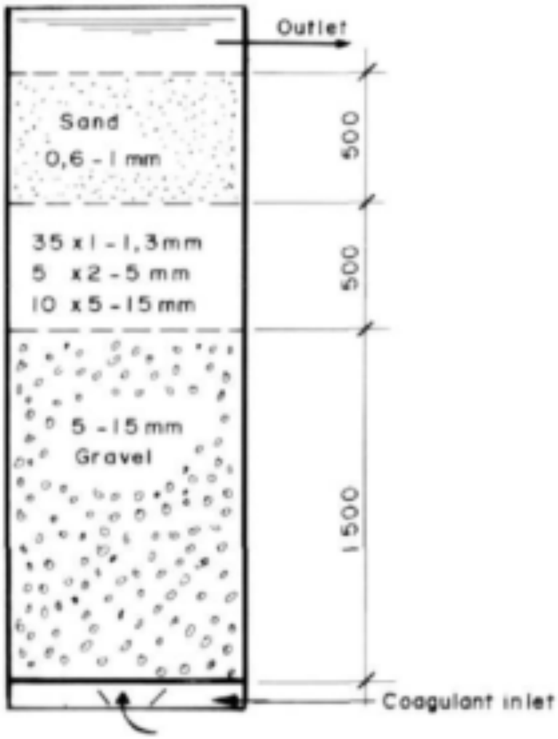


Figure 8.9
Contact filter

8.4.7 AIR DIFFUSERS AND INDUCERS

Aerators can be used for flocculation in such a fashion that circulation and aeration or only aeration takes place. This is shown in Figure 8.6. Attachment of minute air bubbles onto floc particles in a dissolved air flotation process is also a kind of contact flocculation with gas and solids. Universal equations which can describe the attachment process of dissolved air bubbles on flocs from low to high concentration were proposed and solved analytically (Tambo *et al.*, 1990).

In terms of energy input into an aerated tank:

$$kW = \frac{Q_a h}{100V} \quad \text{.........} \quad 8.14$$

where

Q_a = free air discharge, ℓ/s
h = depth of water, m
V = tank volume, m^3.

Aerators have been used in the past in water clarification, but buoyancy is undesirable if sedimentation is the succeeding process. Flocculation followed by flotation is, however, a logical unit operation sequence.

Tapered flocculation can readily be achieved by decreasing the airflow along the flow path of the water in the tank; the number or spacing of diffusers or inducers can be decided as necessary. Velocity gradient can readily be controlled by adjustment of air flow.

8.5 DESIGN CRITERIA

Design of a flocculator can be based on velocity gradient and the related power consumption, retention time and solids concentration. Numerous parameters, however, play a role in effective flocculation.

8.5.1 Velocity gradient

Tapered flocculation G-values of 70 to 20 s^{-1}, and Gt values of 23 000 to 210 000 were recommended for good American practice (Camp, 1953). This is also reflected in present European and American practice (Polasek, 1979; 1992).

In general, the value of G should be greater than 10 s^{-1} in order to promote flocculation but less than 75 s^{-1} if disintegration of floc by shear is to be avoided. Generally, the optimum value of G for flocculation lies between 30 and 60 s^{-1}.

8.5.2 Power consumption

Velocities of flow for baffled tanks range from 0,1 to 1,0 m/s. Head loss at each change of direction is 2 to 3,5 mm, amounting to a total of 0,3 to 1.0 m over the length of a tank. The power consumption is 0,11 kW per Mℓ/d per m of head loss.

Power consumption in flocculators is in general of the order of 0,5 to 1,4 kWh per Mℓ/d of water treated by mechanical means.

8.5.3 Retention time

The value of the dimensionless product of G and time lies between 10^4 and 10^5. Retention times of 10 to 30 minutes should be adequate in channel flocculators; longer retention would imply deficiency in control of floc breakdown.

The retention time in an effective flocculator was taken to be 10 to 60 minutes in the past. Much shorter periods (up to 10 minutes) are now used in high-rate flocculators.

8.5.4 G T C

The kinetics of the process of random flocculation was calculated on the basis of the rate constant, G C°, and the attainment factor, G T C° on a population balance model (Tambo and Watanabe, 1979b). Their process evaluation diagrams included a consideration of the exponent k of the floc density function and the maximum attainable size of floc.

For filtration, the optimal range of G T C was found to be 0,5 to 1 (Van Duuren, 1967). A value of 1 has been recommended for flocculation resulting in 10^{-2} to 10^{-1} mm of micro-flocs in a flash mixer (Tambo, 1989).

For sedimentation of suspensions with clay particle concentrations of up to 500 mg/ℓ and alum dosages of up to 50 mg/ℓ, optimum G is between 10 and 30 s, corresponding to a Gt of 12 000 to 36 000 and optimum G T C_0 of 1 to 6.

9

Sedimentation

F A van Duuren

It is essential in water purification to effect phase separation. For most surface waters this is done after colloidal particles have been aggregated. A first stage phase separation can then be effected by means of sedimentation or clarification. Sedimentation is by definition a discrete, primary mechanism whereas clarification, as dealt with in Chapter 10, consists of a number of mechanisms.

Sedimentation as a unit operation, belongs to the oldest methods of water purification. It is very robust, little controllable and medium efficient. Knowledge of this operation has been passed on through all ages; this explains why sedimentation is also one of the most ubiquitous unit operations in water technology (Hahn, 1990).

The two major aspects involved in sedimentation are the properties of the particles in suspension and the hydraulics of or flow patterns within the unit operation. For any given particle properties, phase separation will best be effected in a unit having ideal hydraulic characteristics. The properties of the particles and the flow dispersion characteristics of the unit therefore have to be considered individually.

The usual approach has been to view particles to be removed to be discrete and hence the accent in the past, on the laws of sedimentation. The point of departure for sedimentation tank conceptualisation has long been that of a quiescent state of the water. This is essential to the theory of the ideal tank which in turn gave rise to the concept of the four zones in a "horizontal flow" tank.

Dimensional analysis of surface loading indicates that it is actually velocity. Consideration of flow direction leads to the conclusion that it is in effect vertical i.e. upwards for the water and downwards for the particles. Weir overflow rate is another parameter which is used in sedimentation tank design. The use of weir overflow rate should not be allowed to obviate consideration of the need for a good flow pattern in a sedimentation tank.

There has been substantial progress in the knowledge and understanding of coagulation and flocculation and thus solids contact clarification. Whereas sedimentation tanks operate generally at an upflow rate not exceeding 2 m/h, solids contact and floc blanket clarifiers have been evolved with upflow rates up to 20 m/h; the corresponding retention times are 8 h and 0,8 h respectively. This disparity clearly has substantial spatial implications, especially in the case of large flows.

Sedimentation is viewed here in terms of sedimentation theory, shape and configuration of tanks, types of tank, suspension characteristics, hydraulics and flow patterns, sedimentation velocities, overflow rates, sedimentation tank efficiency and desludging. Design criteria and examples are also given.

9.1 SEDIMENTATION THEORY

The classical Stokes equation still has the most significance in sedimentation theory. This is so despite the fact that the Stokes equation for sedimentation velocity of a particle in a fluid is applicable to laminar flow conditions. It is however, also used extensively as a component of other equations. Derived from first principles (Van Duuren, 1962) Stokes settling velocity v_s is given by the equation:

$$v_s = \frac{g\,(\rho_s - \rho)\,d^2}{18\,\mu} \quad \text{.......... } 9.1$$

where:

g = gravity constant
μ = viscosity of water
ρ = density of water
ρ_s = density of solid
d = diameter of solid

This equation is valid for a Reynolds number $(\rho\, v_s\, d\,/\mu)$ of 10^{-4} to 0,55 and for particle size $d \leq 0{,}085$ mm.

Allen's equation

$$v = 1{,}21\,[(\rho_s - \rho)/\rho \,.\, g/k]^{2/3}\,[d/(\mu/\rho)^{1/3}] \text{ .....} 9.2$$

is applicable for Reynold's number over the range 0,55 to 1000.

Thomas' equation was evolved for a wider range of Reynolds number; in this equation:

$$v = v_s\,/[1 + 0{,}17(\,v_s\,\rho\, d/\mu)^{1/2}] \text{ .... } 9.3$$

where:

v_s = Stokes velocity of sedimentation

This equation has applicability over a wide range of particle sizes and for Reynolds number of 10^{-4} to 10^{3}.

For practical purposes and from empirical considerations, the upflow velocity for a sedimentation tank can be determined from the equation:

$$Q = A\,v \text{ .... } 9.4$$

Because velocity is a vertical vector, A is the plan area. It should again be noted that while the particles settle downwards, the clarified water flows upwards.

If flow in a rectangular basin is assumed to be laminar, the Reynolds number R_e should be less than 900 (Brater & King, 1976). However, for a tank which is 12 m wide and 3,9 m deep the Reynolds number, for water at 10°C, is in fact 28 283 and flow is turbulent (Greenblau, 1989). These are dimensions of a small tank and leads to the conclusion that most basins will operate under turbulent flow conditions. Stokes' equation is nevertheless a useful guide to sedimentation velocity.

9.2 TANK SHAPE AND CONFIGURATION

Tank shape and internal configuration play an important role in the effectiveness of sedimentation. Sedimentation tanks are rectangular, square or circular in plan. All tanks can be seen to have four operational zones, i.e. inlet, sedimentation, outlet and sludge.

Particular attention must be given to the inlet system, the sedimentation zone, the sludge zone and system and the outlet system. The shape of the tank would have to accommodate and facilitate these operations and zones.

The most usual shape of sedimentation tank is rectangular and the nominal flow direction is horizontal. The rectangular, horizontal flow tank has long been the most extensively used and investigated. Square and circular tanks have vertical or radial flow and are of particular interest in that respect. As will be seen further on, radial flow tanks have poor flow dispersion characteristics.

Whatever the shape of tank, the inlet, sedimentation, outlet and sludge zones will largely determine its hydraulic efficiency and thus its effectiveness. The size of a sedimentation tank plays a role to the extent that temperature, density currents and wind effects prevail. The larger a tank the more prone it will be to these influences. Of course, these effects can be obviated by enclosing the tank; size usually militates against this; the advantage of visual observation is then also lost.

9.2.1 INLET

Flow from a pipe into a tank must be spread effectively, normal to the cross sectional area (csa) of the flow direction. The inlet to a sedimentation tank is therefore usually some type of baffle. Channels were used in the earliest tanks until it was realised that this spreads flow over a line, close to the water surface and not over the csa. A perforated honeycomb type of inlet can be used to improve dispersion and thus sedimentation.

9.2.2 SEDIMENTATION ZONE

The sedimentation zone can hardly be affected other than by the nature of the inlet and outlet. The sedimentation zone has often been neglected or misunderstood to the extent that dead areas are readily perceived in sedimentation tanks; dead areas imply short circuiting and thus poor effluent quality.

Baffles should only be inserted in a sedimentation tank after experimentation with models in order to obtain an indication of the resulting effect on flow patterns. It was demonstrated (Hale, 1970) that laboratory modelling of a sedimentation tank can be done effectively. It might then well become clear that no amount of baffling can correct a problematical flow pattern such as prevails in a pyramidal tank with bottom inlet and peripheral weir outlet.

9.2.3 OUTLET

Outlet systems have progressively been given more attention by design engineers to the extent that it lends itself to improvements which can easily be made. Provision of an evenly distributed surface outlet system will improve the flow pattern and therefore sedimentation. It is preferable to have two or more outlet channels to a horizontal flow tank and thus to reduce overflow rate.

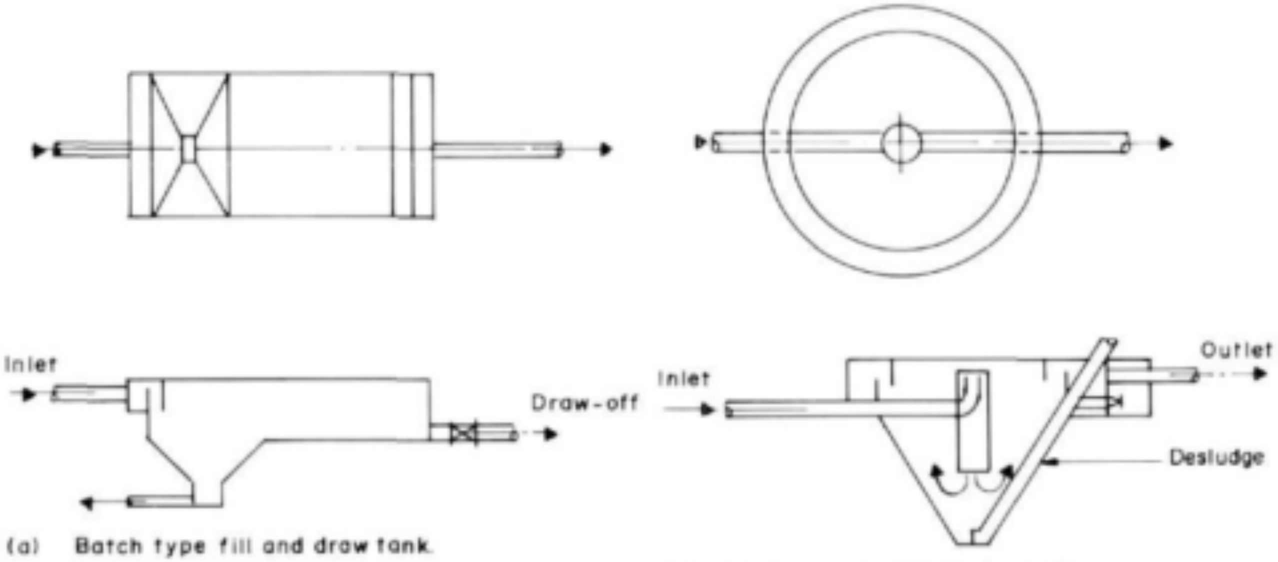
Because of its known properties the provision of the shallow bed system, based on the Hukki principle, will improve the flow pattern at the outlet and thus quality of clarified water.

9.2.4 SLUDGE ZONE

The sludge zone of sedimentation tanks is problematical, especially in large rectangular tanks; extensive hopper bottoms are usually impractical. This problem can be overcome by the introduction of mechanical/suction type of devices, such as used by Rand Water. Sludge is less of a problem in circular tanks because of smaller sizes and the consequent ease of inclusion of mechanical scraper devices.

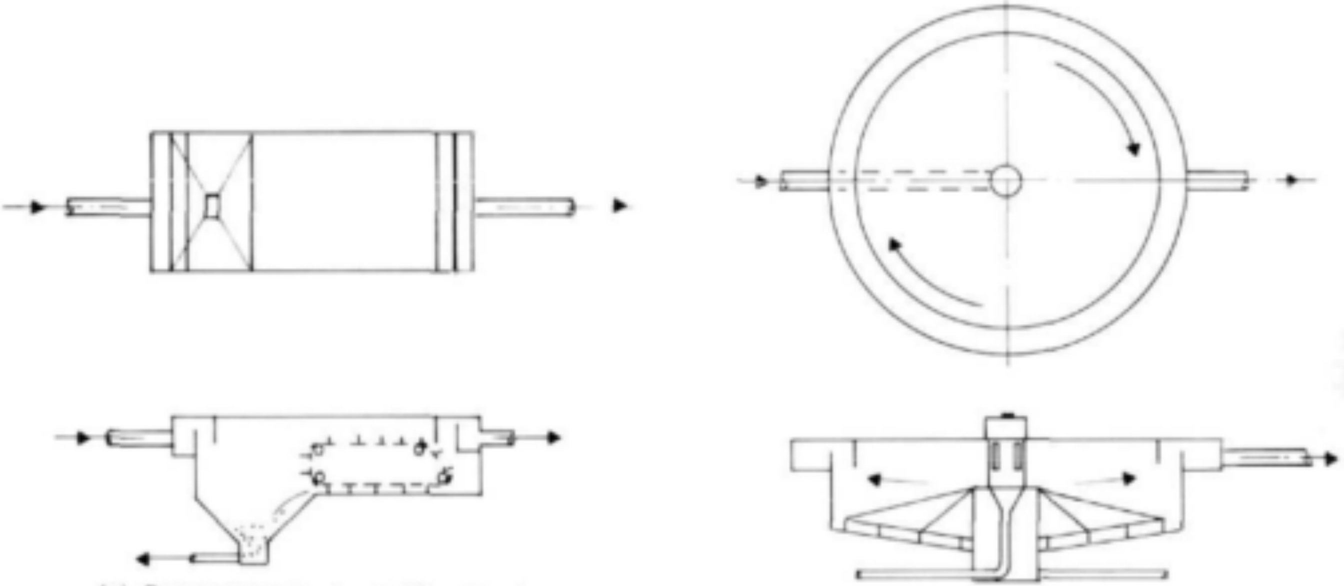
9.3 TYPES OF TANK

There are numerous types of sedimentation tank because designers are innovative and there is scope in this respect. A number of basic shapes and type of tank is shown in Figure 9.1. The aspects reflected include shape, nature of flow inlet, outlet, collectors and desludging systems. The various tanks are described and a generalised summary is given of rectangular and vertical flow tanks.



(a) Batch type fill and draw tank.

(b) Conical tank with hydrostatic sludge removal.



(c) Rectangular (horizontal flow) tank with mechanical desludging

(d) Circular (radial flow) tank with scraper mechanism

Figure 9.1
Typical sedimentation tanks

9.3.1 BATCH TYPE TANK

The batch type of tank, shown in Figure 9.1 (a) is still used in small water works. The tank is filled with the raw water to be clarified, coagulant is added and stirred before sediment is allowed to settle out for a day or more. The clarified water is then drawn off as needed. This method of clarification was used by the Chinese over some thousands

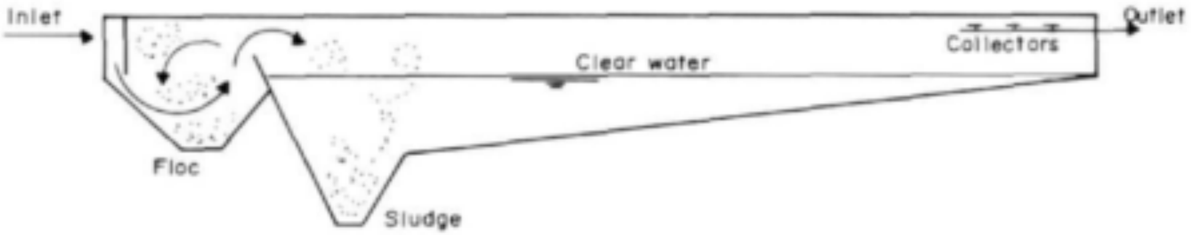


Figure 9.2
Sedimentation tank with flocculator

of years; they used bamboo loaded with clay (containing aluminium salts) to thrash the water before allowing the floc formed to settle out.

9.3.2 CONICAL TANK

The Dortmund type, conical tank shown in Figure 9.1 (b) operates on a continuous basis. The flow pattern in the tank shown, can be improved by using a radial pipe outlet system which would facilitate more uniform collection of clarified water from the surface. These units are also used on smaller water works.

9.3.3 RECTANGULAR TANKS

Rectangular tanks such as shown in Figure 9.1 (c) are used on larger works. They should be designed with proper attention to inlet, outlet and sludge systems in order to improve the flow pattern and desludging.

Rectangular tanks are inherently problematical for sedimentation because effective flow is not horizontal. These tanks also do not have a plug flow pattern and are prone to short circuiting. Numerous attempts have been made to overcome the problems of rectangular tanks. The ideal (rectangular) tank theory has gone some way in solving the inherent problems of inlet, sedimentation, outlet and sludge removal.

A spreading inlet flow would aid sedimentation to the extent that it promotes plug flow. Reducing outlet velocity also improves hydraulics by limiting short circuiting. Multiple outlet channels were introduced by Rand Water some years ago, as an improvement to their rectangular tanks. A rectangular sedimentation tank incorporating a flocculation zone is shown in Figure 9.2.

Sludge withdrawal systems to rectangular tanks require particular care and attention. Rand Water improved their system by the provision of a sludge pump system mounted on a travelling bridge.

Hopper bottoms are used for small tanks, but sludge removal becomes problematical for very large tanks unless some form of scraper or pump system is used. The slopes of hoppers must be adequate for sludge flow to collector points. Very large rectangular tanks are prone to decreased efficiency due to exposure to wind and weather.

Long narrow basins are preferable to square basins; a length to breadth ratio of more than 3:1 is desirable. A small width facilitates economy of sludge hoppers (Langenegger, 1993).

9.3.4 CIRCULAR TANK

A cylindrical, radial flow tank is shown in Figure 9.1(d). The inlet system has provision for the addition of coagulant and flocculation in a central cylindrical compartment. Because a scraper mechanism is provided, desludging can be achieved even for a large tank. However, the radial flow pattern limits the flow through this type of tank.

9.3.5 VERTICAL FLOW TANKS

Debate on square vs. circular tanks which ignores the depth dimension is deficient in its consideration of flow patterns. Whether circular or square in plan, vertical flow tanks usually have higher flow rates and thus shorter retention times than horizontal flow tanks. The advantage that vertical flow tanks have results in smaller units and in greater economy.

By compartmentalising these tanks, but retaining uninterrupted flow, the zones can be given specific attention. It is also then possible to incorporate discrete flocculation, sedimentation and sludge zones in a single tank, thus obviating unwanted flow acceleration and turbulence. By keeping the units small, wind and weather effects are also limited or can be eliminated by placing the tanks indoors.

9.3.6 TUBE OR PLATE SEDIMENTATION

This system utilises the Hukki i.e. shallow bed or inclined plate principle of minimal depth for particles to settle. Several configurations of tube shapes and sizes are possible, with most designs employing upflow configurations (AWWA, 1990). A typical American design is 50 x 50 mm tubes inclined at 60° from horizontal. The modules are constructed of PVC and other plastics.

The laminar flow characteristics and clarification results are the same for tubes and inclined plates. The influent flow moves up through the tubes and the sludge move down. Tube modules are usually installed over 60 to 75% of the surface area of the basin. This provides an open area at the tank influent end for removal of settleable floc particles by conventional sedimentation and for stabilisation of flow patterns.

Because of superior hydraulic stability, application rates for horizontal tube surface can be expected to be two to three times the rate applied to sedimentation tanks. However, inclined plate equipment typically requires deeper, costlier tanks (AWWA, 1990).

9.4 SUSPENSION CHARACTERISTICS

Under South African conditions, the greatest demand for purified water occurs during the hot summer months, when most of the country is subjected to thunder storms resulting in turbid raw waters. It is therefore important to obtain a sample of water that represents not average conditions, but the worst that will pertain to a particular design. The methods of extraction from the raw water source should then also be taken into consideration, e.g. scouring action around a river intake, resulting in extraction of clay, silt and sand.

A suspension entering a sedimentation tank will generally have a density different from that of the contents of the tank. Apart from having particles in suspension and by that token an inherent density, the suspension might well be at a different temperature from that of the contents of a tank. If the suspension is colder, the inlet stream would incline downwards because of higher density; if it is warmer the flow would incline upwards. The implications of these considerations for a poorly designed horizontal flow tank, is clear.

It can readily be shown mathematically, that the removal of particles from a suspension in a horizontal flow tank is independent of the depth of the tank. Axiomatically, for zero depth, particles would require no time for sedimentation. In practice, however, some depth of tank is required - most importantly to avoid disturbance and entraining of settled sludge. This aspect of sedimentation tanks was demonstrated experimentally (Lumley & Balmer, 1990).

The densities and (Stokes) sedimentation velocities of discrete particles, iron (Fe) and aluminium (Al) floc and other particulates is given in Table 9.1 (Masschelein, 1992).

TABLE 9.1
DENSITIES AND SEDIMENTATION VELOCITIES OF PARTICLES

Particle	Wet density (kg/dm^3)	Settling velocity (mm/s)
Activated carbon powder	1,2 to 1,4	2
Al and Fe floc: Turbidity	1,005	0,7 to 1,3
Colour	1,004	
Algae	1,002	0,6 to 0,9
Crystalline calcium carbonate	1,2	2 to 3
Organic waste (polyelectrolyte)	1,001	0,3 to 0,5
Softening precipitate	1,15	1,2 to 1,9
Bentonite	2,0 to 2,2	1 to 5
Sand	2,65	5 to 10

9.5 HYDRAULICS

Because of its significance to sedimentation, the flow in various tanks needs to be characterised. This has been done in general terms in Chapter 7. A closer view is necessary of flow of a suspension and phase separation in sedimentation tanks. The actual flow in a rectangular tank is illustrated in Figure 9.3.

Specific gravity of a suspension flowing through a horizontal flow tank changes as suspended solids fall downwards. Streamline flow can therefore be achieved vertically upwards but not horizontally. The effective upward velocity is least for a given flow rate when the flow is vertically upwards over the whole area of a settler because there are no dead areas and short circuiting is obviated.

Consideration of flow and sedimentation in Figure 9.3 indicates that optimal sedimentation is achieved when the flow direction of the suspension is vertically upwards. This is further illustrated by the nature and description of the flow dispersion curves shown in Figure 9.4.

The long narrow channel indicated in Figure 9.4 would result in plug flow, but this configuration is rarely used in water works sedimentation tanks or clarifiers.

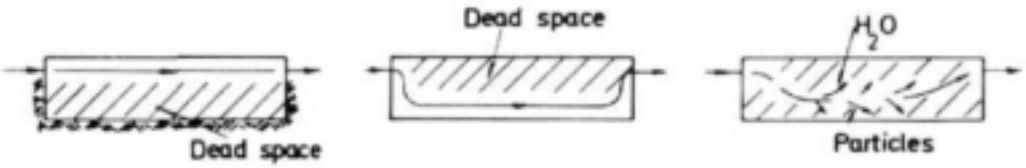
The intractable problems of flow dispersion in horizontal flow tanks has repeatedly been exemplified by observation and the findings of research. In a report on such research it was stated: "When modelling the behaviour of secondary settlers, consideration must be given to short-circuiting effects in the inlet zone, the density current and its effect on classification of the mixed liquor, the settling and thickening characteristics of the sludge and the transport mechanisms in the sludge blanket such as density driven diffusion and the sludge flow patterns" (Lumley & Balmer, 1990).

Seven transport zones in horizontal flow secondary settlers were described by Lumley and Balmer. The effects of short-circuiting in the inlet zone and the creation of a density current were found to be important for determining behaviour of the settler.

Dispersion of flow can be determined experimentally by means of a number. This number is:

$$N_f = D_a / v \cdot L \quad \text{..........} \quad 9.5$$

Figure 9.3
Longitudinal section through a rectangular tank showing flow of suspension, particles and water



(a) Incoming flow warmer than tank contents

(b) Incoming flow colder than tank contents

(c) Incoming flow seeking its own density level

where:

D_a = axial mixing coefficient, m^2/h
v = average flow velocity, m/h
L = tank length

For ideal sedimentation i.e. plug flow, $D_a = 0$; for an ideal mixer D_a = infinity.

Flow dispersion is axiomatically the ratio of theoretical time (t_s) to the actual time (t) taken, for a particle to settle, i.e.

$$\text{Dispersion} = t_s / t \quad \text{..........9.6}$$

9.6 SEDIMENTATION VELOCITIES

The design of sedimentation tanks is still mainly based on surface loading i.e. upflow velocity and weir overflow rate. Retention time is still used as a design parameter but some design engineers do not use it to rate sedimentation basins (Langenegger, 1993).

The Blackheath water works of Cape Town operates (Greenblau, 1990) at a surface loading of 43,98 $m^3/d.m^2$ i.e. an upflow velocity of 1,83 m/h. The Voëlvlei works operates at 1,93 m/h and the Steenbras works at 2,8 m/h.

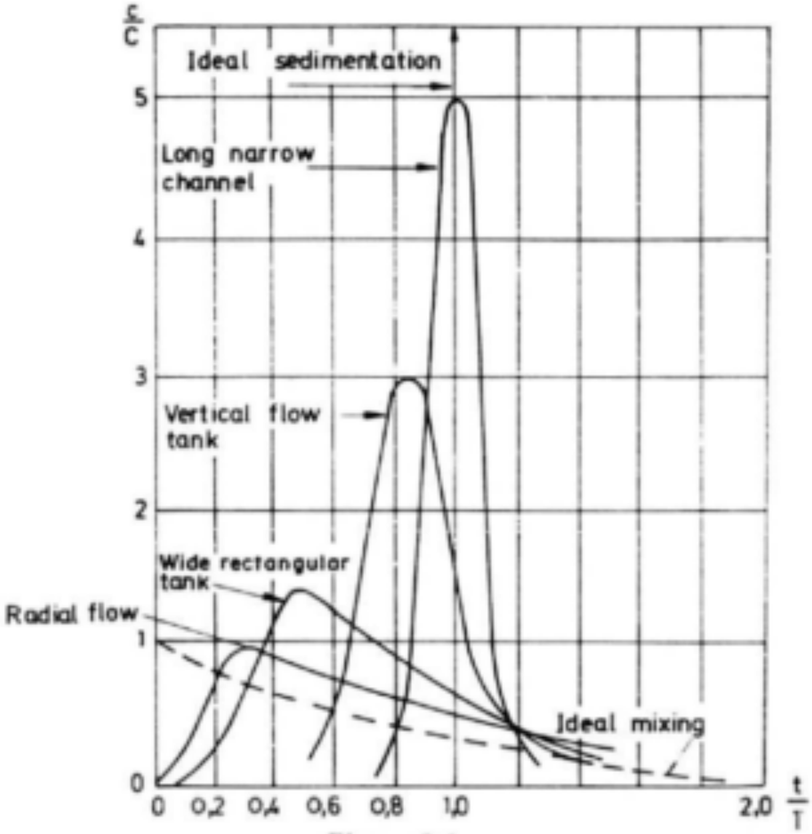


Figure 9.4
Flow dispersion curves for tanks

It is appropriate (Greenblau) to design Cape Town water works on a surface loading of 45 $m^3/d.m^2$ as long as the average horizontal flow is less than 2 cm/s.

As indicated in Table 9.1, sedimentation velocities of particles cover a wide range. The Stokes velocity of a particle is applicable to quiescent conditions; this will not be the case in a flow. Moreover, there will also be collisions and possibly even aggregation. However, it should be helpful to know some benchmark velocities.

Sand, silt and clay particles have a specific gravity of about 2,6 and grain size of about 10^{-2} mm diameter. The sedimentation velocity of this size of particle at 10°C, is upwards of 0,25 m/h and a detention period of 12 h would be required in a 3 m deep upflow tank.

Aluminium and iron flocs possess specific gravities of about 1,002 and have a diameter of about 1 mm. Their settling velocity at 10°C is about 3 m/h and it would theoretically require an hour of retention in a 3 m deep tank for these flocs to settle. However, depth should not be the criterion for this purpose.

Calcium carbonate crystals have a specific gravity of 1,2, and a diameter of about 10^{-1} mm. Their settling velocity is about 1,5 m/h and it would require a theoretical retention of two hours for these to settle in a 3 m deep tank.

Upflow velocities for horizontal flow tanks were stated to be 1,2 to 5,5 m/h in Britain; in the USA the highest upflow velocity was stated to be 2,5 m/h; works investigations indicated upflow velocities in South Africa to be 0,6 to 1,8 m/h (Van Duuren, 1962).

Upflow velocities (m/h) were put forward for sedimentation on the basis of conceptual analysis and practical experience (Hahn, 1990). These have been added to from Belgian experience (Masschellein, 1992) and from South African investigations and experience (Van Duuren, 1962; Greenblau, 1990); these are reflected in Table 9.2:

TABLE 9.2
SEDIMENTATION TANK VELOCITIES (m/h)

	Germany	USA	Belgium	RSA
Pretreatment tanks	0,8 - 1,5 2,5 - 4,0	1,4 - 1,7		1,0 - 8,0
Primary sedimentation	1,0 - 1,5 0,5 - 1,5	0,9 - 12 1,4 - 2,0	1,8 - 5,4	1,8 - 2,8
Secondary sedimentation	0,8 - 1,5	0,8 - 1,6		

9.7 WEIR OVERFLOW RATES

It will be clear from the foregoing that flow patterns and thus the effectiveness of sedimentation are affected by the outlet system and thus weir overflow rate. By concentrating outflow along one side of a single outlet channel to a tank, the upward velocity is made very high and those particles still in suspension will easily be entrained in the effluent. This can readily be seen for density currents in, or for wind effects on a sedimentation tank.

In order to limit the effect of currents and short-circuiting, careful attention must also be given to outlet design. Provision of multiple channels is relatively inexpensive compared to the rest of a tank. Care should be taken to limit surges; this can be done by means of serrated weirs or orifices in the case of an effluent pipe system.

Weir overflow rates of less than 9 m^3/h.m were put forward in design criteria (Van Duuren, 1962). It was suggested (Greenblau, 1990) that less than 6,25 m^3/h.m is necessary to avoid surges at the outlet zone. Rates of less than 10 $m^3/h.m$ are used in the USA (AWWA, 1990).

Overflow rates of 1 to 1,5 $m^3/h.m$ of weir are used in secondary sedimentation units (Langenegger, 1993); the low number can be ascribed to small size of particles at this stage of phase separation.

9.8 DESLUDGING

Because of the large size of sedimentation tanks especially of rectangular shape, desludging requires careful design. In the case of circular tanks the problem is usually solved by having mechanical-driven scraper mechanisms; these mechanisms can also be used for rectangular tanks.

Sludge transport in the sludge zone of a rectangular tank is predominantly horizontal near the outlet end and becomes increasingly vertical as the inlet end is neared. The implication is that maximum sludge deposition takes place near the inlet to a tank; this may well be taken into consideration in detail design.

Extensive use is made of hoppers in square and rectangular tanks. The problem of withdrawing sludge from these units calls for ingenuity. Rand Water developed an effective sludge pump system for their rectangular sedimentation tanks. The sludge pumps are mounted on a bridge which travels the length of the tank and discharges into sludge thickeners, treatment and disposal units.

Hoppers allow sludge to be thickened to concentrations of up to 5 to 6%. Adequate sludge drain velocities are also required (>1,4 m/s). Floor slopes need to be 1:12 for the outer half of basins and 1:6 for the inner half (AWWA, 1990).

9.9 SEDIMENTATION EFFICIENCY

Sedimentation was originally effected by allowing turbid water to stand in a basin for some period of time. It was then found that coagulation, with chemicals, and subsequent flocculation with mechanical devices, resulted in more rapid clarification. These processes and operations were therefore increasingly used, preceding sedimentation. A process which has also come into more general use is chemical treatment in the sedimentation tank.

For a suspension with particles having a range of sizes and thus settling velocities, and for an ideal tank, the actual velocity of a particle is given by Equation 9.4 as $v_a = Q/A$. From Figure 9.5 the removal of particles is given by the equation:

$$\text{Removal} = (1\ p_a) + 1/v_a\, Q \int_0^{p_a} v\, dp$$

The cumulative concentration (p) vs. velocity curve can be obtained by laboratory analysis with an Andreasen pipette apparatus.

In terms of the removal of particulates, the efficiency of sedimentation tanks varies from 50% to 90% as shown in Figure 9.6.

Conventional, rectangular horizontal flow sedimentation tanks were almost universally used until 35 years ago (Van Duuren, 1962). In the past it was simple to compare tanks and determine efficiency based on shape of tank. The event of chemical treatment complicated efficiency concepts to the extent that the subsequent phase separation stage i.e. filtration was affected.

Water clarified with activated silica and lime did not yield improved filter runs. It was therefore put forward that turbidity might not be a suitable index for measuring sedimentation tank performance and that a filtrability index of water should be devel-

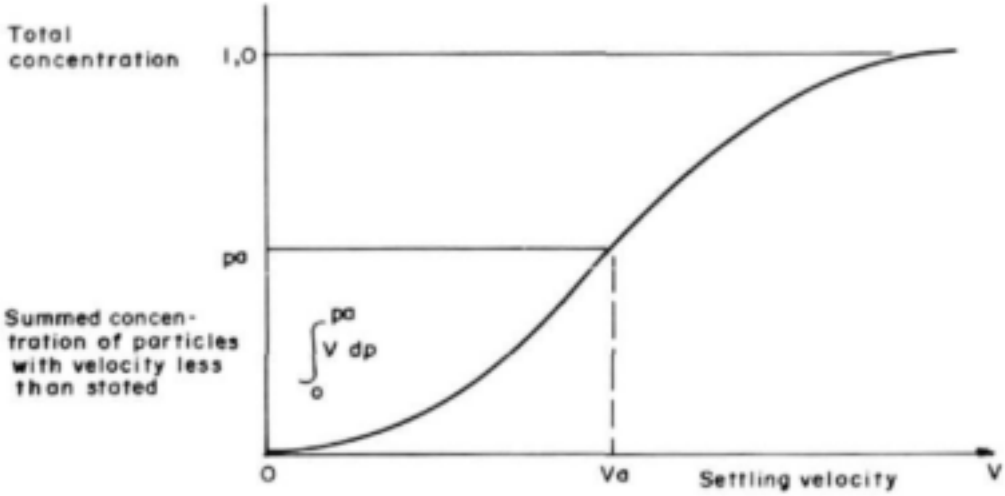


Figure 9.5
Distribution of sedimentation velocities

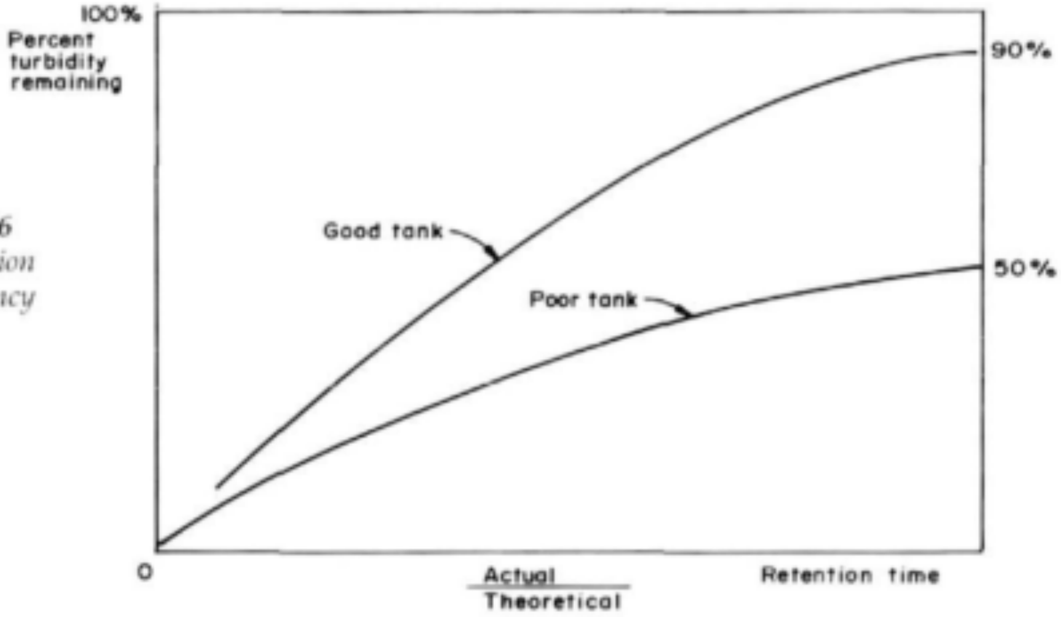


Figure 9.6
Sedimentation tank efficiency

oped. This was given attention - with some success.

By definition, the objective of water purification is usually the removal or diminution of turbidity, colour, hardness and sometimes iron, manganese or other undesirable minerals. Apart from being a measure of the physical removal of particles in suspension, turbidity is also a measure of the presence or absence of bacteria, amoeba and cysts. Colour is also indicative of organic matter and algae.

Turbidity and colour measurement will, for the reasons given and because of the simplicity of determination, remain the most important parameters of the efficiency of sedimentation and also clarification. A turbidity of less than 5 units is attainable for sedimentation tanks.

9.10 DESIGN CRITERIA

An array of design criteria are given here to assist the design engineer. These criteria are based on the literature cited and especially investigations, research and experience. However, the particular conditions that prevail for a given water source and the treatment applied to it will determine the applicability of any particular criterion.

9.11 EXAMPLES

Design examples are given to illustrate the different approaches to design of horizontal flow (rectangular) tanks.

Example 9.1

A rectangular sedimentation tank is required to clarify a flow of 15 Mℓ/d of turbid water. Aluminium sulphate is used as a coagulant, ahead of flocculation, resulting in 0,5 mm flocs of density 1006 kg/m³. At 20°C the raw water can be assumed to have a density of 998 kg/m³ and a viscosity of 10^{-3} kg/m.s.

Determine the theoretical sedimentation velocity of the particles, the area of the tank(s) required and the upflow velocity, taking into account that the flow dispersion is as shown in Figure 9.3.

The theoretical sedimentation velocity of the particles, calculated from Stokes' equation (9.1) is:

$$v_s = \frac{9{,}81\ (1006 - 998)\ (0{,}5 \times 10^{-3})^2}{18 \times 10^{-3}}$$

$$= 1{,}09 \times 10^{-3}\ \text{m/s}$$
$$= 3{,}92\ \text{m/h}$$

Also, theoretically

$$v_s = \text{depth of tank} / t_s$$

Thus depth = $v_s \times t_s$

For flow dispersion of a rectangular tank read as 0,4

$$t_s / t = 0{,}4$$

Now, Q = volume of tank/theoretical retention time (t)

thus $t = A \times \text{depth} / Q$

i.e. $t = A \times v_s \times t_s / Q$

or

$$A = \frac{t \times Q}{t_s \times v_s}$$

$$= \frac{15 \times 10^3}{0{,}4 \times 1{,}09 \times 10^{-3} \times 24 \times 3600}$$

i.e.

Tank area = 398 m² and actual upflow
velocity = Q / A
= $15 \times 10^3 / 10^{-3} \times 24 \times 398$
= 1,57 m/h

Example 9.2

The clay colloids content of a water to be purified is high, as turbidity usually exceeds 250 units and colour is < 50 units. The water is effectively coagulated and flocculated before it reaches the sedimentation tanks.

From the criteria in Table 9.3, design the necessary rectangular sedimentation tanks to clarify 15 Mℓ/d of water.

From Equation 9.3 and assuming 1,2 m/h upflow velocity, the area (A) of tank required is:

$$A = Q / v$$

$$= \frac{15\,000}{24 \times 1{,}2}$$

$$= 520{,}8\ \text{m}^2$$

TABLE 9.3
DESIGN CRITERIA FOR WATER WORKS SEDIMENTATION TANKS

Characteristic	Criterion and description
Inlet velocity	< 1 m/s
Upflow velocity	0,5 to 1,5 m/h for rectangular tanks 1 to 3 m/h for vertical flow tanks
Longitudinal velocity	< 72 m/h for rectangular tanks
Weir overflow rate	< 10 m^3/h.m
Retention time	> 2 h for vertical flow tanks > 8 h for horizontal flow tanks
Solids loading	1 to 1,3 kg/m^2.d
Number of tanks	> 2
Length	not > 100 m
Length to width ratio	> 3
Side wall depth < 5 m for rectangular tanks	< 3 m for vertical flow
Floor slope: with hydrostatic desludging with mechanical desludging	60 for rectangular tanks 1 in 12 to vertical flow tanks 1 in 100 for rectangular tanks
Sludge drain: line diameter flow velocity	 > 150 mm diameter $> 1,4$ m/s
Desludging water loss	$< 3\%$

According to the criteria, two units are required. Hence the area of each tank can be set at 260 m^2.

For a length to width ratio of 3:

$$A = 3 \times \text{width}^2$$
$$= 260$$

i.e. width = 9,3, say 9 m
and length = 28,9 say 29 m.

Using a nominal depth of 5 m, the volume of the tanks is 1 305 m^3 and nominal retention time would be 1 305/ 15 000 = 0,087 day, i.e. 2,088 hours.

For an assumed weir overflow rate of 7,5 m^3/h.m the length of weirs required for the given flow is:

$$\text{Weir length} = \frac{15\ 000}{24 \times 7{,}5}$$

$$= 83{,}3 \text{ m}$$

Using collection channels across the width, at the outlet end, each channel would have 18 m of weir and the total number of channels is 83,3 /18 i.e. 4,65. Using four channels would give a weir overflow rate of 8,74 m^3/h.m , which is < 10 and therefore acceptable. Note that three channels only would increase the weir overflow rate to 11,6 m^3/h.m, which exceeds the criterium. The weirs can be serrated by means of V notches for better flow properties and level control.

Desludging requires a detailed knowledge of the solids loading; the necessary data should be obtained. The choice of systems is hopper bottoms, with greatest depth close to the inlet end, a flat bottom with scraper mechanism or a suction pump system.

10

Clarification

F A van Duuren

Primary phase separation of coagulated particles and water can be effected by gravity or by clarification. Gravity separation is done by sedimentation or flotation. Clarification by floc blanket involves sedimentation, flotation, filtration and floc or sludge thickening. All these unit operations occur within the same container - a clarifier.

The description given to clarifiers, in the American Water Works Association publication (AWWA, 1990), masks the development that has taken place over the past thirty years. The heading "solids-contact upflow sedimentation" used, is both too descriptive and yet restrictive.

Recognition of sedimentation and the other mechanisms involved in a clarifier, is evident from the work done in this regard, in Australia (Bond, 1961) and in Europe (Ives, 1967; and Barnes & Wilson, 1983). A floc blanket clarifier can now readily be understood in terms of the unit operations involved.

A typical clarifier configuration is shown in Figure 10.1. Separation of flocculation, floc decanting and sedimentation, clarification and sludge thickening in separate but contiguous compartments, should be evident from the clarifier illustrated.

Subsequent to effective coagulation, ap-

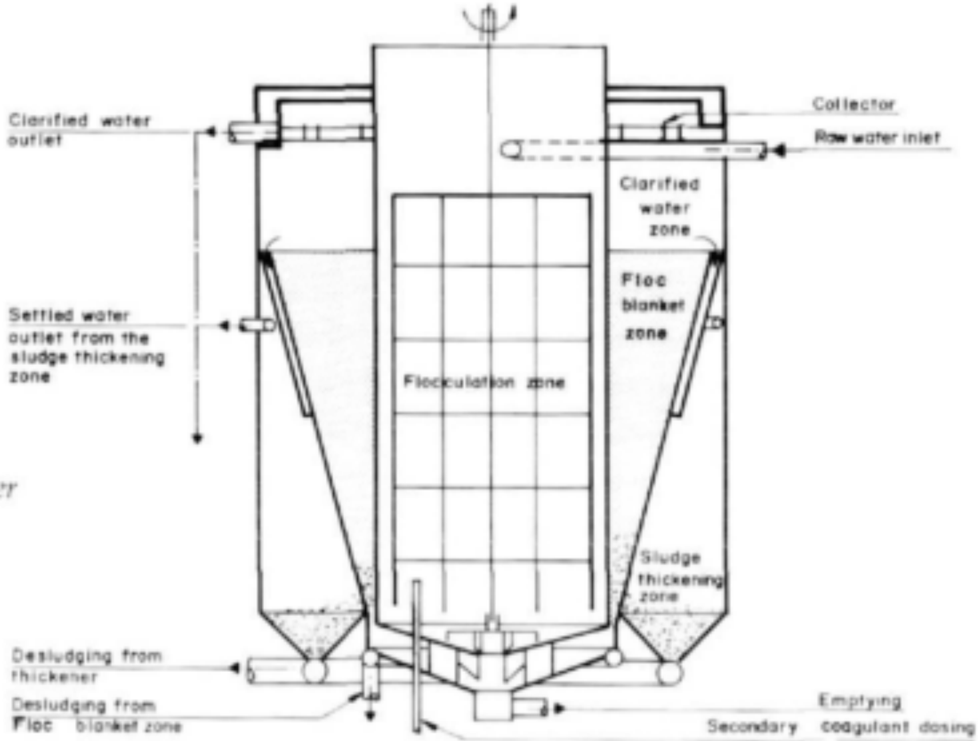
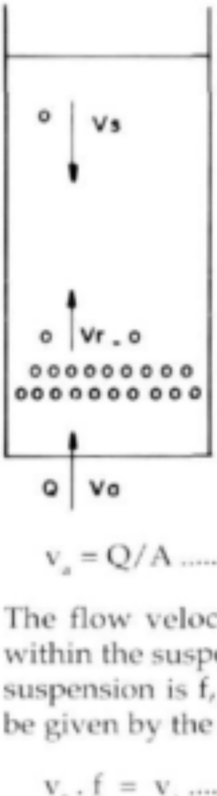


Figure 10.1
High rate clarifier (Bethlehem Water Works)

plication of the known unit operations within a clarifier obviates the need for impractical notions of phase separation. Flow dispersion within the various compartments of a clarifier can be optimised by obviating or limiting short-circuiting.

10.1 THEORY AND OPERATION

An analysis was made of the theory and operation of clarifiers (Bond, 1960; Ives, 1967; Tesarik, 1967). The floc blanket clarifier was analysed in terms of its operation as a flocculator, a filter, a sedimentation unit and a sludge thickener. The results obtained from investigations by a number of workers are also incorporated in the theory presented here.



The diagram shows a simple model composed of a cylindrical tube partially filled with particles of the same size. All the particles have the same relative density which = 1. With no flow of liquid upwards in the tube all the particles would rest on the bottom of the tube.

If a flow Q is now introduced, an approach velocity v_a will exist at the bottom of the tube such that:

$v_a = Q/A$ 10.1

The flow velocity will be greater than v_a within the suspension. If the porosity of the suspension is f, the relative velocity v_r will be given by the equation:

$v_r \cdot f = v_a$ 10.2

If the settling velocity of the particles is v_s then it is clear that the particles in the suspension will be static if $v_r < v_s$.

When v_r is increased, a point will be reached where $v_r = v_s$ and the particles become weightless as the upward drag force becomes equal to gravity. Any further increase in v_r will lift the particles and render the bed fluid. The porosity will increase as a result of the fluidity and the particles will therefore be in equilibrium with $v_r = v_s$ because v_s is constant. Therefore:

$v_r \cdot f = v_s \cdot f = v_a$ 10.3

If S is the volumetric solids concentration fraction in a mixture of solids and liquids and f is the porosity, then:

$f = 1 - S$ 10.4

Replacing this in Equation 10.3:

$v_a = v_s\ (1 - S)$ 10.5

or:

$v_s = v_a / (1 - S) = v_a / f$ 10.6

It is seen from Equation 10.6 that, because v_s is constant, an increase in v_a will result in a corresponding increase in f or a decrease in S. In practice, this means that as the flow rate in a clarifier increases, the floc blanket expands to such an extent that solids are carried over. For a clarifier of cylindrical configuration any flow adjustment therefore has to be made gradually in order to establish a new equilibrium for the resulting porosity.

To allow for the non-sphericity of the solids, Equation 10.6 is modified to:

$v_a = v_s \cdot (1 - mS)$ 10.7

where m is a shape factor. This is similar to the equation (Bond, 1960):

$v_a = v_s \cdot (1 - k\,S^{2/3})$ 10.8

where k is also a shape factor, but the equation is based on the supposition that the particles are layered horizontally.

Experimental results (Tesarik, 1967) fitted both Equations 10.7 and 10.8 in spite of the fact that the one is an arithmetical and the other is an exponential function. The contradiction can be explained by the range of f being 0,72 to 0,90. If the whole range is investigated, the exponential equation gives a better fit.

Fluidisation of uniform particles was investigated (Richardson & Zaki, 1954) and yielded the equation:

$$v_a = v_S \cdot f^n = v_S \cdot (1 - S)^n \quad \text{.......... } 10.9$$

The factor n varies with particle properties, but is unreliable for alum floc and is therefore not used. Most floc particles are gelatinous and contain a large amount of water. It is therefore more correct to consider floc-volume concentration and Bond's equation can therefore be rewritten as:

$$v_a = v_S \; (1 - k\,C^{2/3}) \quad \text{.......... } 10.10$$

where C is floc volume concentration.

For alum and iron floc, k has a value of 2,78 and Equation 10.10 is of practical value, for design.

For a floc particle in suspension, the terminal velocity is equal to the Stokes settling velocity v_s, i.e. $v_t = v_s$ and Equation 9.1 becomes:

$$v_t = \frac{g \cdot (\rho_s - \rho) \cdot d^2}{18\,\mu} \quad \text{.......... } 10.11$$

From Equation 10.10 and 10.11:

$$v_s = \frac{g\,(\rho_s - \rho)\,d^2\,(1 - k \cdot C^{2/3})}{18\,\mu} \quad \text{.......... } 10.12$$

It will be noted from Equation 10.12 that a definite particle size is associated with a given velocity of flow which will expand a suspension until a required porosity is reached; particles with a corresponding velocity will reach an equilibrium and establish a floc blanket.

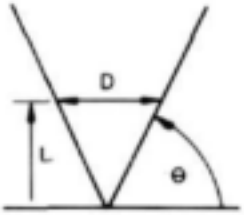
Equation 10.12 can also be written as:

$$d = \left[\frac{18 \cdot v_s \cdot \mu}{[g\,(\rho_s - \rho)]}\right]^{1/2} \quad \text{.......... } 10.13$$

At any height h:

$$v_a = v_h = v_S\,(1 - k\,C_L^{2/3}) = 4Q / \pi \cdot L^2 \ldots \; 10.14$$

For a conical tank as shown



$$D = 2\,L \cot\theta \;\ldots\; 10.15$$

for θ = 63°, D = L

thus $C_L = [1/k - 4\,Q/(k \cdot \pi\,v_s \cdot L^2)\,]^{1/2}$ 10.16

From Equation 10.16 it is seen that for a diverging flow clarifier, for any terminal velocity v_t, the floc concentration C increases as L increases. Therefore the highest concentration occurs at the top of the floc blanket; this is significant in that floc can be decanted from this level into a sludge compartment.

Floc volume concentration can be determined by means of Equation 10.16 and particle diameter can be determined from Equation 10.13 .

Example 10.1

The diameter and settling velocity of alum floc particles can be calculated for a conical tank with an external angle of 63°, a floc volume concentration of 10% at 12,4 m from the bottom and a flow of 130 ℓ/s. Given that the form factor k = 2,78 and floc density = 1,005 g/mℓ, the density of water can be taken as 1,00 g/mℓ and viscosity as 0,0131 cm²/s at 10°C.

From Equation 10.14:

$$4\,Q / \pi\,.\,L^2 = v_s\,(1 - k.\,C_h^{\,2/3})$$

i.e.

$$4 \times 130 / \pi \times (12{,}4)^2 = v_s\,(1 - 2{,}78\,(0{,}1)^{2/3})$$

therefore $v_s = 0{,}269$ cm/s

From Stokes' equation:

$$\begin{aligned} d &= [\,18\,v_s\,.\,\mu\,/g\,(\rho_s - \rho)]^{1/2} \\ &= [\,18 \times 0{,}269 \times 0{,}0131\,/\,981 \times 0{,}005\,]^{1/2} \\ &= 0{,}114 \text{ cm} \\ &= 1{,}14 \text{ mm} \end{aligned}$$

10.1.1 TEMPERATURE EFFECT

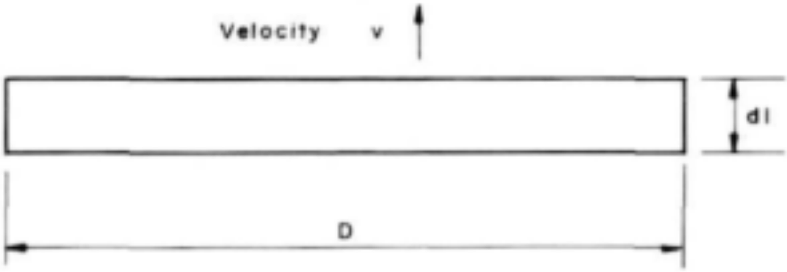
Due to viscosity, the drag effect on a particle increases with a decrease in temperature. It will be seen from Allen's equation (9.2) that settling velocity varies inversely as (viscosity)$^{1/3}$.

Example 10.2

As water temperature changes from 20 to 0°C viscosities are respectively 0,0101 and 0,0179 poise and upflow velocity are thus reduced by 17,5%. This effect has to be taken into account in determining design velocity for a clarifier.

10.2 THE FLOCCULATOR MECHANISM

The operation of a floc blanket clarifier as a flocculator can be done in terms of the D'arcy-Weisbach equation. The diagram below shows a model of a cylindrical unit.



For a pressure drop h, occurring over a unit depth dl and for friction factor f, the D'arcy-Weisbach equation under turbulent flow is:

$$h = f\,.\,dl\,.\,v^2 / 2\,.\,g\,.\,D \tag*{10.17}$$

For a volumetric flow rate Q in the tank,

$$v = 4\,.\,Q / \pi\,.\,D^2 \tag*{10.18}$$

From Equations 10.17 and 10.18:

$$h = \frac{8\,f\,.\,Q^2\,.\,dl}{\pi^2.\,g\,.\,D^5} \tag*{10.19}$$

If power dissipation $P = \rho\,.\,g\,.\,h\,.\,Q$ .... 10.20 then:

$$P = \frac{8\,f\,.\,\rho\,.\,Q^2}{\pi^2.\;D^5} \tag*{10.21}$$

and the power per unit volume V is then:

$$P/V = \frac{32\,.\,f\,.\,\rho\,.\,Q^3}{\pi^3\,.\,D^7} \tag*{10.22}$$

For velocity gradient defined as:

$$G = (\,P / \mu\,.\,V\,)^{1/2} \tag*{10.23}$$

Equations 10.22 and 10.23 yield

$$G = \frac{(32\,.\,f\,.\,\rho\,.\,Q^3)^{1/2}}{(\pi^3.\,\mu\,.\,D^7)} \tag*{10.24}$$

It will be seen from Equation 10.24, that the design variables are Q and D. Usually Q is given and D varies. A high value of G is required for rapid mixing and D is then small. As floc grows, G should become smaller to allow for floc size increase and D should become larger. A conical shaped tank is therefore desirable.

10.2.1 CAMP NUMBER

This number, G T has been used in the design of flocculators for a long time (Camp, 1953). It was an advance on using only the concept of a tapering (reducing) velocity gradient. For slow agitation velocity gradient ranges from 10 to 75 s^{-1} and for rapid agitation it ranges from 200 to 300 s^{-1}. Corresponding duration is 300 to 1 200 seconds for slow agitation and 150 to 450 seconds for rapid agitation (Polasek, 1992).

Over a slow to rapid agitation velocity gradient range of 20 to 300 s^{-1} and for a duration of 150 to 1 200 seconds the Camp number ranges from 3 000 to 360 000.

Example 10.3

A conical clarifier with side angle of 63° (i.e. D = L) receives a coagulated water flow of 13,6 Mℓ/d, (158 ℓ/s). Given that f = 0,03 and kinematic viscosity is 1,3101 centistoke (10^{-2} cm^2/s), calculate the G value at 0.3 m and at 12,4 m from the apex of the cone. Also calculate the distance from the cone point where a flocculation rate of 100 s^{-1} will occur, the average velocity gradient and the Camp number.

From Equation 10.24:

At 0,3 m, $G = 467\ s^{-1}$ and
at 12,4 m, $G = 1{,}2 \times 10^{-3}\ s^{-1}$

For $G = 100\ s^{-1}$,
$L = 0{,}48$ m from the apex of the clarifier.

From $G = 467\ L^{-7/2}$

$$G = \int_{L1}^{L2} \frac{G \,.\, dl}{(L_2 - L_1)}$$

$$= 4{,}8\ s^{-1}$$

Camp number = G T
= G x volume/flow rate

$$\text{i.e. } GT = \frac{4{,}8 \times \pi\, D^2\, L}{12\, Q}$$

$$= 4{,}8 \times 3\,600$$

$$= 17\,280$$ i.e. within the range of 3 000 to 360 000

10.2.2 FLOCCULATION CRITERION

For a floc blanket clarifier the dimensionless flocculation criterion GCT can be derived from Equations 10.24 and Equation 10.14, giving

$$GCT = \frac{L_1^2}{4}\left[\frac{(\rho_s - \rho)\,.\,\pi\,.\,g}{Q\,\mu\,k^{9/2}}\right]^{1/2} .\,F\,\frac{L_u}{L_1} \quad \text{......} \; 10.25$$

For the Camp number over a range of 3 000 to 360 000 and a floc volume concentration of 0.001 yields values of GCT for clarifiers in the range of 3 to 360. Observations indicate a range of 10 to 100 (Ives, 1967). High-rate clarifiers may well have GCT values as low as 1 because of much higher upflow velocities and therefore shorter retention times in the flocculator/clarifier compartment.

10.3 THE FILTER MECHANISM

The floc blanket is a suspension of particles through which water moves; to that extent the floc blanket is a moving bed filter. The question remains whether floc volume concentration is time dependent or not. However, floc blockage does not occur because there is a continuous ventilation of the filter blanket. A first-order filter equation can therefore be valid; thus

$$- dw/dl = La \,.\, W \qquad 10.26$$

where La is a filter coefficient and W is the concentration of particles. The filter coefficient depends on the probability of a suspension particle colliding with a floc surface. The coefficient will be proportional to a specific area S of the floc particle and v_s will be the local (Stokes'equation) particle velocity, thus:

La is proportional to S / v_s

$$\text{and } S = 6\,C / \Psi \,.\, d \qquad 10.27$$

where:

C = floc volume concentration
Ψ = sphericity
d = floc diameter

For a conical tank with D = L, from Equation (10.16) La is proportional to $6 / \Psi.d.v_s (1/k - 4Q/k.\pi.L^2)^{3/2}$.

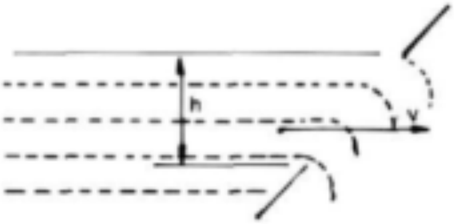
Thus, as L increases, i.e. as floc is carried by the floc blanket, so La and thus filter removal efficiency increases. The evidence supporting this model is based on the finding that particles are filtered out of a suspension stream regardless of the flow being upwards or downwards (Ison, 1967). Flocculation and sedimentation may well both play a role in the floc blanket.

10.4 FLOC PRODUCTION AND SEPARATION

Continuous addition of suspension to a clarifier necessitates the removal of floc from the blanket, otherwise carry-over will occur into the clear water. Removing too much floc will reduce or destroy the blanket. For steady-state conditions, removal of floc should equal suspension feed and also sludge production and removal.

It would appear that 100 to 300 m*ℓ* of floc is produced per 1 gram of Al^{3+} or Fe^{3+} ion used as coagulant. Attempts were made to draw off the surplus floc or sludge by means of sludge pockets or worse still, to hope that it would settle in a counter-current manner. The more satisfactory decanting procedure has come to be recognised as most effective for floc removal.

The discharge of floc over a submerged weir of the orifice type can be analysed as follows:



The velocity and discharge of sludge overflowing a weir crest as shown in the diagram can be calculated from Bernouilli's equation as

$$v^2 / 2g = h \,.\, (\rho_s - \rho) / \rho \qquad 10.28$$

or

$$v = [(2gh / \rho p) (\rho_s - \rho)]^{1/2} \qquad 10.29$$

where h = height of floc above the weir and the other factors are as before.

The discharge of floc Q_s over a unit length of crest is given by:

$$Q_s = [(2gh / \rho) \,.\, (\rho_s - \rho)]^{1/2} \int_0^h h^{1/2}\, dh$$

i.e.

$$Q_s = 2/3\, [(2g / \rho) \,.\, (\rho_s - \rho)]^{1/2}\, h^{3/2} \ldots 10.30$$

For a total weir length b

$$Q_s = 2/3\, [(2g / \rho) \,.\, (\rho_s - \rho)]^{1/2}\, b.\, h^{3/2} \ldots 10.31$$

For suspension density in terms of sludge volume concentration:

$$\rho_s = C\,\rho_f = (1 - C)\,\rho \qquad 10.32$$

where:

ρ_s = suspension density

and

ρ_f = floc density

hence:

$$v = [(2g \,.\, C \,.\, h / r_o)\,(\rho_f - \rho)]^{1/2} \qquad 10.33$$

The discharge of floc over a unit length of crest is given by:

$$Q = 2/3\, b\, [(2\, g\, C\, h^3)/\rho)\,.\,(\rho_f - \rho)]^{1/2} \qquad 10.34$$

Example 10.4

For a 1 m length of weir, a blanket level of 10 mm above weir edge, floc volume concentration of 0,3, floc density of 1,0025 g/mℓ and liquid density of 1,000 g/mℓ, calculate submerged weir flow velocity and flow.

From Equation 10.33:

$$v = [2 \times 9810 \times 0{,}3 \times 10 \times 0{,}0025]^{1/2}$$
$$= 12{,}1 \text{ mm/s}$$

From Equation 10.34:

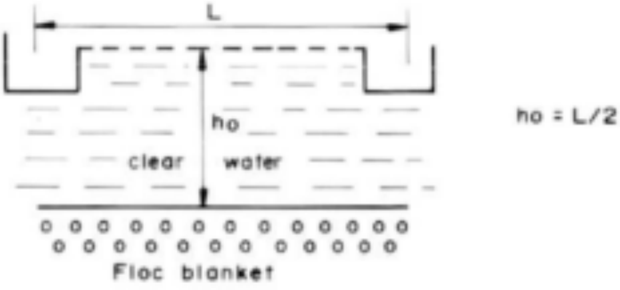
$$Q = 2/3 \times 1\, [2 \times 9810 \times 0{,}3 \times 10^3 \times 0{,}0025]^{1/2}$$
$$= 81 \text{ m}\ell\text{/s}$$

10.5 CLARIFIED WATER COLLECTION

It is essential to collect the clarified water properly in order to avoid short-circuiting and thus floc carry-over. Because floc blanket clarifiers are multi-compartmented, attention must also be given to secondary flows. Some amount of clarified water flows with the floc into and is available from the sedimentation/sludge thickener compartment. Closely spaced collector channels or pipes are required at the top of the upflow compartment of the clarifier.

A freeboard must be allowed in the upflow zone above the top level of the floc blanket. The position of the weirs governs the minimum freeboard in relation to the level of the floc blanket. If distances less than the minimum are used, streaming towards the weir can cause some of the floc blanket to be drawn into the clear water stream.

The relationship between the top level of the floc blanket and the distance to the collecting weir was solved by the sink and source method (Tesarik, 1967). A system of complex numbers and transforms shows that minimum depth h_0 equals half the distance between the weirs (L).



The theory and operation of clarifiers presented show that a floc blanket clarifier can be analysed in terms of its operation as a flocculator, a filter, a sedimentation unit and a sludge thickener. Theories of these unit operations are usable for design purposes.

Some amount of experimentation remains to be done e.g. with regard to other than iron and aluminium floc particles. It is also advisable and is readily possible to model high-rate clarifiers by means of a small unit. A 2-m diameter model can well give meaningful results. Such a model was operated at upflow rates in excess of 20 m/h, with inflow turbidities of more than 100 units and clarified turbidities of less than 1 unit (Polasek, 1979).

Example 10.5

A number of units are required to clarify 90 Mℓ/d (1 050 ℓ/s) of a properly coagulated water; ten units would be acceptable. Floc density is 1,005 g/mℓ. At the ambient temperature of 10°C the kinematic viscosity of water is 0,0131 cm²/s and density can be taken as 1,000. The roughness factor, f can be taken as 0,008 and Bond's s value as 2,78. Floc volume concentration will be of the order 0,001 and GCT can be taken as 100.

Assuming that the floc size will be 1,5 mm, the Stokes velocity is v_s.

$$v_s = \frac{981\ (1{,}005 - 1{,}000)\ (0.15)^2}{18 \times 0{,}0131}$$

$$= 0{,}468 \text{ cm/s}$$

The lowest level of the floc blanket can be calculated from Equation 10.14 for $C_L = 0$

$$v_s = \frac{4\,Q}{\pi L_1^{\,2}}$$

or:

$$L_1^{\,2} = \frac{4\,Q}{\pi v_s}$$

$$= \frac{4 \times 0{,}105}{\pi \times 0{,}00468}$$

hence $L_1 = 5{,}34$ m

From Equation 10.25, for GCT = 100

$$100 = \frac{(5{,}34)^2[0{,}005\ \pi\ 981]^{1/2}}{4\ [0{,}105 \times 0{,}0131 \times 2{,}78^{9/2}]L^{1/2}} .\ F\frac{(L_u)}{(L_1)}$$

i.e.

$$F\frac{(L_u)}{(L_1)} = 0{,}146$$

The function is an integral in an equation which can be treated as a series (Ives,1967). From a semi-log graphical plot of the function against $(L_u)/(L_1)$ the corresponding values are

$F(L_u/L_1)$	0,01	0,10	0,146	0,25	0,50	1,0	2,5	4,5
L_u/L_1	1,00	1,25	1,35	1,50	1,75	2,0	2,5	3,0

$$\text{For } F\frac{(L_u)}{(L_1)} = 0{,}146 \text{ i.e. } \frac{(L_u)}{(L_1)} = 1{,}35$$

hence $L_U = 7{,}21$ m

The floc blanket depth is therefore 7,21 - 5,34 = 1,87 m.

The floc volume concentration at the top of the lanket is given by Equation 10.16

$$C_L = (1/2{,}78 - 4 \times 0{,}105/2{,}78\pi \times 0{,}00468 \times 7{,}21^2)^{1/2}$$
$$= 0{,}079 \text{ i.e. } 7{,}9\ \%$$

The upflow velocity at the top of the blanket is given by Equation 10.18 as

$$\begin{aligned} v &= 4 \times 0{,}105\ /\pi\, 7.21 \times 7.21 \\ &= 0{,}00257 \text{ m/s} \\ &= 9{,}26 \text{ m/h} \end{aligned}$$

The quantity of floc to be drawn off from every tank each day can also be calculated from a consideration of the coagulant used. If a maximum dosage of 100 mg/ℓ of aluminium sulphate is used, then:

$$\begin{aligned} \text{alum dose/d} &= 100 \times 9 \times 10^6 \times 1/1000 \text{ g/d} \\ &= 9 \times 10^5 \text{ g/d} \end{aligned}$$

However, 1 g of alum contains 0,81 g of aluminium ion and therefore:

$$\begin{aligned} \text{alum dose/d} &= 0{,}081 \times 9 \times 10^5 \\ &= 7{,}29 \times 10^4 \text{ g/d} \end{aligned}$$

If 1 g of alum yields 0,240 ℓ of floc, then
floc produced/d $= 0{,}240 \times 7{,}29 \times 10^4$
$= 17{,}5 \times \text{m}^3/\text{d}$

For floc volume concentration of 0,079, the volume of floc produced per day at the top of the floc blanket would be:
$= 17{,}5 / 0{,}079$
$= 221{,}5 \text{ m}^3/\text{d}$

This represents 221,5/9 000 = 0,0246, i.e. 2,46%. It has been found with a high-rate clarifier which has a satisfactory thickener system that up to 15% of solids concentration is attained. The sludge draw-off can therefore be reduced substantially to less than 0,5% of the flow.

10.6 TYPES OF CLARIFIER

Various types of clarifier have been developed and these can be classified to some extent. There are the classical rectangular, square and circular tanks which provided specifically for one or more of the mechanisms described.

The classical tanks were usually free of mechanical devices. The solids contact type of tank contained a number of mechanical devices for various purposes such as flocculation or recirculation. Other developments were effected, which indicated a greater realisation of the mechanisms involved and less reliance on mechanical devices.

Clarifiers are rectangular, square or circular. Whatever the shape, the flow direction is vertical thus providing for solids to move downwards and the clear water to rise upwards.

10.6.1 HYDRODYNAMIC CONFIGURATIONS

A flocculator/clarifier/sedimentation tank was developed at the CSIR for use in gold mines (Chunnett, 1961). The objective was to obtain uniform flow velocities in the sedimentation section of the tank by providing a tangential inlet flow section, exploiting the resulting stilling and temperature stabilisation effects of the particular configuration.

A modified version of the CSIR tank is shown in Figure 8.4 (floc blanket flocculator); it has a proper draw-off system at the top of the floc blanket; the bottom sludge draw-off is retained for shutdown operations.

The CSIR tank, shown in Figure 10.2, was

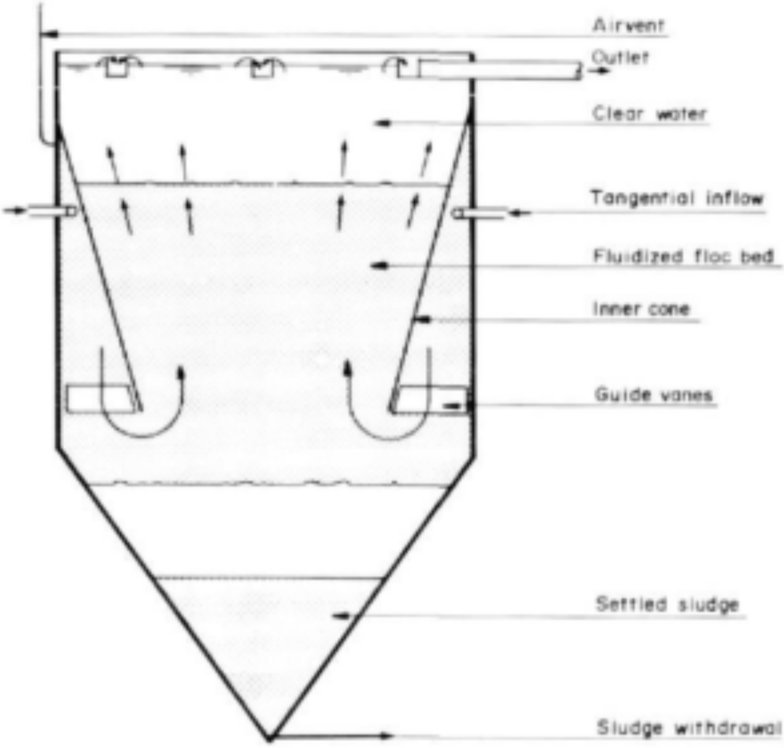


Figure 10.2
CSIR floc blanket clarifier and sedimentation tank

based on work originally carried out on sewage clarification.

This tank inlet was modified from the original model, to suit water clarification. The conical skirt replaced a cylindrical annulus, thus creating spiralling flow into a widening section. The ideal of tapered flocculation would take place in this section of the tank.

For the skirt fixed at any particular sedimentation angle, only a certain flow would yield optimum flocculation in the spiral flow section. The sludge removal system of this tank was deficient due to inadequate design attention to this aspect.

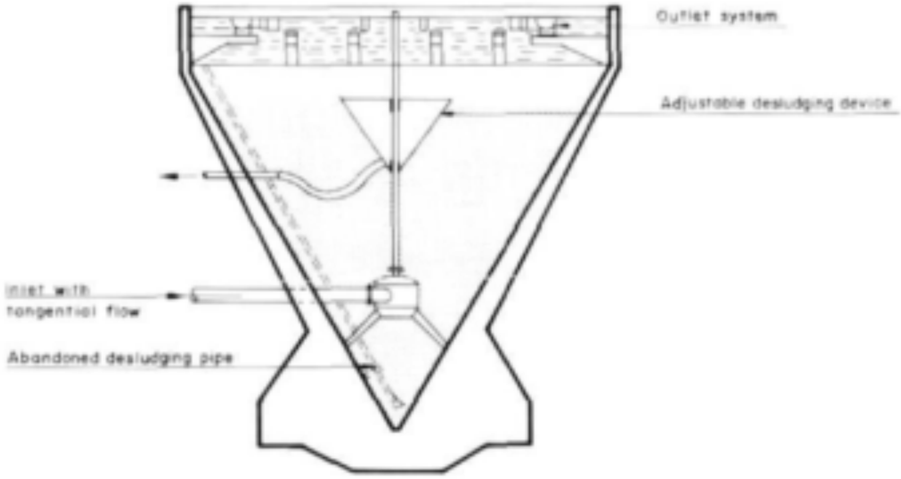


Figure 10.3
Clarifier with hydrodynamic flocculator

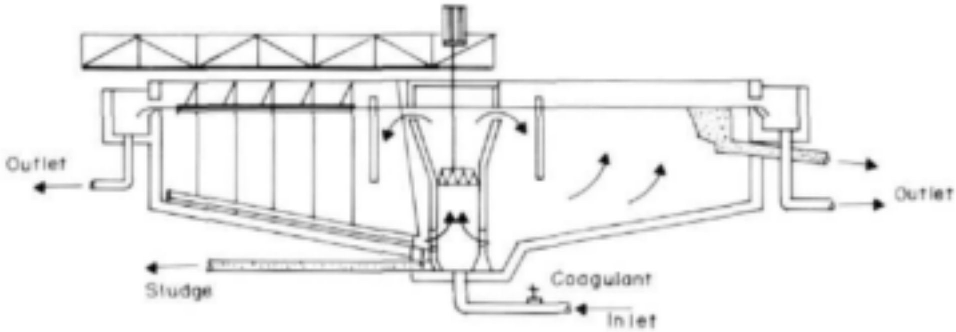


Figure 10.4
Degremont Turbocirculator (Courtesy Aquazur)

Another hydrodynamic clarifier, free of mechanical devices, was developed for Krefeld am Rhein. This is a Candy tank which was modified by the addition of a tangential flow, bottom inlet device and an adjustable, desludging device at the top of the floc blanket.

This tank is similar to the CSIR unit to the extent that they both operate hydrodynamically. The problem of separation floc and sludge zones was not resolved in the CSIR tank.

The Krefeld clarifier is shown in Figure 10.3. The downflow inlet to the Candy tank was modified to create a spiral flow flocculator in the Krefeld tank.

The desludging pipe which reached the apex of the pyramidal hopper bottom of the Candy tank was removed and replaced by an adjustable conical floc/sludge draw-off device. The top level of the floc blanket can therefore be controlled.

As a result of the removal of the Candy tank desludging pipe the Krefeld tank can not be desludged hydrostatically, in the event of shutdown of the unit or works.

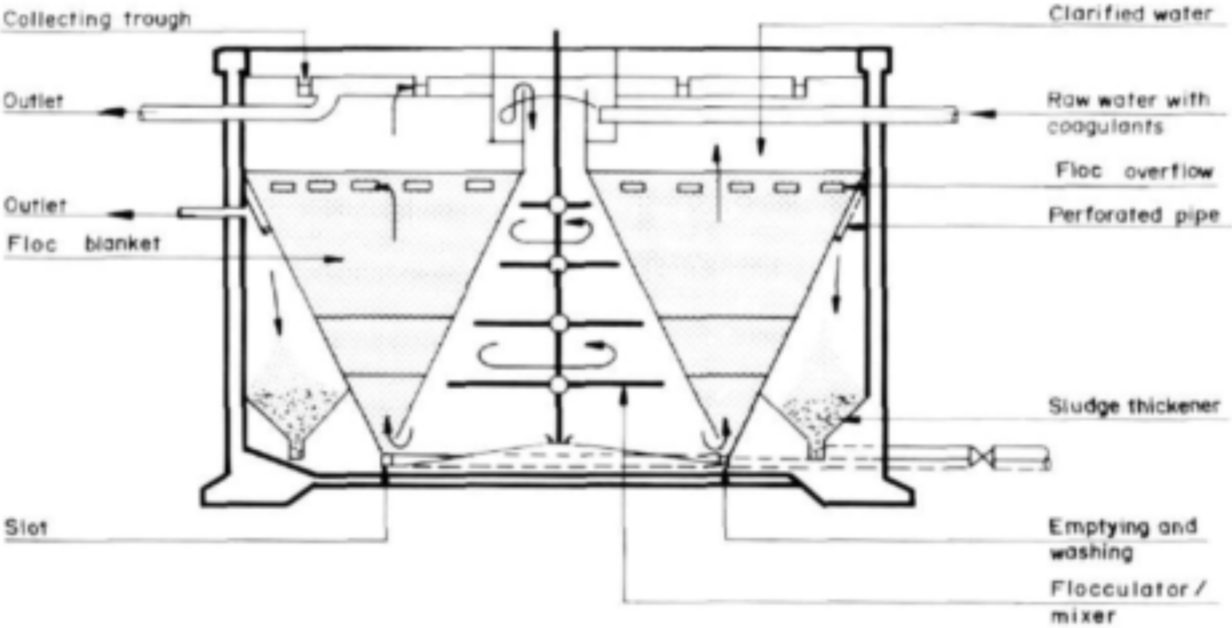


Figure 10.5
BRNO floc blanket clarifier

10.6.2 MIXER/FLOCCULATOR CLARIFIERS

The clariflocculator is the classical design of radial flow clarifier, with an internal flocculation facility. The raw water inlet is into a central, flocculation compartment and flows radially upwards from there into an annular clarifier compartment. Integration of the units avoids the problem of floc break-up caused by a pipe connection between the typical channel flocculator and a sedimentation tank.

Because of its radial flow pattern, resulting from the central inlet and the peripheral, ring outlet launder, the upflow velocity in this unit is usually less than 1 m/h. The hydraulics of the tank can be improved by offsetting the launder away from the vertical outside wall. The design upflow velocity is also affected by the suspension, coagulant and floc characteristics.

The weir loading rate of the tank is directly proportional to the diameter. Upflow velocity is determined by the surface area and is thus proportional to the square of the diameter. Small Clariflocculators are limited by the upflow rate whereas the larger unit is limited by the weir overflow rate.

The Degremont Turbocirculator, shown in Figure 10.4, is similar to the Clariflocculator. It has a variable speed, low-lift pump flocculator which recirculates floc via a central compartment. It has an upflow velocity of some 1,5 m/h and similar weir loading rates to the Clariflocculator. This unit can well be used as a primary phase separator in the same way as a roughing filter.

The BRNO type of tank is shown in Figure 10.5 (Ives, 1967). It has a tangential, top inlet, leading to a conical shaped mixer/flocculator, an inverted cone shaped clarifier compartment and floc overflow into a sludge compartment.

This slot clarifier, developed in Czechoslovakia, clearly has specific provision coagulant dosing, mixing and all the unit operations - with good flow continuity. Tapered flow is provided for in all the compartments, thus continuously reducing velocity and thereby momentum through all the operations.

10.6.3 SOLIDS CONTACT CLARIFIERS

This type of tank was developed for lime softening of water. However, the collision opportunity provided in the inlet led to its use for turbid waters which require flocculation. Typical of these is the Infilco-Accelator, shown in Figure 10.6.

10.6.4 MULTIPLE COMPARTMENT TANKS

This type of tank was developed in Germany (Roth) and Australia (Bond, 1959; 1966). As shown in Figure 10.7 there are no mechanical devices and these tanks are usually rectangular in shape. These tanks indicate a recognition of the need to provide for all the mechanisms of a floc blanket clarifier to take place by defined operations. The interconnected compartment configuration ensures a lack of short-circuiting, low turbulence and effective clarification.

The Roth tank demonstrates the obviation of mechanical devices and successful reliance on hydrodynamic flocculation, clarification and sedimentation. Provision was also clearly made for thickening, desludging and shut-down requirements.

10.7 EFFICIENCY OF CLARIFICATION

Two aspects of efficiency are the physical comparison with sedimentation and the relative qualities achieved. The physical comparison is based on flow velocities and retention time; these affect capital expenditure not only on the tanks and the entire water works but also on site infrastructure and operational ease. A high quality effluent can readily be produced by a clarifier.

Sedimentation as a unit operation is very robust, difficult to control and medium efficient. Its long history of reliability despite the lack of understanding and poor operation, has probably caused it to be one of the most ubiquitous unit operations in water engineering. Clarification is a modern development of closely linked unit operations.

Maximum upflow rate in a sedimentation tank is 1,5 m/h whereas 10 m/h can very readily be attained in a clarifier.

The comparison of quality can be based primarily on turbidity and thus solids removal, but also on other parameters such as precipitates and colour. Similar turbidities can be attained in sedimentation tanks and clarifiers. However, due to short retention times and frequent desludging, colour, taste and odour are greatly reduced in clarifiers.

10.8 DESIGN CONSIDERATIONS AND CRITERIA

The upflow rates in the clear-water compartment of a clarifier can be compared with the upflow rate in a sedimentation tank. In the past sedimentation design was based on so-called surface loading. In Imperial measure terms the surface loading units were gallons per day per square foot. Analysing this dimensionally it is found that surface loading is $L\ T^{-1}$ i.e. velocity, and the question then

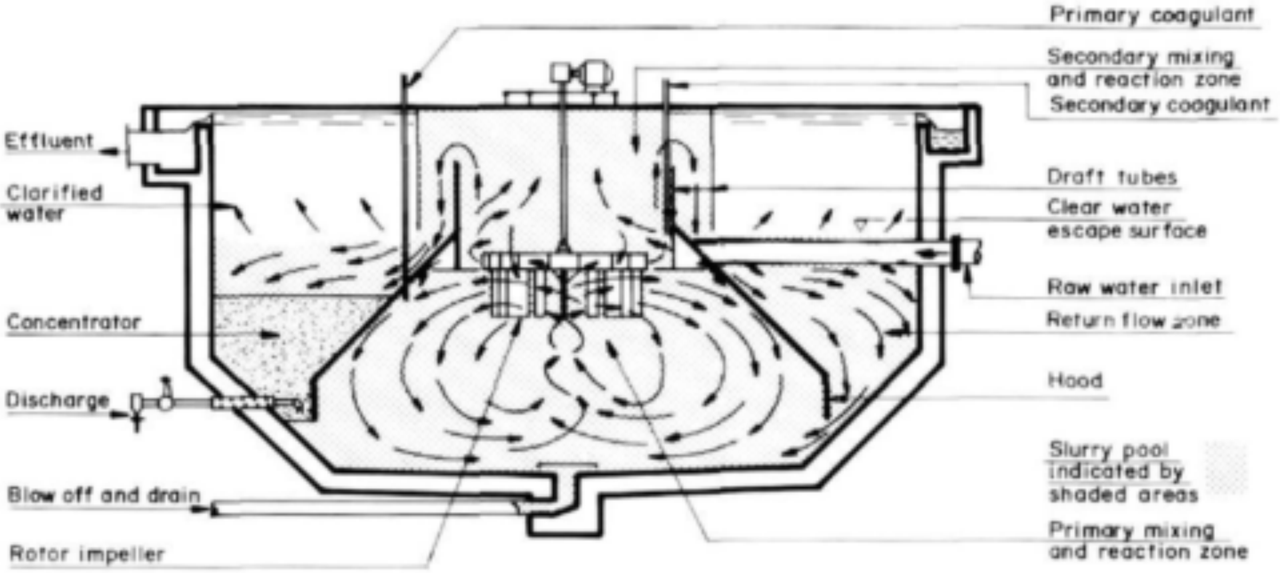


Figure 10.6
Infilco-Accelator

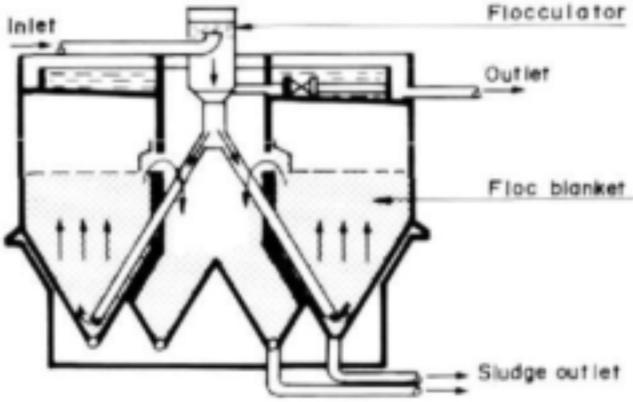


Figure 10.7
Roth, floc blanket clarifier

arises - what velocity?

Logically, sedimentation of solids must be downwards and this must exceed the upward flow of the water out of the tank. Therefore the velocity parameter for sedimentation design purposes is the effective upward velocity of the clear water in the tank.

The high-rate clarifier shown in Figure 10.1 was model tested over a period of more than two years at upflow rates as high as 23 m/h. At the highest upflow rate, turbidity exceeded 10 units because of the difficulty of accurate coagulant dosage control for the small flow. Up to 17 m/h the effluent turbidity from the model was readily maintained at < 10 units.

A 20 Mℓ/d water works was designed and built at Bethlehem in the Orange Free State, in 1977 with two high-rate clarifiers operating in parallel. The design upflow rate of 10 m/h readily yields a turbidity of < 5 units from a water which has a turbidity of up to 250 units. The works was expanded in 1992 to 40 Mℓ/d to deal with water which will have very low turbidity at times. A polyelectrolyte or poly aluminium chloride is used as coagulant, depending on the level of turbidity of the raw water.

The high-rate operation made it possible to enclose the units, thus eliminating the influence of climate and improving operation in all weather conditions.

In view of the high rate that can be attained with clarifiers compared to sedimentation takes it would appear that there is especially scope for these units on larger works. The area occupied by large works can be sub-

TABLE 10.1
DESIGN CRITERIA FOR WATER WORKS CLARIFIERS

Characteristic	Criterion and description
Inlet velocity	5 m/s
Upflow velocity	5 to 15 m/h for conical tanks
Weir overflow rate	< 10 m^3/h.m
Retention time	< 2 h for conical tanks
Solids loading	5 kg/m^2.d
Number of tanks	>2
Side wall depth	10 m for conical tanks <5 m for rectangular tanks
Floor slope:	
with hydrostatic desludging	1 in 12 for conical tanks
with mechanical desludging	1 in 100 for rectangular tanks
Sludge drain:	
line diameter	>150 mm diameter
flow velocity	1,5 m/s
Desludging water loss	< 2%

stantially reduced. It would appear thus far that there are no contra-indications to the use of clarifiers. The use of clarifiers at Bethlehem was preceded by three years of model testing. Further units of this kind have since been installed at various water works in South Africa.

10.8.1 DESIGN CRITERIA

Design criteria are given in Table 10.1 to assist the process and design engineer. These criteria are based on the literature cited and on investigations, research and experience. However, the particular conditions that prevail for a given water source and the treatment applied to it will determine the applicability of any particular criterion. Experimentation and model testing will yield the data and information required to use clarifiers with confidence.

11

Dissolved Air Flotation

L R J van Vuuren

Dissolved air flotation (DAF) is the process whereby small bubbles are precipitated when water, supersaturated with air under high pressure, is released under atmospheric conditions (see Figure 11.1 on next page). It deals essentially with the production of potable water from eutrophic surface waters.

The more relevant fundamental aspects and literature are briefly reviewed. This is followed by subsections on design guidelines and examples for design. For more detailed information including DAF thickening of sludges, the reader is referred to a *South African Design Guide* published by the Water Research Commission (1993).

DAF will be dealt with here in terms of fundamentals, design considerations and extent and design examples.

11.1 FUNDAMENTALS

There are generally two applications for DAF in the water industry, namely **clarification** or **thickening**. In clarification the emphasis is on the quality of the subnatant or effluent. In thickening the emphasis is on the character of the float layer on the surface of the tank. The aspects of the fundamentals of clarification that will be dealt with here are :

- **Reaction zone**: Flotation is more complex than other phase separation processes, in that three phases are involved. The particles, water and bubbles come together for the first time in the reaction zone.
- **Chemical pretreatment**: It is necessary to coagulate/flocculate particles before successful flotation can be achieved.
- **Bubble production system**: Bubble production comprises the abstraction and pumping of treated water (the recycle) through an air saturation system, and injection of the supersaturated water into the reaction zone.
- **Flotation zone**: After formation of stable particle/bubble agglomerates, the sought-after phase separation takes place in the flotation zone.
- **Float layer removal**: The various methods of float layer removal each has distinct advantages and disadvantages, for sludge consistency and subnatant quality alike.
- **Combined flotation/filtration**: This particular DAF application has a number of special considerations, dealing mostly with the filterability of the subnatant from the flotation zone.

11.1.1 REACTION ZONE

The first step towards successful flotation is the formation of stable particle/bubble agglomerates. These are formed in the reaction zone, where the pressurised recycle is released in close proximity to the incoming feed.

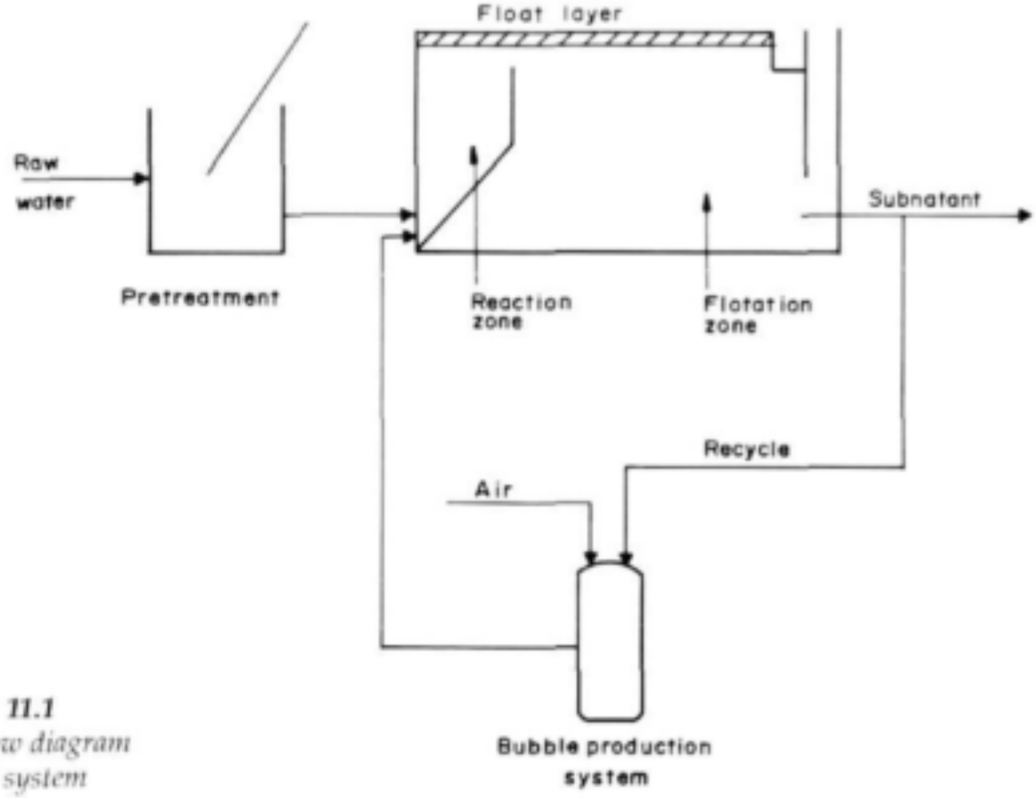


Figure 11.1
Schematic flow diagram of a DAF system

11.1.1.1 Formation models for particle/bubble aggregates

Four conceptual models for the formation of particle/bubble aggregates have been proposed (Kitchener & Gochin, 1981; Schade, 1982; Vrablik, 1959) namely:

• Model A	Adhesion of bubbles to preformed flocs.
• Model B	Mechanical enmeshment of bubbles with preformed flocs.
• Model C	Incorporation of bubbles into growing flocs.
• Model D	Growth of bubbles on nuclei within flocs.

Although Models A and B are considered to be the dominant ones, Model C was demonstrated to be feasible under conditions of high-particle concentration, whereas Model D (growth within flocs) has been proposed as a method to make more efficient use of air.

11.1.1.2 Bubble size and coalescence

Small bubbles are essential for DAF for the following reasons (Rovel, 1976; Komline, 1976):

- Smaller bubbles imply that more bubbles can be precipitated from the same quantity of air.
- Smaller bubbles rise more slowly and will offer more time and better opportunity for attachment.
- Larger bubbles will rise faster and will be more likely to be detached.
- Longhurst & Graham (1987) recommends that the bubbles be smaller than the particles to be floated.

Considerable emphasis had therefore been placed on the measurement of bubble sizes actually encountered in DAF. The bubble sizes generally follow a normal distribution, with fairly uniform sizes. The average bubble sizes range from about 40 µm to 110 µm (Jones & Hall, 1981; Shannon & Buisson, 1980; Lovett & Travers, 1986).

Three factors favour bubble coalescence (Ramirez, 1979):

- Non-uniform bubble diameters, where differential rise rates will increase bubble/bubble collision opportunities.
- Violent agitation, where bubbles forcefully collide.
- The presence of a large surface area, which supplies many deposition points where microbubbles will coalesce.

11.1.1.3 Particle sizes

The typical particle size is much more variable than the bubbles, and it could be substantially different for every application.

Theoretically, it was shown (Edzwald *et al.*, 1990) that the ideal particle diameter for water treatment DAF applications should be between 10 and 50 µm; although in practice DAF can be successful in floating much larger particles (for example flocculated algae).

11.1.1.4 Transport mechanisms

A number of important general conclusions were drawn (Edzwald *et al.*, 1990; Reay & Ratcliff, 1973):

- The two dominant transport mechanisms are Brownian diffusion and interception. Gravity (settling) is of secondary importance.
- The most influential parameter is the bubble size. The collision efficiency improves dramatically as the bubble size decreases.
- The effects of particle density and temperature on the collision efficiency are negligible.

11.1.1.5 Attachment and hydrophobicity

For a bubble to "stick" to a particle, the particle surface must be hydrophobic to allow a finite contact angle between particle and bubble (Ramirez, 1979). Naturally occurring suspensions automatically have enough hydrophobicity to allow successful particle/bubble adhesion. Attachment is dominantly achieved through Van der Waal's forces.

11.1.1.6 Detachment mechanisms

A combination of turbulence and gravitational forces will detach a fraction of the particles. The detachment probability gets higher as the particle size increases (King, 1982).

11.1.1.7 Reaction zone configuration

A number of general prerequisites for the ideal reactor zone can be based on fundamental considerations such as those described above:

- Particles and bubbles should be in their respective size ranges.
- Enough turbulence to provide adequate collision opportunities (Kitchener & Gochin, 1981).
- Regions of high shear, such as edges of blades, should be avoided.
- The hydraulics through the reaction zone should approximate plug flow.
- Sufficient time should be allowed to establish particle/bubble agglomerates.
- The reaction zone should be confined to a separate compartment ahead of the flota-

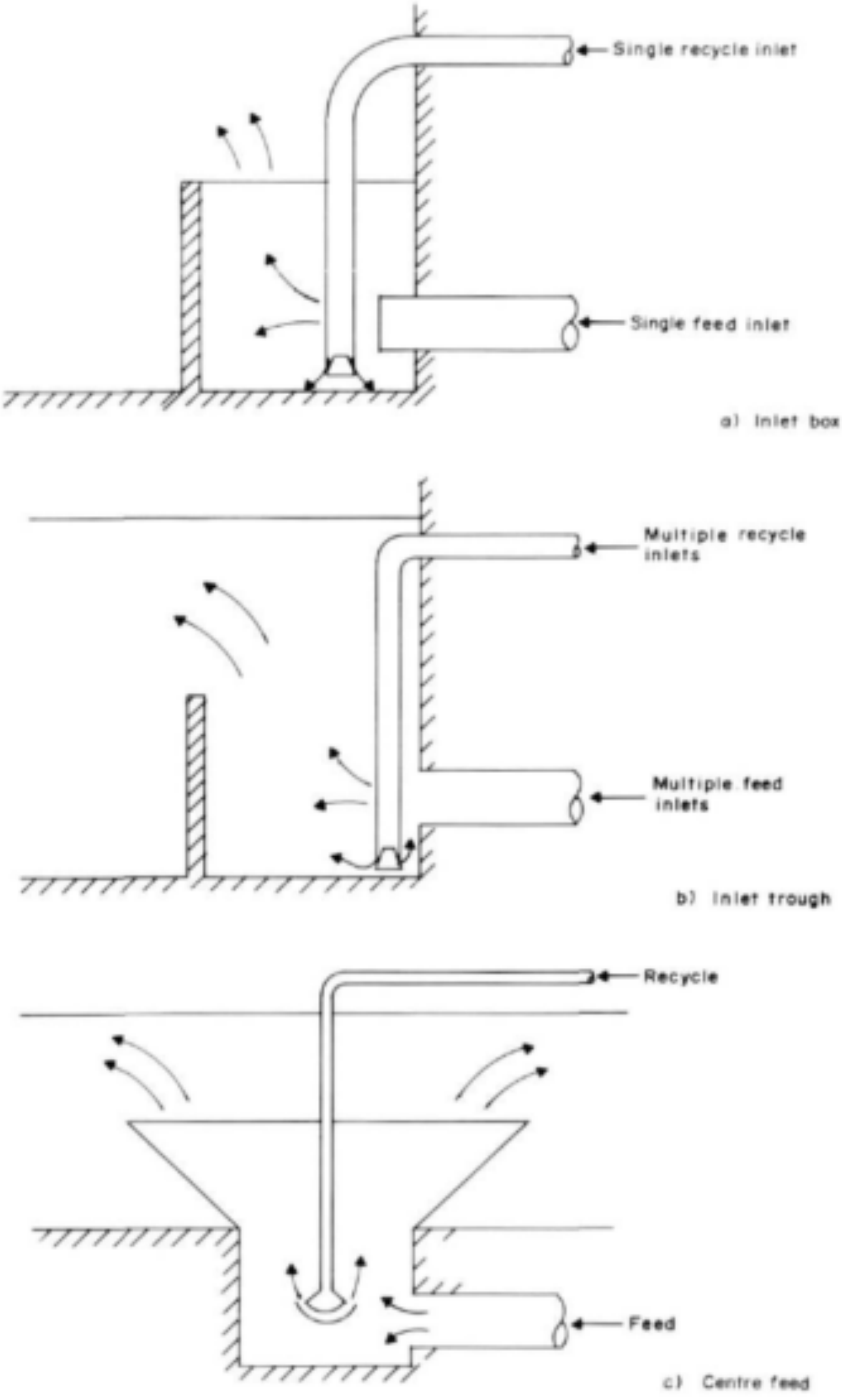


Figure 11.2
Typical reaction zone configurations

tion zone (Kitchener & Gochin, 1981).

- Air must be released very close to the point where the recycled water is mixed with the flocculated water (Zabel, 1985).

It is not a simple matter to design a reaction zone to comply with all the above requirements. A number of different arrangements have been recorded in the literature, of which a few are illustrated in Figure 11.2.

One principal difficulty lies with the quantification of the turbulence within the reaction zone. One method is to calculate a rise rate within the reaction zone itself. For the flotation of algal waters, for example, a rise rate of 60 m/h has been suggested (De Wet, 1980). Similarly, a crossflow rate from the reaction zone to the flotation zone has also been suggested. For algal waters, De Wet (1980) recommends 80 m/h.

11.1.2 CHEMICAL PRETREATMENT

Ferric chloride, aluminium sulphate and synthetic polymers are typically used as coagulants, with polymers as flocculants. The pH is typically adjusted by lime or acid (as modifiers) to get to the range where the coagulants operate best, and to render the water stable in terms of its corrosivity.

11.1.2.1 Chemical dosage determination

For conventional clarification processes, chemical dosage is normally determined with a standard jar test, in which the processes of rapid mixing, flocculation and settling are simulated. A detailed systematic procedure of conducting a bench-scale flotation test is given in the literature (Zabel & Melbourne, 1980).

11.1.2.2 Coagulation /flocculation parameters

The rapid dispersal of chemicals into the water was shown to be important and an in-line mixer was eminently suitable for clarification of a range of raw waters (Sandbank, 1983; Rees *et al.*, 1980b), without the need for a separate coagulation vessel. An in-line mixer was found to be more effective than a standard flash mixer (Zabel, 1978; Rees *et al.*, 1979).

There are conflicting suggestions on the need and extent of flocculation prior to flotation.

The following findings are reported on a number of flocculation aspects:

- **Flocculation time**
  For algal-laden oxidation pond water, a flocculation chamber was found to be unnecessary (Bratby & Marais, 1973). Contrary to these findings, pilot programs showed that a separate flocculation step is indispensable (Rees *et al.*, 1980b; Hyde, 1975; Packham & Richards, 1975b). A flocculation time of 12 minutes was typically required for algal water, but in the case of a soft, coloured water, a significant improvement was found at a flocculation time of 16 minutes. A study specifically directed at three different types of algal water (De Wet, 1980) recommended a minimum flocculation period of 6 minutes - longer periods did no harm. In Sweden, flocculation periods of 20 minutes or longer are commonly provided (Rosen & Morse, 1976).

- **Flocculation intensity**
  A study on a wide variety of waters (Rees *et al.*) found that a velocity gradient (G) of 75 s^{-1} was typically required.

- **Gt-product**
  Very few studies have suggestions on the traditional dimensionless product of flocculation time and velocity gradient. For algal water, a minimum value of 1×10^4 was suggested (De Wet, 1980).

- **Tapered flocculation**
  No benefit was found from tapered flocculation after aluminium sulphate addition, except for a marginal improvement in the case of a turbid river water (Bratby, 1982). A prominent designer of flotation plants in Sweden, however, stated unequivocally that tapered flocculation was the most successful method, where a flocculation time of 20 minutes or more was commonly adopted (Conference on, 1977).

- **Type of flocculation**
  There is no agreement on the best type of flocculation device. Baffled hydraulic channels and pipe flocculators are favoured because of their simplicity and because they closely approximate plug flow. It was stated (Conference on, 1977 : 407-

416), that three minutes of hydraulic flocculation is equal to ten minutes of mechanical flocculation. On the other hand, hydraulic channels are not favoured because they cannot offer the same degree of control (Longhurst & Graham, 1987).

11.1.3 BUBBLE PRODUCTION SYSTEM

Firstly, a pumping system is required to pump a controlled amount of water through the recycle system. Secondly, the water needs to pass through the saturation system, where enough contact with air under pressure is allowed. Thirdly, the water must be distributed among flotation tanks where necessary. Fourthly, the pressurised recycle must be released into the tank through a injection device which lowers the pressure to precipitate small bubbles.

11.1.3.1 Saturation systems

Numerous cost comparisons between DAF and the more conventional settling have been published. In all these comparisons, it is clearly shown that DAF has lower capital cost, but higher operating costs. About half of the high operating cost is caused by the energy required to dissolve air into water at high pressure (Rees *et al.*, Rodman & Zabel, 1980a). Practical designs apparently leave much to be desired in terms of efficiency. A survey of six DAF systems showed saturator efficiencies of 77%, 62%, 48%, 45%, 31% and 30% (Roberts *et al.*, 1978) while it appears from other studies that an efficiency of at least 50% should be attainable regardless of the saturation system used.

11.1.3.2 Air solubility

The solubility of air is mainly determined by the temperature and absolute air composition. The above arguments are embodied into Henry's Law, which relates the partial pressure of the gas in the atmosphere above the water to the quantity of gas dissolved in the water.

The composition of atmospheric air (excluding the small quantity of other gases) is about 79% nitrogen and 21% oxygen. When this mixture is pumped into a saturator, the oxygen fraction dissolves more rapidly in the water than nitrogen due to its higher solubility. The oxygen fraction in the saturator air will therefore decrease while the nitrogen fraction will increase. When equilibrium is reached, the composition of saturator air will be about 88% nitrogen and 12% oxygen. Saturator air will therefore be about 9% less soluble in water than atmospheric air, as is indicated on Figure 11.3.

This point has an important bearing on the expression of the air transfer efficiency of saturators. It is therefore more appropriate to rather define saturation concentration in terms of saturator air, in which case a totally efficient saturator will in fact show up as 100% efficient (Zabel & Melbourne, 1980).

A second point that has a bearing on the expression of saturator efficiency, is whether the absolute air concentration is considered, or only the air concentration beyond the atmospheric equilibrium. For DAF applications, the interest is only in the fraction of the air that will precipitate as bubbles, which makes it more appropriate to work with the air in excess of atmospheric solubility. For this reason, Figure 11.3 is expressed in terms of the gauge pressure, which gives the excess air concentration directly.

11.1.3.3 Measurement of air concentration

For water clarification, the air concentration is simply given as a mass concentration rather than the dimensionless air/solids

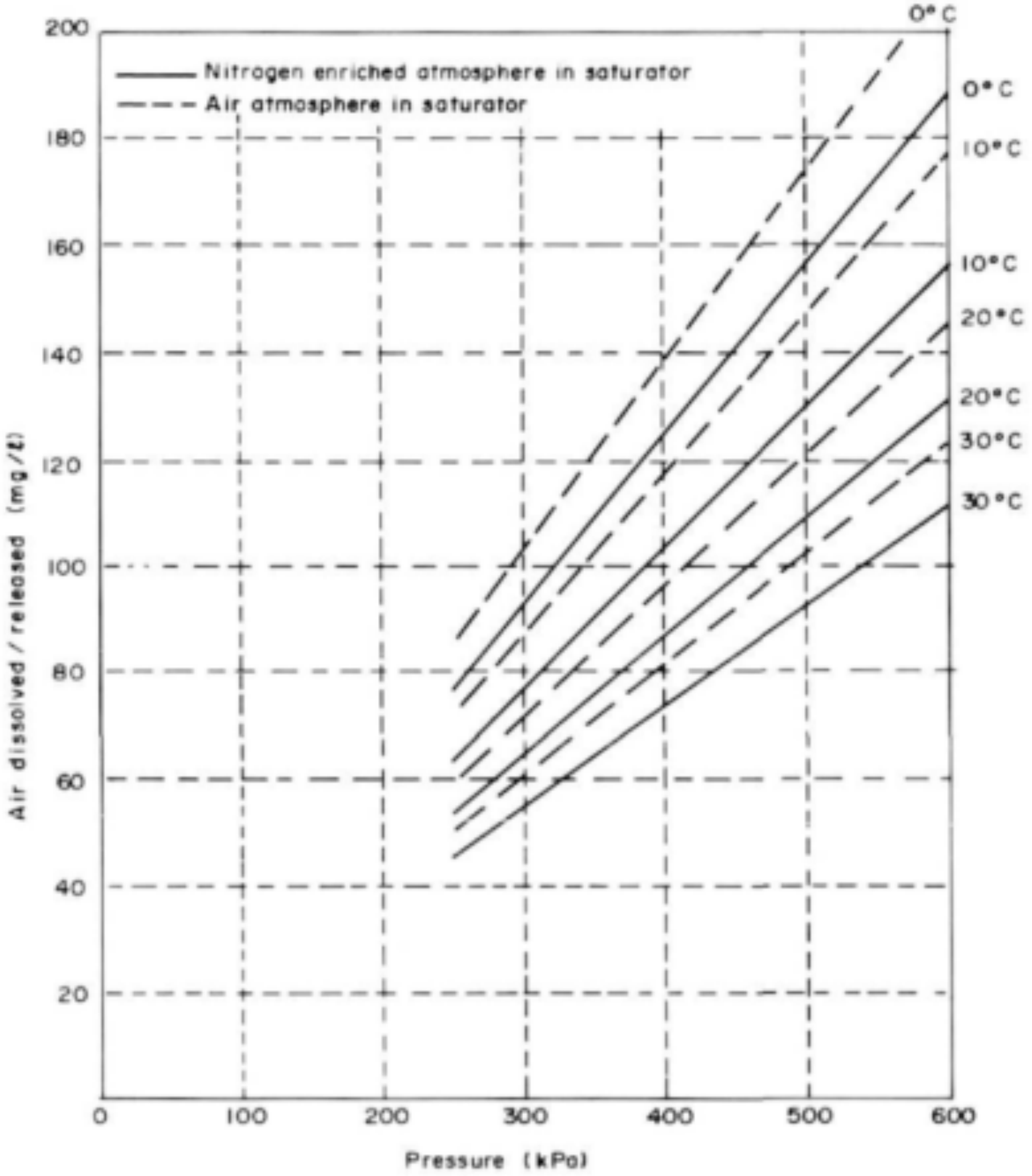


Figure 11.3

Maximum air dissolution in water as a function of pressure and temperature (from Rykaart, 1991)

mass ratio a_s which is essentially applicable to thickening applications.

Different air measurement techniques have been described in the literature:

- **Batch measurement of air precipitated from solution**. A sophisticated method has been described (Packham & Richards, 1975a) which samples the pressurised water **before** pressure release.
- **Continuous measurement of air precipitated from solution**. If an air precipitation vessel is connected to a saturator and steady state is achieved, the precipitated

air will vent off at a fixed rate (Bratby & Marais, 1975b).

- **Measurement of oxygen concentration by electrode**. Direct measurement of the oxygen concentration will enable calculation of the air concentration if the oxygen concentration in air is assumed.

11.1.3.4 Calculation of theoretical solubility

The theoretical solubility of air can be expressed in the form:

$$a_P = S \, . \, P \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \; 11.1$$

with a_P the excess air concentration in mg/ℓ, P the gauge pressure in kPa and S the solubility constant in mg/ℓ.kPa. The following values for S had been reported at 20°C (adapted from Rykaart, 1991) :

S_{20} = 0,242 for atmospheric air
S_{20} = 0,219 for saturator air

For temperatures other than 20°C, the solubility constant in Equation 2.1 can be adapted with the following relation (from Rykaart, 1991):

$$S_T = S_{20} \cdot \left(\frac{293}{273 + T} \right) . \, 10^{-500 \left(\frac{1}{293} - \frac{1}{273 + T} \right)} \ldots 11.2$$

with T the water temperature in °C.

11.1.3.5 Recycle percentage

There are two pressurisation options, namely full-stream or part-stream.

The usual option is part-stream DAF, where a fraction of the treated water is recycled to a saturator which is operated under much higher pressure. The air requirements for DAF can be met by either recycling a small percentage under very high pressure, or by recycling a larger percentage under reduced pressure.

The recycle ratio r can be calculated in a rational manner for any given combination of variables. For clarification applications, where the air requirement is expressed as a mass concentration per volume, the relationship is derived as:

$$r = \frac{C}{S_T \, . \, P \, . \, \eta_s} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \; 11.3$$

where:

r = recycle ratio dimensionless
S_T = air solubility constant, mg/ℓ.kPa
P = saturator gauge pressure, kPa
η_s = saturator air transfer efficiency, dimensionless
C = required air concentration, mg/ℓ

The recycle ratio for clarification usually varies from 6% to 12% of the raw water flow rate.

11.1.3.6 Saturator pressure

Practical saturator pressures for part-stream recycle are typically between 350 kPa and 600 kPa gauge pressure. Two considerations come into play when the actual saturation pressure has to be fixed. The first consideration deals with the efficiency of the saturator. It was shown (Bratby & Marais, 1975b) that the saturation efficiency of packed saturators drops off sharply below a pressure of approximately 300 kPa. This does not pose a significant restriction on saturator design or operation, as the vast majority of saturators are operated at pressures higher than 300 kPa.

A second, more nebulous relationship, is claimed between the saturator pressure and the diameter of the bubbles produced in the reaction zone. There is no consistency in the reports from the literature.

11.1.3.7 Packed saturators

Packed saturators are vertically mounted cylindrical pressure vessels which are partially filled with packing material. The packing material consists of proprietary plastic packing pieces of about 25 to 50 mm in diameter, and these are randomly dumped during installation. The flow enters the saturator from the top through a flow distributor and is collected below or just above the packing support plate.

There are four saturator design parameters of importance, besides the saturator pressure, i.e. liquid loading rate, packing depth, packing size and packing type.

Under conditions of appropriate liquid loading rate and packing depth, packed saturators have been found to be highly efficient. Values of 90% and higher are commonly reported.

11.1.3.8 Unpacked saturators

Unpacked saturators are cylindrical pressure vessels which are mounted vertically or horizontally. The increased interfacial area is obtained in one of two ways. The first method is to spray the water into the saturator in the form of small droplets. The second method is to continuously distribute the saturator air through the water with an internal recycling system driving an air eductor. These configurations are illustrated in Figure 11.4.

The spray saturator is not very efficient. A thorough evaluation of this type of saturator showed it to be 60% to 70% as effective as packed saturators (Zabel, 1978). To equal the air transfer rate of packed saturators, unpacked saturators have to be operated at pressures 100 kPa to 200 kPa higher than packed saturators (Zabel, 1978).

11.1.3.9 Saturator control

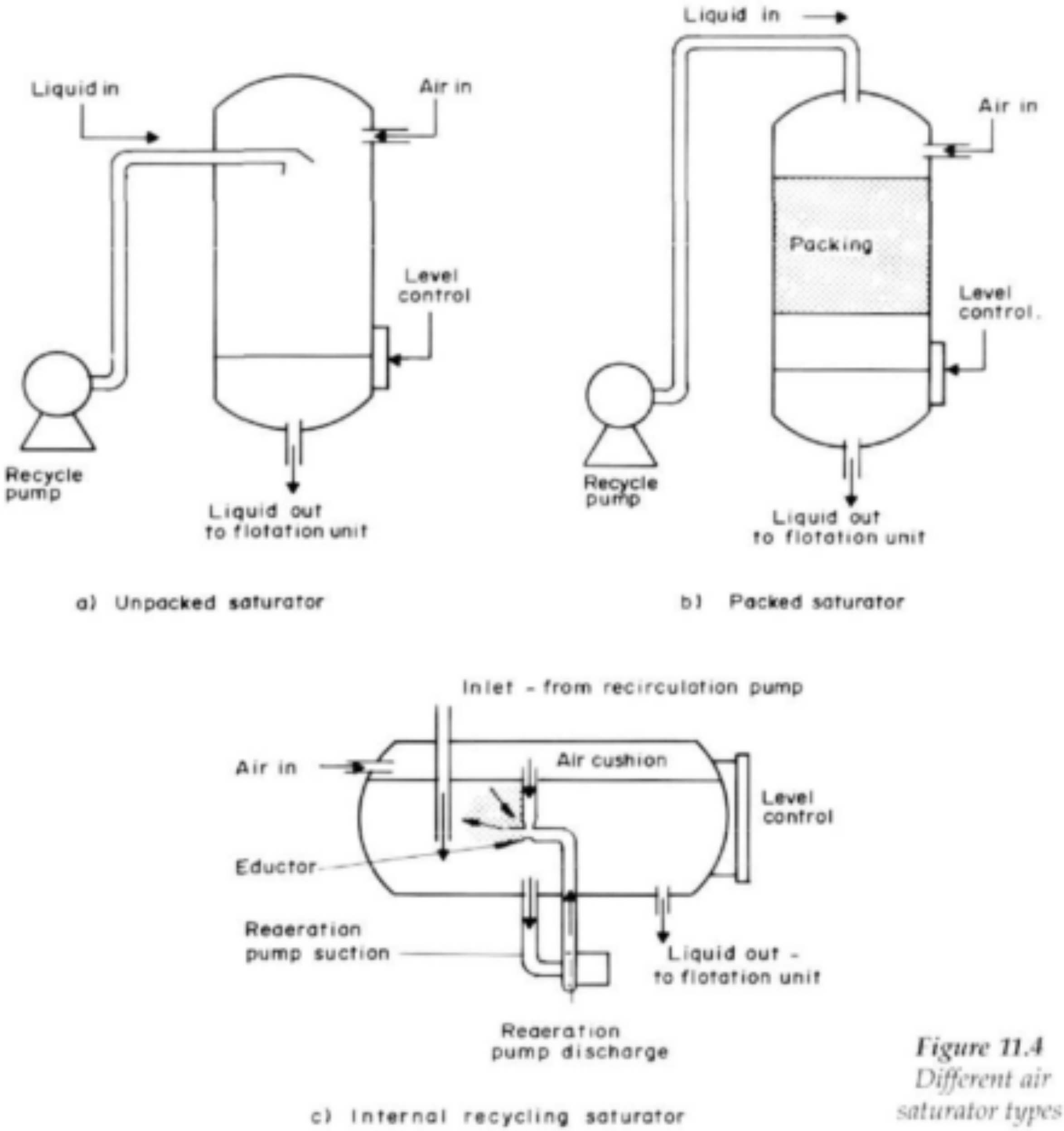
There are three external connections to any saturator, namely the recycle inlet, the recycle outlet and the air inlet. It is required to maintain an air cushion inside the saturator; in other words, the water level in the saturator must be controlled. This can be done in a number of ways:

- The recycle inlet and outlet are uncontrolled, which means that a constant flow rate is maintained. The water level is then controlled between a maximum and minimum level by simply closing the air inlet when the minimum level is reached, and by opening it when the maximum level is reached. The air injection rate has to be calculated on the basis of saturator pressure, saturator efficiency and recycle flow rate.

- The air is supplied by a constant pressure source, but otherwise uncontrolled. The water level is then maintained by controlling the recycle outlet. If the water level drops too low, the outlet is throttled until the level rises again, etc. This has the disadvantage of not having a constant recycle rate, but a recycle rate fluctuating according to the water level in the saturator.

- The air is supplied at a constant rate, and the water level is maintained by modulating the recycle flow rate into the saturator. If the level drops too low, more water is directed towards the saturator, and *vice versa*. This system has the advantage of being able to set a predetermined air injection rate at any time, but also suffers from a fluctuating recycle rate.

In all cases, it is recommended that an external sight glass be provided.



a) Unpacked saturator

b) Packed saturator

c) Internal recycling saturator

Figure 11.4
Different air saturator types

11.1.3.10 Injection nozzles

The aim of the injection nozzle(s) is to release the recycle stream into the reaction zone, while at the same time reducing the recycle pressure in such a way as to generate microbubbles of uniform size. A few practical suggestions appear in the literature:

- The pressure must be reduced as close as possible to the point of dissolved air injection into the process stream (Zabel, 1978).
- The pressure reduction must occur instantaneously to avoid any premature partial reduction of pressure (Van Vuuren & Offringa, 1985; Williams & Van Vuuren, 1984).

- The bubble formation can be improved by releasing the pressure through a number of small holes, rather than through a single injection point (Vosloo *et al.*, 1985).
- The bubble size distribution can be improved by letting the high-velocity jets impinge on a solid surface (Vosloo *et al.*, 1985).
- A shroud around the jet results in more satisfactory bubble formation.

A nozzle developed by the NIWR has a ring of orifices at the end of a blanked-off pipe, with a cup-shaped orifice cover directly outside the ring of orifices (Figure 11.9).

An injection nozzle passing turbulent flow must obey the fundamental hydraulic relationship:

$$\frac{r \, . \, Q}{n} = C_d \, . \, A_n \, . \, \sqrt{2.g.h_n} \quad \text{.......................} \; 11.2$$

with:

A_n = nozzle opening(s) (m^2)
C_d = nozzle discharge coefficient (-)
h_n = head loss across injection nozzle (m)
n = number of orifices per nozzle
$r \, . \, Q$ = recycle flow rate (m^3/s)

With adjustable nozzles, C_d and A_n are variable and the operator can thus manipulate the ratio between flow rate and head loss. With fixed nozzles, this is not possible and the design must be precisely done.

11.1.4 FLOTATION ZONE

The flotation zone is essentially for phase separation which includes air requirements, hydraulic loading, zone configuration, etc.

11.1.4.1 Air requirements

Many discussions on the fundamentals of flotation start off by considering how much air is required to counteract the gravitational force on the solids in the water.

A number of studies reported widely varying air efficiencies. When clarifying activated sludge, the air efficiency is between 2,5% and 25% while it is as low as 1% when clarifying slightly turbid river water (Kitchener & Gochin, 1981). A clear trend is evident from the data presented, namely that air is less efficient in dilute suspensions than concentrated suspensions. For most clarification applications, the air requirement seems to be in the range of 5 mg/ℓ to 10 mg/ℓ, based on the influent flow (Q).

11.1.4.2 Hydraulic loading

The effluent quality is mainly determined by the hydraulic loading, (water downflow/particle upflow) whereas the float layer concentration is mainly determined by the solids loading. For clarification of an effluent already low in solids concentration, the design is thus mainly based on the hydraulic loading, which has to be set lower than the limiting hydraulic loading.

For clarification, the hydraulic loading practically varies between about 5 m/h and 15 m/h. In one study, an upper limit of 12 m/h was reported above which performance deteriorated, regardless of how much air was applied.

In cases where high levels of inorganic turbidity are experienced, DAF performance will deteriorate during high turbidity (Conference on, 1977). In such cases, it will become necessary to employ pre-sedimentation ahead of flotation.

TABLE 11.1
COMPARISON OF FLOTATION TANK SHAPES

Circular	Rectangular
Decreasing velocity towards outer edge Simpler rake arm mechanism Cheaper - more area per unit wall length Lower outlet weir loading Bottom scrapers are cheap (thickening)*	Easier housed inside buildings Multiple units cheaper due to shared walls Hoppered bottom eliminates bottom scraper (thickening)* Prefabricated units can be transported Easier introduction of water
* Especially for thickening	

11.1.4.3 Flotation zone configuration

Once the hydraulic loading is fixed, the flotation tank area is determined by the design flow rate (including recycle) and the number of flotation units. The first choice then lies between rectangular or circular units. The advantages of both types are summarised in Table 11.1 (Everett, 1983; Bennet, 1988).

For clarification of algal water, it was suggested that circular tanks are used for smaller applications, and rectangular tanks with an L/W ratio of 2,5 for larger applications (De Wet, 1980). Of practical interest is the fact that single scraper arms in rectangular tanks have a maximum span length of about 3 m, which does restrict the choice somewhat as far as tank width is concerned (Zabel & Melbourne, 1980).

Two recommendations on the maximum design flow for a single flotation tank were found, namely 420 m^3/h (De Wet, 1980) and 750 m^3/h (Zabel & Melbourne, 1980). Above these rates, multiple tanks are unavoidable. A tank area of 50 to 60 m^2 was proposed as the upper limit of economical design (Conference on, 1977 : 401-406).

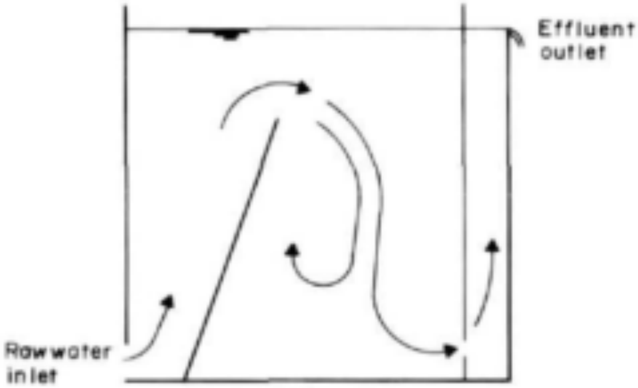
The minimum side depth required for a flotation tank is an important cost parameter. At pilot trials with an eutrophic surface water, the turbidity was vertically tracked from the top (Bernstein *et al.*, 1985). The turbidity decreased in a downward direction up to 900 mm from the top. Below this level, the turbidity stayed constant. From a clarification point of view, it would thus seem that a minimum of 1 m is required for satisfactory phase separation of eutrophic waters.

11.1.4.4 Flotation outlet

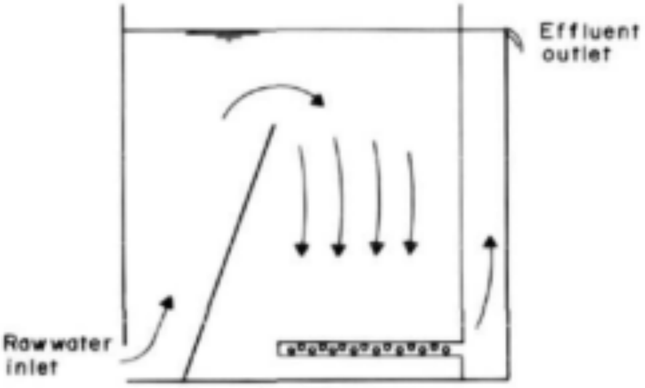
The design of the outlet was shown to have a marked effect on the performance of the flotation zone (De Wet, 1980). With a vertical collection system, superior results were achieved compared to a radical draw-off (see Figure 11.5).

Provision for vertical draw-off, particularly for rectangular tanks, appears to be recommendable.

The use of a roof over the flotation zone is generally recommended. A roof will eliminate float layer disruption by rain and by wind.



Recirculation due to single tank outlet position



Even downflow due to collection manifold

Figure 11.5
Flow patterns induced by different collection systems (after De Wet, 1981)

11.1.5. FLOAT LAYER REMOVAL

For float layer removal various options need to be considered with due consideration of float layer stability.

11.1.5.1 Float layer stability

Two different phenomena are understood under the general term of float stability. The first describes the stability of the float layer under normal operating conditions. In the case of water with low turbidity and high colour, the float layer became unstable after 30 minutes, while a turbid river water allowed float layer accumulation of up to 48 hours without a deterioration in effluent quality (Zabel, 1978). The second type of stability pertains to the stability of the float layer when the feed and recycle are interrupted, i.e. that no more air and particles are added to the float layer. This is not so much a design as an operational consideration, for it determines the DAF start-up and shutdown procedures.

11.1.5.2 Removal options

The simplest option is to drain the tank from time to time, thereby scouring the float layer with the tank contents. Most DAF applications, however, have special provision for the continuous or intermittent removal of the float layer. The two options are flushing, where the float layer is drained off the top by lifting the water level relative to an overflow weir, or by scraping the top of the float layer into a sludge collection trough. Flushing produces a very dilute sludge with fairly high water loss, while scraping achieves the opposite.

Intermittent float layer flushing can be achieved in one of two ways. The float layer can be flushed off the top by closing the outlet of the flotation tank, which causes the water level to rise up to an overflow weir. The float layer can also be flushed by lowering one side of the tank adjacent to the overflow channel. It was reported that flushing is the usual way of removing the float layer in Swedish water treatment plants (Rosen & Morse, 1976). Continuous flushing, however, results in higher water losses (Zabel, 1978).

11.1.5.2.1 Float layer scraping

A simple mass balance on the solids across the flotation zone yields the following expression:

$$CF = \frac{Q}{q} \cdot (SS_{in} - SS_{out}) \cdot 10^{-4} \qquad 11.3$$

with

Q = flow rate into the flotation zone, and
q = the rate at which sludge is removed.

Equation 11.5 shows that the float solids concentration C_F is inversely proportional to the sludge removal rate q. With scraping, C_F values of up to 5% are attainable.

The most important drawback of float layer scraping is the deterioration of the effluent while the scraping is being done. There is also a danger with intermittent scraping if the float layer is left too long between scrapings. The water quality may deteriorate as the accumulation time increases. In one case, 1 hour was already too long (Stock, 1976). In the case of combined flotation/filtration, no effect on filtered water quality was found even after 40 h (Rosen & Morse, 1976).

One way of reducing the effect of scraping is to minimise the length of travel of the scraper blades. This is done with partial scraping, where the scraper blades do not travel the full length of the tank, but only a short length of the tank in the vicinity of the sludge trough. In the case of a soft, coloured water where the float layer starts to break up after 30 minutes, scraping has to be carried out over the full length to be successful (Zabel, 1978; Zabel, 1985). The speed at which the scraper blades travel, has an effect on the removal of the float layer. For algal water, a scraping speed of 60 m/h is recommended (De Wet, 1980).

11.1.5.2.2 Float stabilising grids

To prevent the disruption of the float layer during scraping and to prevent "rolling" of the float layer, a fixed grid is sometimes placed at the surface of the flotation tank. A prominent international manufacturer uses a sludge grid 100 mm deep, with the facility to adjust the water level in the tank relative to the grid to attain the desired float solids concentration. This level is generally found to be 5 to 10 mm below the upper surface of the grid (Conference on, 1977, 68-76). Care should be taken to design these grids with sufficient structural rigidity to prevent upward deflection when the highly buoyant float layer pushes the grid from below.

11.1.6 COMBINED FLOTATION/ FILTRATION

For clarification purposes, the flotation and filter functions are often combined into a single reactor with flotation being carried out on top of the filter bed. The process is commonly known in SA as the DAFF process (dissolved air flotation filtration) and is also known as a FLOFILTER in Europe and as the SANDFLOAT system in the USA.

11.1.6.1 Advantages

The advantages of the combined flotation and filter functions over separate reactors are twofold. Firstly, there is the obvious cost saving by only having to build one structure instead of two. Secondly, a saving is achieved if the designer opts to leave out the float layer collection mechanism altogether.

DAFF is also used to upgrade conventional treatment plants. A common problem in SA is the deterioration of raw water sources resulting in short filter runs. In such cases, DAFF will extend the filter runs by capturing a fraction of the suspended solids prior to filtration. Monitoring of a full-scale DAFF plant indicated that about 80% of the incoming solids is floated in a DAFF reactor, leaving 20% to be filtered and increasing filter in lengths by 63% (Haarhoff & Fouche, 1989).

11.1.6.2 Air-binding and head loss development

A principal concern about DAFF is the possibility of drawing those minute air bubbles (with rise rate smaller than the filtration rate) into the sand bed where they will coalesce into larger bubbles which will ultimately block the interstitial pores. This phenomenon is known as air binding and will lead to a rapid increase in the head loss development during the filter run.

There is an indication that the degree of air binding may be dependent on the saturation pressure, particularly where full stream aeration at relatively low pressure is applied.

11.1.7 FULL-SCALE APPLICATIONS

DAF applications for the clarification of surface waters only gained momentum in SA during the late seventies; the reason being the increased eutrophication of several major impoundments and associated problems encountered with conventional treatment such as short filter runs, taste and odour and THM formation.

Several existing plants were upgraded and new plants constructed during the eighties all with a high degree of success. Criteria derived from a survey of these plants are included in Subsection 11.2 below.

A large-scale DAF plant with nominal capacity of 150 Mℓ/d was commissioned during the early eighties. This plant was designed to treat eutrophic water from a lake in the Northern Natal coastal region to high quality requirements for use mainly in a fine paper mill (Bernstein *et al.*, 1985). These studies showed that DAF had its limitations when inorganic turbidities exceeded a critical value, in this instance about 80 NTU.

11.2 DESIGN CONSIDERATIONS AND CRITERIA

The empirical guidelines and recommended criteria contained in this section are based on the literature cited of South African DAF plants, and personal communications with designers and operators.

Before the designer can start with the detailed design of a flotation plant, it is necessary to first consider the overall process design for the entire treatment facility. Typical decisions that could be made at this point, are:

- From the design flow, the number of flotation units can be determined.
- If high raw water turbidity above the critical turbidity, where flotation fails (a variable that has to be tested - normally about 60 - 100 NTU), is a possibility, provision should be made for pre-sedimentation. If so, a second facility for dosing, mixing and flocculation between pre-sedimentation and flotation should be provided.
- The designer also has to compile and interpret all available data which could impact on the design of the flotation plant. Examples are the variation in flow about the mean anticipated mode of operation (continuous or stop-start), anticipated raw water quality, and requirements for the flotation effluent and float layer concentration.

11.2.1 REACTION ZONE

The configuration of the reaction zone depends in large measure on the shape of the flotation unit. As a general rule, one would place the reaction zone in the centre of a circular tank, but on the side of a rectangular tank. In both cases, the designer further has

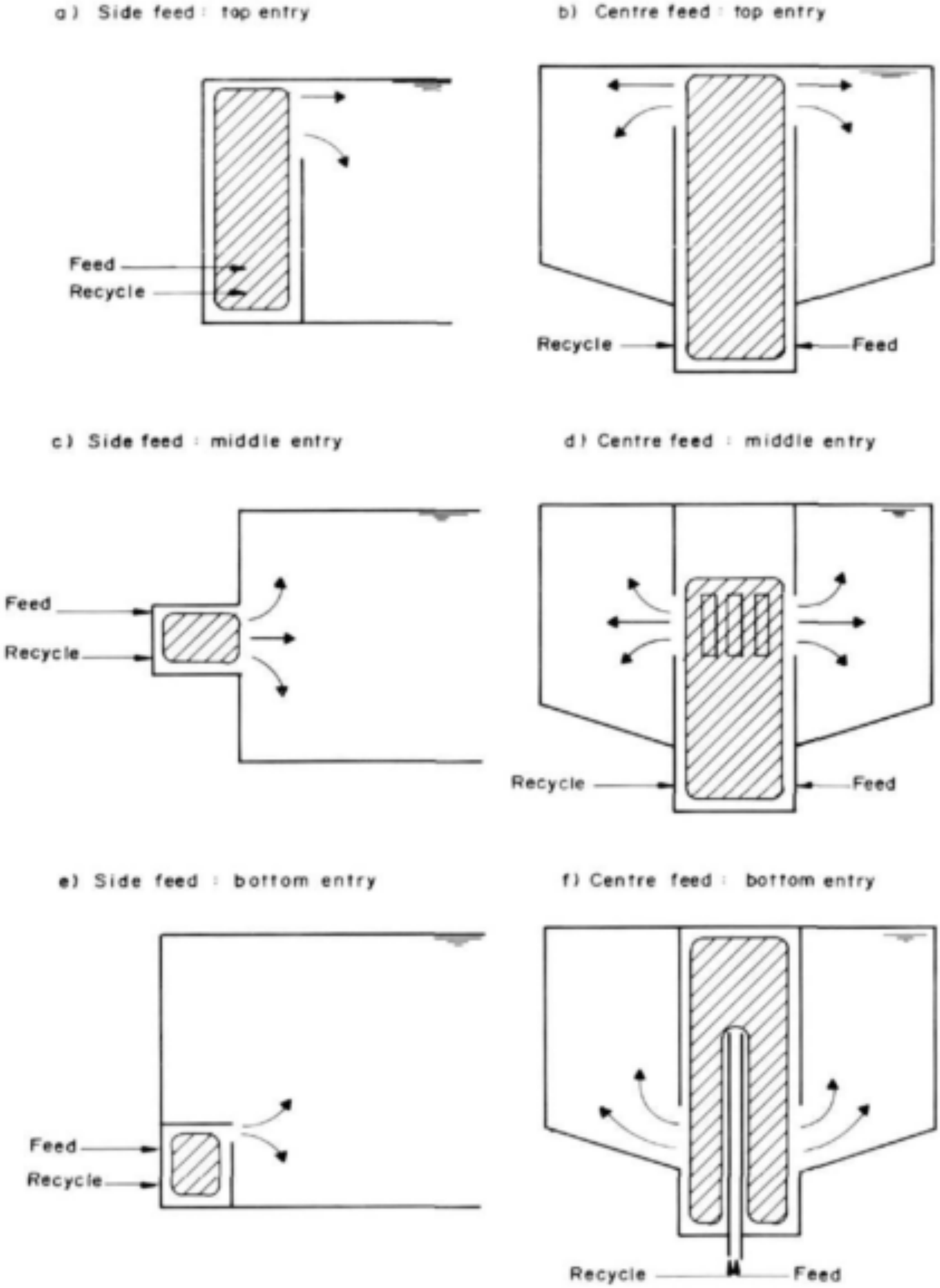


Figure 11.6
Reaction zone configurations of surveyed plants

Table 11.2
Design Guidelines for Reaction Zone*

Clarification		Minimum	Maximum
Residence time	s	60	240
Hydraulic loading	m/h	40	100

* based on total flow, i.e. feed plus recycle

the option of placing the reaction zone outlet at the top, middle or bottom of the tank. These variations are shown in Figure 11.6. During a South African survey, all the different combinations of tank shape and reaction zone positions were encountered, and no obvious advantages amongst the plants could be ascribed to any particular one. It would therefore appear that the reaction zone configuration could be based on practical preference, provided that the rest of the reaction zone design guidelines are adhered to.

A second factor to consider is that of scale. In general, it is easier to "spread" the reaction zone over multiple injection points when rectangular flotation tanks are used. In this case, the reaction zone could take the shape of a trough alongside one of the edges, and raw water and recycle could be introduced through a manifold running in the trough. With circular tanks, the reaction zone is confined to the centre of the tank and it may be more difficult to avoid turbulence.

Analogous to conventional flocculation, it is postulated that two factors determine the reaction efficiency, namely the length of the mixing period and the mixing intensity. The length of the mixing period is simply calculated as the average retention time in the reaction zone. This calculation is based on the total flow, i.e. raw water as well as recycle. It is more difficult to quantify the mixing intensity, as there is no measurable energy input into the reaction zone. For these guidelines, the mixing intensity is crudely approximated by the mean flowthrough velocity, based on the largest cross-sectional area in the reaction zone. The recommended reaction zone parameters are summarised in Table 11.2.

11.2.2 Chemical pretreatment

There are significant similarities between the chemical pretreatment for flotation and the chemical pretreatment for conventional settling. It may be advantageous to add a polymeric coagulant aid from the viewpoint of float layer stability and compaction.

Indications are that flotation has less stringent flocculation requirements than conventional settling. Provided that the water is properly dosed, only a minimal amount of flocculation is required to get the floc to a certain minimum size. The flocculation requirements for surface water clarification are summarised in Table 11.3.

Table 11.3
Flocculation Requirements for Clarification

		Minimum	Maximum
Velocity gradient (G)	s^{-1}	50	120
Flocculation time	min	4	15^{**}
Product GT	-	40 000	80 000

**or longer if not for eutrophic water or municipal wastewater reclamation

A number of points are evident from the SA survey of clarification plants:

- The primary coagulant in all cases is either ferric chloride or aluminium sulphate. The choice between the two is predominantly made on economic grounds.
- Coagulants are dispersed by mechanical mixers in only two out of fourteen plants.
- Despite the simplicity and good performance of plug-flow flocculators (pipes and baffled channels), they are only used at three out of fourteen plants. The other plants use back-mix reactors in the form of flow-through turbulent tanks or externally stirred tanks.
- The flocculation time shows a wide scatter with five minutes as minimum.
- The mixing intensity, on the average, is fairly high with only one plant below G = 80 s^{-1}.
- The Gt-product varies, with one exception, within the range 3 x 10^4 to 8 x 10^4.

11.2.3 BUBBLE PRODUCTION SYSTEM

The aspects of the system which are of interest are recycling, saturators, and injection nozzles.

11.2.3.1 Recycling

The designer must know the air requirements of the DAF application before the recycle system can be designed. Table 11.4 shows the range of values encountered in South Africa for the main variables.

The water for recycle could be drawn off directly from the bottom of the flotation tank, which is a simple, cheap option at those sites where there is no better water available. The other option is to use water after at least sand filtration to prevent fouling of the recycle system.

TABLE 11.4
DESIGN GUIDELINES FOR RECYCLE SYSTEMS

Clarification		**Minimum**	**Maximum**
Air requirement C_{air}	mg/ℓ	6,0	8,0
Recycle ratio r*	-	0,06	0,10
Saturation pressure P	kPa	400	600

* should be calculated for every case

A good case can be made out for fitting the recycle pumps with variable speed drives. As a large proportion of DAF running costs is ascribed to energy consumption, the operators can therefore trim the air dosage to the minimum.

11.2.3.2 Saturators

The saturator efficiency η_s plays an important role in determining the recycle rate and is difficult to estimate. Empirical design guidelines for saturators make little reference to the efficiency achieved except to say that an unpacked saturator must be operated at a pressure of about 200 kPa higher than a packed saturator to achieve the same efficiency (Zabel, 1978). A conservative general estimate of 75% is suggested for packed saturators, and 50% to 60% for unpacked saturators without internal recycle. Recommended values for the main variables are shown in Table 11.5.

The designer needs to consider two important practical points. The first deals with the material of construction. Water with a high air concentration corrodes towards mild steel, and chrome steel should be considered. The second deals with the optimum size/number of saturators. The number of saturators is normally dictated by the site layout. The optimum size of saturators

TABLE 11.5
DESIGN GUIDELINES FOR SATURATORS

		Minimum	Maximum
Packed saturators			
Hydraulic loading V_s	m/h	50	80
Packing depth H_p	m	0,8	1,2
Water depth	%	15	25
Unpacked (no internal recycle)			
Retention time*	s	50	150
Water depth*	%	40	60

* tentative guidelines due to very few available data

would probably be decided on the basis of cost. The larger the saturator, the greater the care that should be taken to ensure even flow distribution and to prevent vortex formation at the outlet.

It is recommended that a level control system is used which results in an even recycle stream with constant air concentration. For this reason, it is better to regulate the air flow into the saturator than to regulate the water flow rate. It seems to be the simplest to keep the water flow rate constant and to simply stop/start the air flow to maintain the water level within the desired control band.

11.2.3.3 Injection nozzles

The injection nozzles have to match the saturator pressure and recycle flow exactly, two parameters which normally have been chosen by the time the nozzle selection or nozzle design has to be done. Two general nozzle types can be used, namely fixed nozzles or adjustable nozzles. With adjustable nozzles, the final setting of the nozzles can be done after commissioning to ensure the exact combination of recycle rate and pressure drop. With fixed nozzles, the designer must know the exact hydraulic characteristics in order to specify the correct number of nozzles. A few large nozzles for circular reactors and an array of smaller nozzles for longitudinal reactors need to be considered.

For the typical pressure drop across DAF injection nozzles, the water velocity through the nozzle orifice has to be in the region of about 20 m/s. At this velocity, the erosion by the water jet can be considerable if improper materials are specified for the nozzle, or if an unsuitable valve is adopted as an adjustable nozzle.

The designer has the option to design injection nozzles from first principles (see design example), but there are numerous rules of thumb to ensure bubbles of desired size and narrow distribution which makes nozzle design an art as much as a science. The other option is to revert to any of the numerous proprietary nozzle designs used by the major water treatment companies.

11.2.4 FLOTATION ZONE

The number of flotation units has to be determined first. This is mostly a practical consideration, which has to take into account the likely flow variation, space constraints on the site, modular layout of the treatment plant to facilitate phased construction, etc. For clarification, a maximum size of $750\ m^3/h$ is recommended.

The important parameters for clarification are the cross-flow velocity between reaction and flotation zones, the hydraulic loading in the flotation zone and the side depth of the flotation tank. These parameters, with recommendations, are shown in Table 11.6.

TABLE 11.6
DESIGN GUIDELINES FOR FLOTATION ZONE

Clarification		Minimum	Maximum
Cross-flow velocity	m/h	20	100
Hydraulic loading	m/h	5,0	11,0
Side depth	m	1,5	3,0

11.2.4.1 Outlet design

A poorly designed outlet could set up a recirculation pattern in the flotation zone. In larger flotation tanks, it is recommended that multiple draw-off points are provided which are evenly distributed over the entire flotation area. This particularly applies to rectangular tanks which have a shorter draw-off length in comparison with circular tanks.

The water level in the flotation tank will need to be frequently manipulated. At small plants, this can be conveniently done with a pipe overflow with an adjustable sleeve. At larger plants, it may be necessary to use an adjustable rectangular weir.

11.2.4.2 Float layer removal

The different options for float layer removal are float layer flushing and float layer scraping with or without a float layer stabilising grid. The choice among these systems is determined by the sludge management system adopted for the entire treatment plant.

The survey results included only two float layer stabilising grids, which have grid openings of 150 mm x 150 mm and 200 mm x 200 mm respectively. These work satisfactorily and provide some guideline to designers. The depth of the grids and the thickness and material of the plate must be sufficient to withstand the float layer uplift force to prevent jamming of the scraper. The literature review indicated that the grids work best if the water level in the tank is maintained at about 5 to 10 mm below the top of the grids. The designer should allow for water level manipulation around this level.

11.2.4.3 Float layer scraping

The three variables to be fixed by the designer are:

- the number of scrapers, or scraper spacing,
- the speed of the scrapers, and
- whether to scrape continuously or intermittently.

There are also very important practical scraper details in terms of shape, scraper depth, scraping angle, material of construction, beach plates and discharge troughs will affect the scraping efficiency. These aspects, however, fall outside the scope of this manual.

A mass balance equation for the solids in the flotation zone was given as Equation 11.5, with which the number of scrapers and scraping speed can be estimated. In the case of clarification, where sludge volumes are relatively small, operation is usually intermittent. Here designers should provide timers for the on-off cycle which can be easily adjustable by the operators to control the depth of the float layer. Table 11.7 provides some guidelines.

The results of the survey reflect a similar philosophy amongst designers of South African plants. Of the 12 clarification plants (excluding the DAFF plants where bottom scraping is not possible), only two make provision for bottom scraping.

TABLE 11.7
DESIGN GUIDELINES FOR FLOAT SCRAPING EQUIPMENT

		Minimum	Maximum
Radial scrapers (for circular tanks)			
Tip speed	m/h	50	200
Linear scrapers (rectangular tanks)			
Scraper speed	m/h	20	100

11.3 DESIGN EXAMPLES

The purpose of this section is to illustrate the application of the theoretical principles as well as the empirical data and recommendations covered earlier.

11.3.1 INITIAL PARAMETERS

A DAF plant must be designed to clarify chemically flocculated eutrophic surface water and the maximum suspended solids concentration, including treatment chemicals, is 20 mg/ℓ. The effluent will be filtered and chlorinated for potable use. The maximum flow rate will be 800 m³/h, and the maximum temperature is expected to be 24°C.

The sludge has to be landfilled and float layer stabilisation grids will be required. The recycle will be drawn from the potable ground storage on site and will have to be repumped through the recycle system. Choose two rectangular DAF units, each with a production capacity of 400 m³/h. A typical design sequence is shown in Table 11.8.

11.3.2 BUBBLE PRODUCTION SYSTEM

The aspects to be considered include cycle flow rate, saturation, aeration/flotation zone and typical nozzle design.

11.3.2.1 Recycle flow rate

This is a typical clarification application where the air requirement is expressed as the volumetric concentration based on the raw water flow. Choose a minimum air concentration of 8 mg/ℓ. The solubility constant for saturator air at 24°C is:

$$S_{24} = 0{,}219 \,.\left(\frac{293}{273+24}\right) .\, 10^{-500\left(\frac{1}{293}-\frac{1}{273+24}\right)}$$

$$= 0{,}205 \text{ mg}/\ell\text{kPa}$$

Choose a maximum saturator pressure of 500 kPa. As the recycle will be potable water, a packed saturator with assumed efficiency of 75% will be used. The recycle ratio is thus calculated with Equation 11.3 as:

$$r = \frac{8}{0{,}205 \,.\, 500 \,.\, 0{,}75} = 0{,}104$$

The recycle flow rate will therefore be:

$$r \,.\, Q = 0{,}104 \,.\, 800 = 83 \text{ m}^3/\text{h (both tanks)}$$

and the total flow rate:

$$Q.\,(1 + r) = 800 + 83 = 883 \text{ m}^3/\text{h}$$

11.3.2.2 Saturator

Choose one saturator with a maximum hydraulic loading of 80 m/h. The saturator cross-sectional area is:

$$A_s = \frac{83}{80} = 1{,}04 \text{ m}^2$$

TABLE 11.8
TYPICAL DESIGN SEQUENCE

Background information, projections and assumptions	
Design flow rate per tank:	400 m^3/h
Suspended solids concentration and characteristics:	Algae/ferric (20 mg/ℓ)
Requirements for float layer concentration and subnatant quality:	C_F(5%), FTU (<5)
Number and shape of units :	2 Rectangular
Bubble production system	
Air requirements:	8 mg/ℓ
Saturator pressure:	500 kPa (max), efficiency 75%
Recycle ratio:	± 10%
Saturator sizing:	One packed saturator, hydraulic load 80 m/h
Recycle pumps and air compressors:	
Injection nozzles:	Single nozzle with multiple orifices
Reaction/flotation zone	
Shape/sizing of reaction zone:	Rectangular, 60 m/h
Shape/sizing of flotation zone:	Rectangular 8 m/h
Side depth:	2.2 m
Cross-flow velocity:	100 m/h
Draw-off system:	Bottom manifold collection
Level control:	Effluent weir (30 cm adjustable)
Float layer removal	
Removal method:	Surface scraping
Estimated volume:	7,68 m^3/d
Sizing/speed of scraping (if applicable)	
Sludge discharge system:	landfill
Hoppers/bottom scraping:	Not of relevance

and the saturator diameter:

$$D_s = \sqrt{\frac{4\,.\,1{,}04}{\pi}} = 1{,}15 \text{ m}$$

Choose a packing depth of 1 000 mm with plastic packing between 25 mm and 50 mm, and maintain the water depth just above the bottom of the packing.

11.3.2.3 Reaction/flotation zone

Choose a rectangular flotation tank with a length/width ratio of 2,0, with a layout as shown in Figure 11.7.

The exact boundaries of the reaction zone are unknown and they have to be arbitrarily assigned. With a tank configuration as shown in Figure 11.7, the reaction zone (for the sake

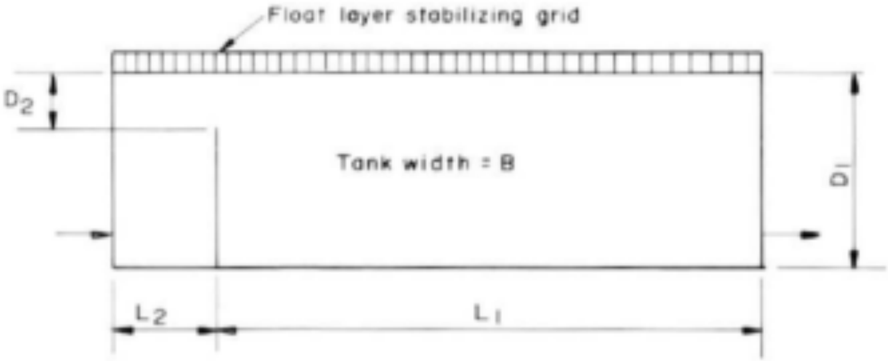


Figure 11.7
Schematic section of nozzle for which design example is given, based on the NIWR-nozzle principle with enclosing cup (not to scale)

of the plant survey and the design recommendations) was assumed to enclose the water volume from the floor level all the way to the underside of the grid. When applying these recommendations, the same definition therefore has to be used.

11.3.2.4 Reaction /flotation zone

Choose the hydraulic loading for the reaction zone 60 m/h, the cross-flow velocity 100 m/h and the hydraulic loading for the flotation zone 8 m/h. Choose the side depth for flotation to be 2 200 mm from the bottom of the tank to the underside of the float layer stabilising grid. The area of the flotation zone is therefore (there are two tanks) :

$$A_f = \frac{883}{2 \,.\, 8{,}0} = 55 \text{ m}^2 \text{ /tank}$$

The L/W ratio = 2. The dimensions are thus:

$$B = \sqrt{\frac{55}{2}} = 5{,}24 \text{ m}$$

and

$$L_f = 2 \,.\, 5{,}24 = 10{,}5 \text{ m}$$

The clearance between the top of the division wall and the underside of the float layer stabilising grid is determined by the cross-flow velocity:

$$D_c = \frac{883}{2 \,.\, 100 \,.\, 5{,}24} = 0{,}84 \text{ m}$$

The reaction zone width is:

$$L_r = \frac{883}{2 \,.\, 60 \,.\, 5{,}24} = 1{,}40 \text{ m}$$

Check the retention time in the reaction zone:

$$t = \frac{2{,}2 \,.\, 5{,}24 \,.\, 1{,}40 \,.\, 60 \,.\, 2}{883} = 132 \text{ s}$$

11.3.2.4.1 Float layer removal

It was earlier determined that a float layer stabilising grid would be used. Choose grid openings of 200 mm by 150 mm, and make the grid 200 mm deep. Estimate the volume of sludge that must be removed under average operating conditions:

$$q = \frac{Q}{C_f} \,.\, (SS_{in} - SS_{out}) \,.\, 10^{-4}$$

With the assumptions that $C_f = 5\%$, $SS_{in} = 20$ and $SS_{out} = 0$:

$$q = \frac{800}{5} (20 - 0) \,.\, 10^{-4} = 0{,}32 \text{ m}^3\text{/h}$$

$$= 7{,}68 \text{ m}^3\text{/d} \quad \text{(2 tanks)}$$

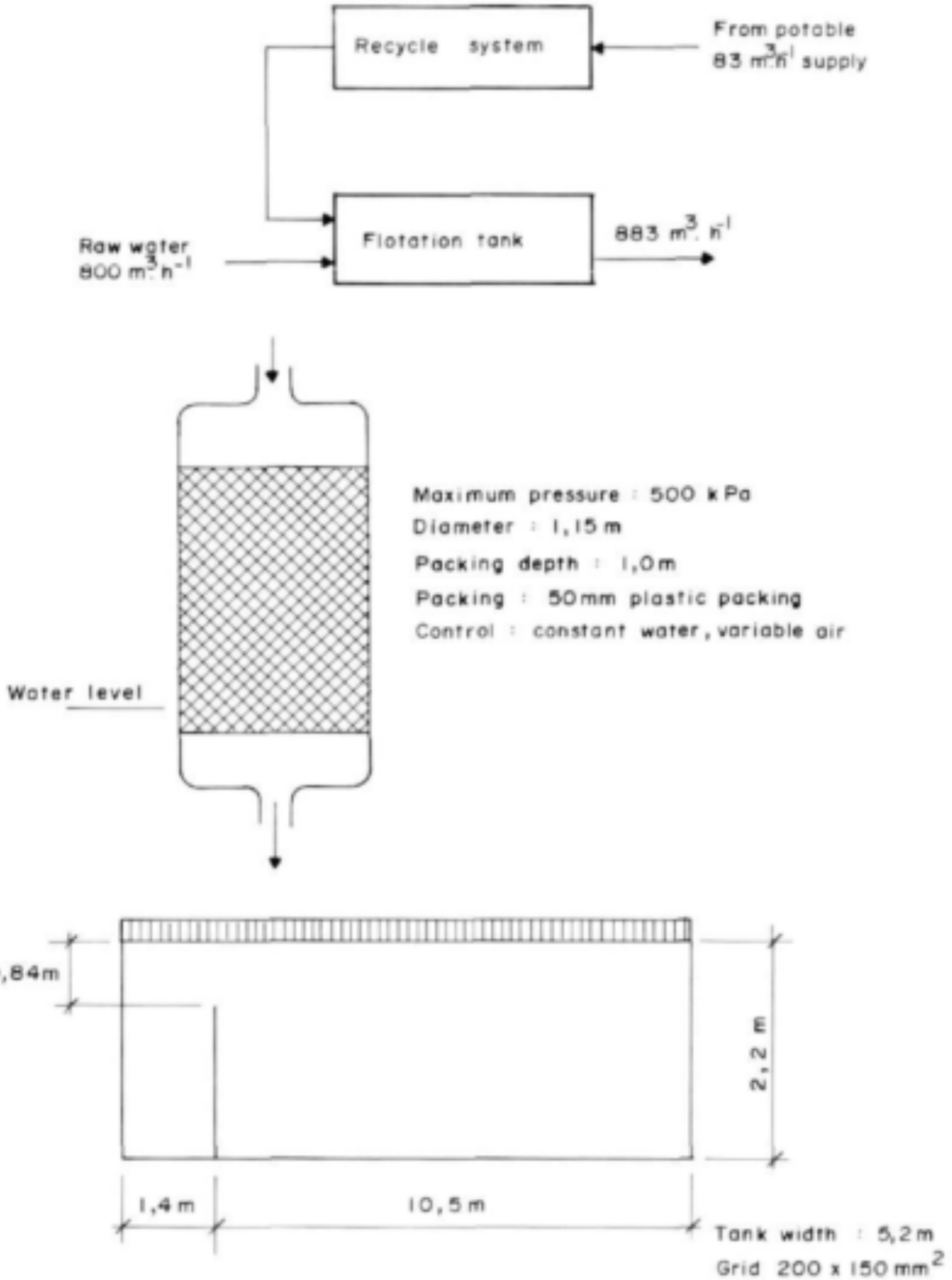


Figure 11.8
Summary sketches for the empirical design example for clarification

11.3.2.4.2 Summary sketches

The main design parameters are shown in Figure 11.8.

11.3.2.5 Typical nozzle design

The following illustration is for the design of a single nozzle with multiple orifices used at each of the DAF tanks as above.

In this example, the design raw water flow rate is 800 m³/h per tank. The recycle percentage was calculated at a maximum of 10.4% at a saturation pressure of 500 kPa. The correct combination of orifice diameter and number of orifices has to be determined.

- Typical discharge coefficients for orifices range from $C_d = 0{,}61$ to $0{,}64$. Select a conservative value of $C_d = 0{,}60$.

- For practical reasons an orifice diameter of 2 mm is selected. The cross-sectional area per orifice is thus:

$$\frac{\pi \,.\, 0{,}002^2}{4} = 3{,}14 \,.\, 10^{-6}\ \text{m}^2$$

- The recycle flow rate is calculated from the known recycle percentage and raw water flow rate:

$$\frac{0{,}104 \,.\, 400}{3600} = 0{,}0116\ \text{m}^3/\text{s}$$

- The pressure drop across the injection nozzle should be about the same as the saturation pressure in a well-designed system, namely

$$\frac{500 \,.\, 10^3}{100 \,.\, 9{,}81} = 50{,}96\ \text{m head of water}$$

- The number of holes can be solved by setting up Equation 11.4:

$$\frac{0{,}0116}{n} = 0{,}6 \,.\, 3{,}14 \,.\, 10^{-6} .\, \sqrt{2 \,.\, 9{,}81 \,.\, 50{,}96}$$

from which

$$n = 194$$

A total number of 194 orifices each with a diameter 2,0 mm has to be provided. The design should therefore provide for 12 rows of 16 orifices as schematically shown in Figure 11.9.

Check velocity through holes

$$= \frac{0{,}0116}{194 \,.\, 3{,}14 \,.\, 10^{-6}} = 19{,}0\ \text{m/s}$$

11.3.2.5.1 Comment

For a simple nozzle geometry as this, textbook values for the discharge coefficient C_d may be successfully used. For more complicated geometries, for example where the orifice channels are not straight, or the outlets are tightly capped, or for adjustable nozzles, the discharge coefficient must be determined experimentally.

11.3.2.5.2 Effect of recycle flow rate variation

The designer has to provide a recycle system that will provide sufficient air under the worst possible combination of conditions, namely the maximum air requirement at the maximum flow rate at minimum air solubility (or highest water temperature). These conditions will very rarely be encountered in actual practice. Under average conditions it will very likely be viable to operate the recycle system at a reduced rate and/or reduced pressure to save energy costs. This section presents a typical example of such an analysis.

There are two variables that determine the

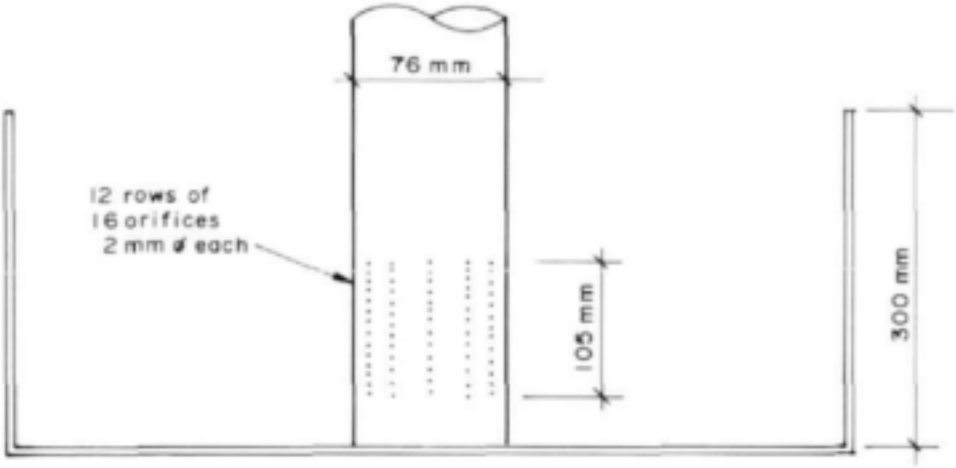


Figure 11.9
Schematic section of nozzle for which design example is given, based on the NIWR-nozzle principle with enclosing cup (not to scale)

actual air transfer, namely the saturator pressure (which determines the mass of air that can be forced into the recycle stream) and the recycle flow rate itself. These variables are mathematically linked as is shown in Equation 11.3. To manipulate the actual air transfer, either the saturator pressure, or the recycle rate, or both have to be adjusted. In the case of *adjustable* injection nozzles (for example when a valve is used), the adjustment is fairly trivial because the recycle flow rate and the saturator pressure can be independently adjusted (the recycle flow rate by pump speed control or by throttling, and the saturator pressure by adjustment of the injection nozzle). In the case of *fixed* injection nozzles, only the recycle flow rate can be adjusted, which makes the analysis more involved.

When the recycle flow rate is reduced, the saturator pressure is simultaneously reduced. Under normal conditions static head and pipe friction losses are negligible compared to the pressure drop across the injection nozzles. The first assumption is thus that the saturator gauge pressure is approximately equal to the pressure drop across the injection nozzles. As the flow through the recycle system is highly turbulent, the saturator pressure is proportional to the square of the recycle flow rate. If Equation 11.3 is set up for *maximum and minimum* air transfer conditions, the following relationships can be derived:

$$\frac{P_{min}}{P_{max}} = \left(\frac{C_{min}}{C_{max}} \cdot \frac{S_{min}}{S_{max}}\right)^{2/3} \quad \ldots\ldots\ldots\ldots\ldots\ldots 11.6$$

$$\frac{r_{min}}{r_{max}} = \left(\frac{C_{min}}{C_{max}} \cdot \frac{S_{min}}{S_{max}}\right)^{1/3} \quad \ldots\ldots\ldots\ldots\ldots\ldots 11.7$$

11.3.2.5.3 Example

The air requirement for clarification ranges from a minimum of 6 mg/ℓ to a maximum of 10 mg/ℓ. The raw water temperature range is from 10°C to 25°C . The plant operates at an a minimum of 70% of the maximum design flow of 800 m^3/h. The maximum saturator pressure is 550 kPa. If fixed air injection nozzles are used, determine a) the recycle flow rate under conditions of maximum air transfer, b) the saturator pressure under conditions of minimum air transfer, and c) the recycle flow rate under conditions of minimum air transfer. Therefore:

P_{max} = 550 kPa
Q_{min} = 0,70. Q_{max}
T_{max} = 25°C
T_{min} = 10°C
C_{max} = 10 mg/ℓ
C_{min} = 6 mg/ℓ

Calculate solubility constants first:

$$S_{max} = 0{,}219 \,.\, \frac{293}{273+10} \,.\, 10^{-500\left(\frac{1}{293}-\frac{1}{273+10}\right)}$$

$$= 0{,}261 \text{ mg}/\ell.\text{kPa}$$

$$S_{min} = 0{,}219 \,.\, \frac{293}{273+25} \,.\, 10^{-500\left(\frac{1}{293}-\frac{1}{273+25}\right)}$$

$$= 0{,}202 \text{ mg}/\ell.\text{kPa}$$

a) The recycle flow rate under conditions of maximum air transfer is calculated with Equation 11.3:

$$r_{max} = \frac{10}{0{,}202 \,.\, 550 \,.\, 0{,}75} = 0{,}120$$

$$r_{max}.\, Q_{max} = 0{,}120 \,.\, 800 = 96 \text{ m}^3/\text{h}$$

b) The saturator pressure under conditions of minimum air transfer is calculated with Equation 11.6:

$$P_{min} = 550\left(\frac{6}{10} \,.\, \frac{0{,}202}{0{,}261}\right)^{2/3} = 330 \text{ kPa}$$

c) The recycle flow rate under conditions of minimum air transfer is calculated with Equation 11.7:

$$r_{min} = 0{,}120\left(\frac{6}{10} \,.\, \frac{0{,}202}{0{,}261}\right)^{1/3} = 0{,}093$$

Word of caution

The examples presented in this chapter have deliberately been simplified only to illustrate the integration of empirically derived recommendations with a theoretical framework that links air solubility, recycle flow rate, air requirements, water quality, etc. The actual design problems that will be faced in practice will undoubtedly require much more than is contained in this manual, for example:

- At most applications, some practical constraints will force the designer to reverse the illustrated design sequence to some extent.
- Considerable judgment and analytical tests are required to judge whether these empirical guidelines may be followed, or whether supporting pilot-scale testing will be necessary.
- The recommended guidelines do encompass quite a broad range in most cases, and judgment and experience will be required for their proper application.
- Important design decisions will be based on cost considerations, and it will therefore be necessary to design on cost alternative solutions if the designer is not intimately familiar with the typical costs involved for the specialised items such as saturators, injection nozzles, float scraping equipment, etc. and aspects relating to materials for construction.

12

Filtration

J Haarhoff

Filtration is an indispensable unit operation in water purification. Filtration is considered desirable or necessary even for the most pristine raw water sources, as evidenced, for example, by changes to the United States Safe Drinking Water Act over the past decade. In South Africa, filtration is found at every water works, and in practically all cases either in the form of rapid gravity filtration (the vast majority), or slow sand filtration. This chapter, therefore, focuses primarily on rapid sand filtration, with some attention to the design of slow sand filters.

Although filtration has been used worldwide for more than a hundred years, it still is the subject of continuous conceptual and technological improvements. Several options have to be considered in filter design; they may be designated as upflow or downflow; hydraulic control may be upstream or downstream; backwashing can be done with or without auxiliary air; filter beds may be single- or multimedia, to name but a few. The filter designer has to keep abreast of new developments, and has to recognise the many common principles that underly all the systems. This chapter will, therefore, stress the fundamental principles rather than differentiate among the multitude of peripheral options available in South Africa today.

Successful filtration is overwhelmingly dependent on the nature of the suspension to be filtered. The nature of the suspension is firstly dependent on the combination of raw water quality and the phase separation processes preceding filtration. A highly turbid water, for example, cannot be successfully filtered unless the bulk of the solids is removed first by sedimentation. Chapter 3 provides some guidelines for process selection for different types of raw water. The nature of the suspension is secondly dependent on proper chemical dosing, mixing and flocculation (covered in previous chapters). Their importance cannot be overemphasised. A filter, correctly designed and operated, will fail disastrously if the chemical pretreatment is inadequate. Good filter design is a necessary prerequisite for meeting the desired filtrate quality, but not on its own a guarantee that filtrate of desired quality will be produced.

12.1 CAPTURE OF PARTICLES IN A GRANULAR BED

12.1.1 FILTRATION MECHANISMS

Rapid gravity filtration usually implies *deep bed* filtration, where suspended particles are not strained out of suspension at the media surface, but penetrate into the media bed to be captured some distance below the media surface. Two requirements must be met before a suspended particle is captured in a media bed. Firstly, it has to be drawn from its flow streamline to the surface of one of the media grains, by means of one or more transport mechanisms. Secondly, the particle has to adhere or stick to the media surface, by means of one or more attachment mechanisms.

The most likely *transport* mechanisms have been identified as sedimentation, interception and diffusion. Sedimentation occurs when the suspended particle is drawn by gravity from its flow streamline towards the media grain surface. Interception occurs when a flow streamline passes so close to the media surface that the suspended particle collides with the media grain. Diffusion is caused by the random motion of the water molecules, which will bump small particles from the flow streamlines.

The most important attachment mechanism is Van der Waal's attraction. Once a particle is moved towards the surface of a media grain, it will be kept there by Van der Waal's forces, which only operate effectively at short range.

Much effort has gone into the identification and quantification of these mechanisms, resulting in detailed theoretical equations for each of these mechanisms. These expressions provide a rational explanation for the effects of particle and grain size, particle and grain density, filtration velocity and the water temperature (which affects the density and viscosity of water).

Readers will recognise the analogy of the filtration mechanisms described in this chapter with the mechanisms described in Chapter 8, which deals with flocculation. Whether a particle joins with another particle (flocculation) or with a much larger solid grain surface (filtration), many of the underlying principles are the same.

12.1.2 EFFECT OF MEDIA DEPTH

It is generally assumed that first-order removal applies to a suspension flowing through a homogenous bed. Mathematically:

$$C = C_0 \cdot e^{-\lambda L} \quad \text{...} \quad 12.1$$

with:

C = particle concentration leaving media bed

C_0 = particle concentration entering media bed

λ = filtration coefficient

L = bed depth

Although this assumption is extensively used for mathematical modelling of the filtration process, as will be shown in later paragraphs, it is not adequate for the direct determination of media depth. In a real bed, the filtration coefficient (even if it would be known in advance for a particular application) is continuously changing due to the specific deposit forming within the media pores. The required bed depth for a specific application, therefore, has to be selected on the basis of experience or pilot-plant results.

Practical filter bed depths in South Africa usually vary between 600 mm and 1000 mm, excluding the coarser supporting layers if there are any. A notable exception is the "moving bridge" or "automatic backwash" filters of which a few are used in South Africa for potable water treatment. These filters are backwashed by suction from the top and the bed depths therefore have to be limited to approximately 400 mm or less.

12.1.3 EFFECT OF MEDIA SIZE

As filter media become finer, their ability to remove particles improves. At the same time, finer sand will also lead to faster clogging of the media bed. When using higher filtration rates, it becomes necessary to use coarser media in order to limit the head loss to acceptable limits. To off-set the lower filtration efficiency of coarser media, these beds are then made deeper to compensate. In South Africa, this is demonstrated by two popular commercial systems which both give excellent results. In the one system, fine sand with d_{10} of approximately 0,7 mm is used in beds which are about 800 mm deep. In the other, coarser sand with d_{10} of approximately 0,9 mm is used in beds which are 1 000 mm deep or even deeper. The combination of coarse media with deep beds, as an extreme example, is also demonstrated by the Los Angeles Aquaduct Filtration Plant where coarse anthracite (d_{10} of 1,5 mm) is usedin beds which are 1 800 mm deep (AWWA, 1990b).

12.1.4 EFFECT OF HYDRAULIC LOADING

Filtration efficiency is adversely affected by high filtration rates. At the same time, the capital cost of filtration can be meaningfully reduced by adopting higher filtration rates. Designers therefore strive to find the maximum allowable filtration rate which will still guarantee acceptable filtration efficiency. The evolution of filtration rates in the USA, summarised by Cleasby (AWWA, 1990b), is a good example of how engineers worldwide have succeeded in maintaining filtration efficiency at ever-increasing rates. For the first half of the twentieth century, US designers used a uniform filtration rate of 5 m/h for practically all applications. After a number of pilot studies, it was recognised that this rate could be doubled to 10 m/h, without the addition of filter aid polymers, while still maintaining reasonable efficiency. If filter aid polymers are used, the filtration rate can be increased even further, as evidenced by numerous pilot and plant trials. Full-scale plants are currently authorised to operate up to 24 m/h.

As stated in the introduction, filtration efficiency is in large part dependent on proper chemical pretreatment. A designer can therefore only adopt a high filtration rate if there is convincing evidence that chemical pretreatment will always be optimal. In practice, it will require a high level of operator skill, on-line monitoring equipment controlling an automated chemical dosing system, and possibly also the use of a filter aid polymer dosed directly upstream of the filter. Moreover, there must be assurance that these systems will be meticulously maintained over the full design life of the treatment plant. The inherent difficulty in meeting all these requirements, has led to conservative filtration rates of below 10 m/h in South Africa. Even the largest and most sophisticated treatment plants adopt these conservative rates; filtration rates around 5 m/h are commonly encountered.

The above discussion dealt with the average design rate. A further factor to be taken into account, is the presence of fluctuations in filtration rate. It is generally known that a sudden increase in flow rate through a partially clogged filter will cause temporary turbidity breakthrough. This phenomenon was investigated in detail by Cleasby *et al.* (1963). This factor also comes into play when deciding on the filtration rate. If the filters will be subject to sudden flow fluctuations (when raw water pumps are started or stopped, for example), or if the flow control system does not provide perfectly smooth control, a conservative filtration rate is justified.

12.1.5 MATHEMATICAL MODELLING OF FILTRATION

Although many mechanisms are at work within a media bed, the net result of their complex interactions is manifested in only two external parameters, i.e. the filtration efficiency and the head loss through the media bed. These parameters show remarkably consistent patterns during each filter run and significant deviation from these patterns is cause for alarm. Filtration efficiency will initially improve (filter ripening) and later on deteriorate (turbidity breakthrough). Head loss will increase linearly with time (depth penetration of solids into the media bed), or exponentially with time (indicative of surface filtration due to clogging on the media surface). Mathematical modelling of the filtration cycle strives to predict the filtration efficiency and head loss for a typical filter run.

Firstly, the conservation of particle volume is assumed, namely that the particles removed from the suspension must be trapped in the bed:

$$-\frac{\delta C}{\delta L} = \frac{1}{v} \cdot \frac{\delta \sigma}{\delta t} \text{ .... } 12.2$$

with
- C = particle concentration
- L = bed depth
- v = approach velocity
- σ = specific deposit
- t = time

Secondly, some relationship between filtration efficiency and the depth of the media bed has to be assumed. Practically all models in use today assumes first-order removal, previously formulated as Equation 12.1, which is customarily expressed in differential form:

$$-\frac{\delta C}{\delta L} = \lambda\, C \text{ .... } 12.3$$

with λ = filtration coefficient

Thirdly, some relationship between the filtration coefficient and specific deposit has to be assumed. As the media grains are coated with the specific deposit, their ability to capture more particles is affected; also, the growing specific deposit leaves less void volume for the suspension to flow through. Numerous different assumptions have been made at this point, each leading a different mathematical model. One of the most general and well known models was proposed by Ives, based on the following relationship between specific deposit and filtration coefficient (Ives, 1982):

$$\lambda = \lambda_0 \cdot \left(1+\frac{B\beta\sigma}{\varepsilon_0}\right)^y \cdot \left(1-\frac{\beta\sigma}{\varepsilon_0}\right)^z \cdot \left(1-\frac{\sigma}{\sigma_u}\right)^x \text{ ...... } 12.4$$

with
- λ_o = initial clean bed filtration coefficient
- B = ripening coefficient
- β = bulking factor of the particles upon deposition in the bed
- ε_o = initial clean bed porosity
- σ_u = ultimate specific deposit, after which no particles are deposited
- x,y,x = exponents

The Ives model is the most general and flexible and will accommodate a number of other models with appropriate choice of the exponents x, y, and z. It is mathematically consistent and satisfies all the boundary conditions imposed by the physical nature of a deep bed filter. The model is derived from plausible assumptions and the model parameters, exponents excluded, have physical meaning. The first bracketed factor accounts for filter ripening, and the second for a decrease in removal efficiency as the media surface area is reduced by the filling of the media pores with specific deposit. The third bracketed factor decreases the filtration efficiency as the interstitial deposit approaches the ultimate deposit.

If the partial derivative of concentration with respect to time is eliminated from Equation 12.2 and Equation 12.3, and the Ives model is substituted for the filtration coefficient (assume $x = y = z = 1$):

$$\frac{\delta\sigma}{\delta t} = v.C.\lambda_0 . \left(1+\frac{B\beta\sigma}{\varepsilon_0}\right) . \left(1-\frac{\beta\sigma}{\varepsilon_0}\right) . \left(1-\frac{\sigma}{\sigma_u}\right) \quad ..12.5$$

This is a classical initial value problem of the form $\sigma' = f(\sigma)$ which can be solved with any of numerous numerical methods. Two boundary conditions apply to this equation. Firstly, at time 0 the filtration coefficient throughout the entire bed is equal to the clean bed filtration coefficient; secondly, the concentration entering the top of the bed is constant throughout the filter run and equal to the raw water particle concentration. For numerical calculation, the bed is treated as a series of discrete layers, and the filtration cycle as a series of discrete time steps. The mathematical solution proceeds from the top of the bed to the bottom, and from time zero to the end of the filtration cycle. The calculation sequence is formulated in detail by Adin (1978).

The modelling approach outlined above is based on a macroscopic view of the media bed. Additionally, the modelling can be approached on particle level, i.e. to calculate the trajectory of a particle as it flows around a media grain. By extending the calculation to include the effect of the operational parameters on the probability of collision, an estimate can be obtained for the filtration coefficient.

Mathematical filtration models currently, despite their increasing sophistication, still find limited use in practice. Although they are used in the interpretation and extrapolation of pilot-plant data, their practical application is hampered by the uncertainty associated with the many unknown parameters in the equations. Mathematical models did, however, bring about a much better fundamental understanding of the interplay between the principal filtration variables, which is of considerable qualitative value to designers and operators alike.

12.2 FLOW THROUGH POROUS MEDIA

The aspects of filtration which are of interest include media, flow through the media, backwash hydraulics and flow control.

12.2.1 MEDIA CHARACTERISATION

In South Africa, there is no standard specification exclusively for filter media. Designers therefore have to develop their own specifications on the basis of the few available overseas specifications, practical experience as well as knowing the available products on the local commercial market. A detailed comparison of the few formal specifications that do exist locally and internationally, demonstrated that there is a wide choice of parameters that could be specified, and that limited agreement exists on the desired range of any one parameter (Ceronio, 1993).

The first group of parameters that are normally specified, deals with the hydraulic properties of the media (their ability to filter and be backwashed efficiently), while the second group deals with the stability of the media (their ability to maintain its properties over the many years that filter media are generally used before replacement). The most important parameters in each group will be discussed below.

The effective size d_{10} and uniformity coefficient UC are used to fix the grain size and grain size distribution of a given media sample. These parameters are determined using a standard sieve analysis and d_{10} denotes the sieve size that will pass 10% of the sample mass while the UC is equal to the ratio d_{60}/d_{10}. Here d_{60} denotes the sieve size that will pass 60% of the sample mass. (Strictly speaking, this ratio is a coefficient of non-uniformity, but is not called such in practice.)

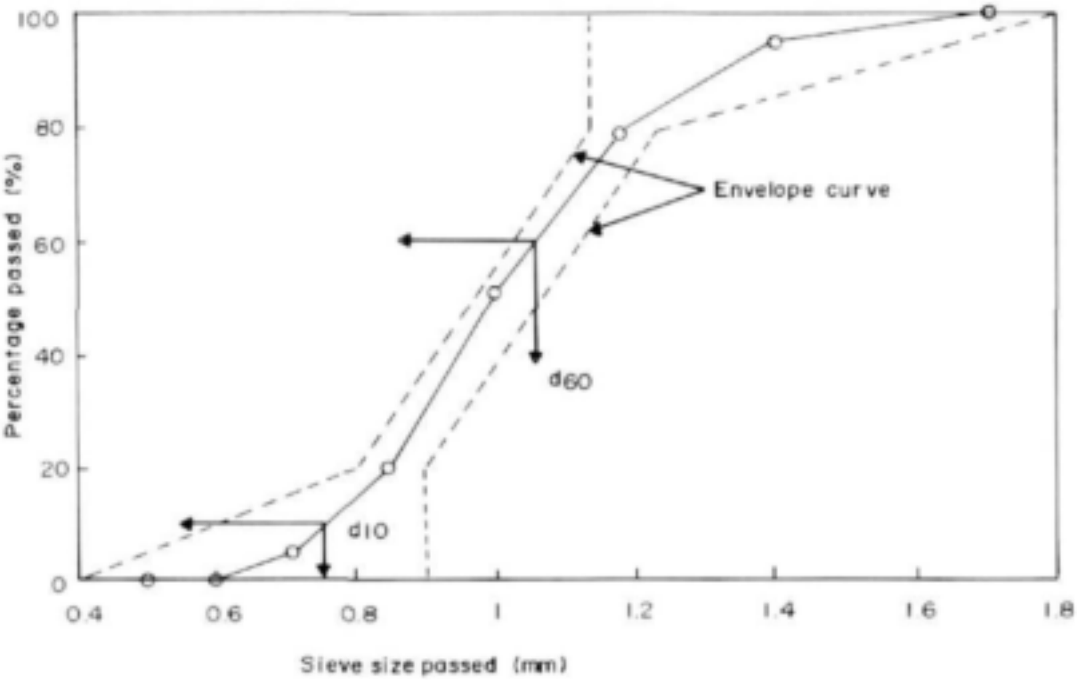


Figure 12.1
Typical grading curve for rapid filter media

A typical result from a sieve size analysis is presented in Figure 12.1. The choice of d_{10} is selected by the designer of a filter bed, while the UC should be as low as practically possible to approach a homogeneous media bed with minimum stratification. The maximum allowable UC values in international specifications range from 1,25 to 1,80. A maximum UC of 1,50 is easily obtainable from South African media producers, but it may be difficult or expensive to produce local media if a maximum UC of less than 1,35 is specified (Ceronio, 1993). Filter media could be specified by the d_{10} and UC only, but envelope curves (also indicated in Figure 12.1) could also be used as they allow for a practical tolerance around the specified grading. A deviation of 5% from the curve is considered acceptable (De Lathouder, 1973).

The *porosity* ε of a media bed is defined as the ratio between the void volume in the bed and the total bed volume. Although it is the determining parameter for the media bed's hydraulic performance, it is not a property of the media grains themselves, but rather a function of the media grain shape. For this reason, specifications rather call for the media grain shape than for the porosity. The bed porosity can be calculated with:

$$\varepsilon = 1 - \frac{M}{\rho_s . V_t} \quad \text{.......................................} \quad 12.6$$

with

ε = bed porosity
V_t = total bed volume
M = media mass
ρ_s = media density

Many expressions and definitions for the *media grain shape* have been proposed, but the most fundamentally correct definition seems to be that of the sphericity ψ, which is defined as the surface ratio of a sphere to a grain where both have the same density and volume (Dharmarajah & Cleasby, 1986). It is, however, very difficult to measure the surface area of a single media grain; for a representative media sample with thousands of grains, it is impossible. Recourse has to be taken to indirect methods of measurement, such as the use of a head loss equation. The media sample is subjected to flow in a test column and the head loss across it is measured. All the other variables in the equation are measured separately, and the sphericity ψ is solved from the head loss equation. In reality, it then is nothing more than a calibration constant for the head loss equation, but if the head loss equation is formulated from sound fundamental assumptions, it will give a good indication of sphericity. Cleasby & Fan (1981) suggested the use of the Ergun head loss equation, which is discussed further on.

The behaviour of the media bed during backwash, when the bed is partially or fully fluidised, is determined to a large extent by the density of the media. This is especially important for multimedia beds, where particular density differences are needed to maintain the stratification between two different media layers. The density is determined by measuring the volume of water displaced by a known mass of media. The density of commercially available silica sand in South Africa ranges in a narrow band around 2 630 kg/m³.

To ensure that media does not break down during the design life of the filter, a number of parameters are used. Stability against low pH is ensured by specification of hydrochloric acid solubility, or conversely by the silica content. Stability against high pH (required for some anthracites) is ensured by specification of sodium hydroxide solubility. Stability against physical abrasion is quantified by weight loss during a period of continuous backwashing. In all of these cases, all the media commercially available in South Africa comfortably complied with the limits laid down by the overseas specifications (Ceronio, 1993).

12.2.2 HEAD LOSS THROUGH CLEAN MEDIA

Fair and Hatch (1933) developed a rational head loss formula by assuming flow through a media bed as analogous to laminar flow through a parallel bundle of pipes. This was subsequently refined by Ergun (1952), by adding an additional turbulent term in the formula. The complete Ergun equation is:

$$\frac{H}{L} = 150 \cdot \frac{\mu \cdot (1-\varepsilon)^2}{\rho \cdot g \cdot \varepsilon^3} \left(\frac{1}{\psi d_{eq}} \right)^2 \cdot v + 1{,}75 \cdot \frac{(1-\varepsilon)}{g \cdot \varepsilon^3} \left(\frac{1}{\psi d_{eq}} \right) \cdot v^2 \quad \text{..........} \; 12.7$$

with:

H = head loss across the media bed
L = depth of media bed
μ = kinematic viscosity of water
ρ = density of water
g = gravitational acceleration
ε = bed porosity
v = approach velocity
ψ = sphericity of the media grains
d_{eq} = equivalent diameter, i.e. the diameter of a perfect sphere with the same volume as a media grain.

The equivalent diameter d_{eq} has to be determined by the count-and-weigh method. A minimum of about 100 media grains has to be weighed individually to get the average mass of a media grain. By knowing the media density, the average volume and d_{eq} can be calculated.

There are two types of silica sand available on the South African market. The first group is mined from a silica deposit in the Transvaal which extends through the Magaliesberg to as far east as Delmas. The sphericity of media from this deposit ranges from 0,5 to 0,7. A second media source is from an aeolian deposit near Cape Town, where wind action had rounded the grains considerably more to result in a sphericity range of 0,7 to 0,8 (Ceronio, 1993).

The Ergun equation above allows head loss calculation for a perfectly homogenous bed, i.e. where all the grains are identical. For application to a real bed, where a variety of sizes are found, the bed has to be treated as separate layers, each with its own representative media size (Fair & Hatch, 1933). The total head loss would then be the sum of the head losses across the individual layers. During such an analysis the bed is normally divided into about 10 layers and the equivalent diameter of each layer could be approximated by the geometrical mean of the smallest and largest grains in that layer.

12.2.3 HEAD LOSS THROUGH PARTIALLY CLOGGED BEDS

The head loss predicted by the Ergun equation is only valid at the beginning of each filter run, when the bed is clean and there is no specific deposit. As soon as the particles in the suspension are trapped to form a specific deposit within the grains, the effective porosity decreases and the head loss increases. The maximum allowable head loss in a rapid gravity filter is typically about 2 000 mm, while the clean bed head loss is typically only about 400 mm. It is clear that the head loss increase by the specific deposit is substantial; it increases the hydraulic gradient through the bed about five times.

The nature of the relationship between head loss and specific deposit can be approximated from the first laminar term of the Ergun equation. If the head loss is calculated at time 0 (when the porosity is ε_0) and again at time t (when the porosity is reduced to $[\varepsilon_0 - \beta.\sigma]$, where σ is the specific mass deposit and β is the factor to convert mass to volume), the following head loss ratio is obtained:

$$\frac{H_t}{H_0} \propto \left(\frac{\psi_0 d_{eq,0}}{\psi_t d_{eq,t}}\right)^2 \cdot \left(\frac{1-\varepsilon_0+\beta.\sigma}{1-\varepsilon_0}\right)^2 \cdot \left(\frac{\varepsilon_0}{\varepsilon_0-\beta.\sigma}\right)^3 \quad \text{....12.8}$$

Some terms in this equation are difficult, if not impossible to measure. A number of simplified relationships, which reveal part of the structure of the above equation, have been proposed (Sakthivadivel *et al.*, 1972). Despite its very important role in rapid gravity filters, there is, at present, no commonly accepted relationship between specific deposit and head loss.

12.2.4 BACKWASH HYDRAULICS

A proper analysis and understanding of backwash hydraulics can answer the following questions:

- What is the head loss through the media bed during backwash?
- At which backwash velocity will the media start to fluidise?
- How much will the media bed expand during backwash?

The head loss during backwash, if the sand is not fluidised, should be the same as during filtration. The specific deposit, however, is rapidly scoured from the media grains, and the head loss during backwash can for practical purposes be calculated with the Ergun equation, always with the important proviso that the bed is not fluidised. If the bed is indeed fluidised, the head loss is equal to the buoyant weight of the media, regardless of the backwash velocity. If the backwash velocity is thus slowly increased from zero, the head loss will increase about proportional to backwash velocity, until the sand is fluidised; thereafter, the head loss remains constant. The head loss gradient through a fluidised sand bed is:

$$\frac{H}{L} = \left(\frac{\rho_s - \rho_w}{\rho_w} \right) . (1 - \varepsilon) \text{ .... } 12.9$$

with

ρ_s = density of sand
ρ_w = density of water

The fact that the head loss gradient through a media bed loss is constant after fluidisation can be used to predict the minimum backwash velocity v_{mf} at which the bed will fluidise. The expression above can be equated to the Ergun equation, and v_{mf} can be solved from the resulting expression. In order to do this, however, values for ε and ψ are required, but they are not always available. Wen & Yu (1966) introduced two equations relating ε and ψ in such a way that both are eliminated from the expression for v_{mf}. The two Wen & Yu equations have been tested on a variety of South African silica sand samples and found to be valid (Ceronio, 1993). After the elimination of ε and ψ, v_{mf} can be calculated with the formulas given as Equation 12.10. (Re_{mf} is the Reynolds number at v_{mf} and Ga is the Galileo number).

$$v_{mf} = \frac{Re_{mf} . \mu}{d_{eq} . \rho}$$

$$Re_{mf} = \sqrt{33{,}7^2 + 0{,}0408 . Ga} - 33{,}7 \text{ .... } 12.10$$

$$Ga = \frac{d_{eq}^3 . \rho . (\rho_s - \rho) . g}{\mu^2}$$

The above procedure assumes a homogenous bed with identical particles. For a practical bed with variation in grain size, the v_{mf} of the bed must be calculated for the larger grains to ensure that the entire bed is fluidised and it has been suggested that d_{90} be used for this purpose (Cleasby & Fan, 1981). As v_{mf} is a critical design parameter, an additional safety factor of 1,3 has been suggested (Cleasby & Fan, 1981).

Should the backwash velocity be increased beyond v_{mf}, the bed will expand. Many equations have been put forward to predict the expansion of the bed (required for determining the freeboard of the backwash trough above the media surface). The most comprehensive model to date is a correlation put forward by Dharmarajah & Cleasby (1986), and is given below in the form of a series of equations which have to be solved iteratively:

$$\log(A1) = 0{,}56543 + 1{,}09348(\log Re_B) + 0{,}17979(\log Re_B)^2 - 0{,}00392(\log Re_B)^4 - 1{,}5(\log\psi)^2$$

$$A1 = \frac{\varepsilon^3}{(1-\varepsilon)^2} . \frac{\psi . d_{eq}}{216} . \frac{\rho . (\rho_s - \rho) . g}{\mu^2} \text{... } 12.11$$

$$Re_B = \frac{\rho . v . \psi . d_{eq}}{6 . \mu . (1 - \varepsilon)}$$

The Re_B in the above equation is known as Blake's Modified Reynolds number. The authors of the model warn that the model deviates from observed behaviour at a) $\varepsilon > 0{,}85$ for media grains with $Re_t < 100$, b) $\varepsilon > 0{,}90$ for $Re_t > 100$ and c) media density greater than 4 300 kg/m³.

These equations provide an iterative solution for ε_e, which is the porosity of the ex-

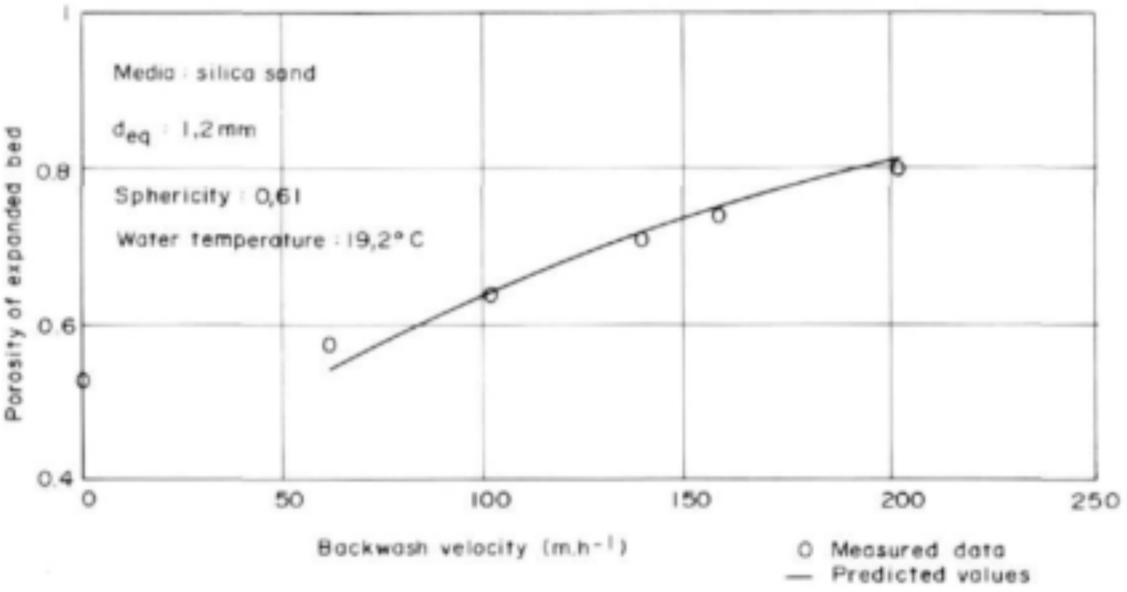


Figure 12.2
Filter bed porosity vs. backwash velocity

panded bed. Designers, who are mostly interested in the linear expansion of the media bed during backwash, can convert ε_e to L_e with:

$$L_e = L \left(\frac{1-\varepsilon}{1-\varepsilon_e}\right) \quad \text{.................................} \quad 12.12$$

The Dharmarajah/Cleasby model was compared to the experimentally determined expansion of a number of South African commercial silica sand samples, and the predictive merit of the method is illustrated in Figure 12.2.

The prediction of bed expansion in a real media bed with variation in sand grain sizes is done similarly to the prediction of head loss. The bed is divided into several layers according to grain size and density, and the expansion for each of these layers is calculated individually. The individual results are added to determine the total bed expansion.

12.2.5 Flow control through porous media

An important aspect of the flow through porous media, is the need to control the flow rate at the desired level. Clean sand is highly permeable, whereas partly clogged sand has much higher resistance. Without flow control, a high filtration rate would be observed at the beginning of a run, gradually decreasing as the media bed gets clogged. To obtain a constant flow rate, a means must be provided to dissipate a large quantity of head loss at the beginning of the filter run, decreasing as the run progresses.

A commonly used option in South Africa is to provide a throttling valve downstream of the filter, called the downstream control system. The valve is highly throttled at the beginning of a run, and gradually opens up during the run. When the valve is totally open, a signal is given to backwash the filter. Because of the flow proportioning downstream of each filter, the water levels in the filters and the common inlet channel are all the same.

A second option, also frequently encountered in South Africa, is the upstream control system. Here, the water is proportioned to each filter through inlet weirs in the inlet channel. The excess head loss is then dissipated by a free fall up to the water level in each filter. The water level in each filter will start at a low level when the bed is clean, and will rise as the bed clogs. A high water level in a filter is a signal for the operator to backwash a filter. There is no downstream flow control device.

The downstream control system has the advantage that it does not subject the incoming water to high energy losses, and chances of floc break-up are minimal - an important consideration for direct filtration. Furthermore, the filter outlet box can be placed at a lower level than the media bed, which leads to lower filter walls. The upstream control system, on the other hand, requires neither mechanical devices to maintain a constant flow rate, nor instrumentation to indicate the need to backwash - the inlet water level acts as a pressure gauge.

It is now increasingly accepted that the filter flow rate need not necessarily be constant during the entire filter run. With declining rate filtration, a clean filter will start out at an initially higher than average filtration rate, but will end the filter run at a lower than average filtration rate. This is done by allowing all filters to have a common inlet and outlet, without mechanical throttling. It may be necessary to insert an orifice plate in the outlet line to restrict the flow variations to the designer's specifications, but otherwise no other control devices are necessary. A comprehensive design method for these filters is described by Cleasby (1993). Contradictory arguments for and against declining rate filtration, in terms of water quality, are:

- The filtration rate is lower at the end of the run, and therefore less likely to slough the specific deposit from the media grains when it is at its thickest. This should lead to better filtrate quality towards the end of the filter run.
- The filtration rate is highest at the beginning of the filter run during the critical period before the filter has ripened. This should lead to worse quality for the first part of the filter run.

While these statements have merit for individual filters, practical results do not appear to indicate significant quality differences for a number of filters working together. The worst filtrate from a clean filter is discharged at the same time as the best filtrate from the dirtiest filter, making for an acceptable average filtrate quality at all times.

12.3 MEDIA CLEANING

Filter cleansing is done by backwashing with water and air or water alone. The aspects which have to be considered include backwash initiation, mechanisms, water wash only, air and water wash, and management of backwash.

12.3.1 BACKWASH INITIATION

Filters have to be backwashed when one of three conditions occurs, namely when:

- filters reach their terminal head loss,
- filtrate quality deteriorates beyond an acceptable limit, or
- the run length exceeds an acceptable maximum.

The terminal head *loss* is easy to determine for constant-rate filters, which are by far the most commonly used in South Africa. For the downstream flow control system, it is that point in time at which the outlet control valve is fully opened. Further clogging of the media cannot be compensated for and the flow rate will decrease. For the upstream flow control system, it is that point at which the water level in the filter backs up over the flow distribution weir into the distribution channel, which will also lead to a decrease in flow rate through the filter. For declining rate filters, the terminal head loss is reached when a predetermined water level in the filter is reached.

In practice, filtrate quality is normally not used to terminate a filter run. This would require on-line water quality monitors (usually low-range turbidimeters) on every filter, as well as computer surveillance of every monitor. The need to monitor filtrate quality can be reduced by limiting the maximum hydraulic gradient through the filter. The higher the gradient, the better the chance of detaching previously deposited particles into the filtrate. In the ideal filter, the maximum head loss will be reached at exactly the same time as the onset of turbidity breakthrough. In practice, however, it is preferable to limit the maximum hydraulic gradient to the point where it is almost certain that the maximum head loss will be reached before turbidity breakthrough. In South Africa, for the conservative filtration rates normally used, it is customary to allow approximately 2 000 mm for maximum head loss through the filter bed.

The filter run length is also used for the termination of a filter run, regardless of the head loss, filtrate quality or the volume of water filtered. This is done to prevent the caking of filter media by biological growths, or the bacterial breakdown of the specific deposit. Both problems could be encountered in South Africa, with warm temperatures, abundant sunlight and eutrophic water sources. In practice, filter run lengths are generally limited from 48 to 72 hours.

12.3.2 BACKWASH MECHANISMS

Deposits can be removed from media grains by two distinctly different mechanisms. The first mechanism is hydrodynamic shear, when a grain is suspended in a vertical flow field and its surface is subjected to the shear forces exerted by the water. The second mechanism is abrasion, when particle surfaces rub against each other to remove the specific deposit. After considerable debate during the previous twenty years, it has been conclusively demonstrated by eminent workers in this field that abrasion in a bed fluidised by water alone does not occur to any significant extent, and that abrasion as a mechanism of cleaning is thus of negligible importance (Amirtharajah, 1971; Baylis, 1959; Cleasby & Lorence, 1978).

Backwash systems currently used in South Africa, which will be discussed below, can be categorised as follows:

- Backwashing with water only
- Backwashing with air and water consecutively
- Backwashing with air and water simultaneously.

12.3.3 WATER ONLY

Backwashing with water only is a cleaning method widely practised in the USA, and is often referred to as the "American" system. This method is rarely used in South Africa, with the exception of the backwashing of some pressure filters mostly used in swimming pools. Even in the USA, the original system has since been enhanced by backwash auxiliary systems such as surface wash (AWWA, 1990a).

Typically, the backwash rate into the filter underfloor system is gradually increased until the final backwash rate of 35 to 50 m/h is reached after about 30 seconds. The filter media gradually assumes a fluidised state as the backwash flow rate is increased and the bed is expanded. It is a very simple method which only requires a single washwater distribution system. Two variants of the process exist:

- Low-rate backwash causing less than 10% expansion of the filter medium
- High-rate backwash where the wash rate is sufficient to expand the filter medium by 10% to 50%, or even higher.

Both these options tend to stratify the granular media (the second option faster than the first), with a resulting thin layer of fine grains at the top of the bed after backwashing, which could lead to shorter filter runs and higher head loss through the media. Backwashing with water alone is therefore an inherently weak cleaning process, as fluid shear is not very effective. Amirtharajah (1971) showed that maximum hydraulic shear in a fluidised bed occurs at porosities of 0,68 to 0,71 for typically sized silica sand, which correspond to bed expansion of 80 to 100%.

To improve the media cleaning in a bed, it is necessary to introduce more energy into the media bed which will lead to abrasion between grains and higher hydraulic shear forces in the bed. High-rate backwash may still be adequate when filtering solids with weak adhesive forces holding the deposits on the grains such as filtration without chemical treatment or with Al^{3+} or Fe^{3+} coagulation. There is little doubt that air scour or surface wash becomes indispensable for effective cleaning when filtering wastewater or water with polyelectrolytes or whenever the solids become attached to the filter grains with stronger adhesion forces (Amirtharajah, 1980).

Surface wash is a common backwash auxiliary in the USA and is accomplished either with a grid of fixed pipes placed above the granular media, or with rotary water distribution arms, containing orifices or nozzles that supply high pressure jets of water above the fixed-bed surface prior to the backwash and into the upper layers of the media during part of the bed expansion. It is relatively simple since it only comprises of a system of distribution nozzles injecting high-pressure water into the bed.

12.3.4 CONSECUTIVE AIR AND WATER

This type of backwash operation, often called the "British" system, is in common use in South Africa and has proved itself over many years as an effective system when correctly designed and constructed. The air scour first abrades the deposit from the media grains and is followed by the water backwash to flush the deposits out of the bed. Air is introduced through the bed at rates of 18 to 36 m/h, followed by water backwash just above the point of incipient fluidisation at 12 to 30 m/h. The duration of the air scour is usually between 2 and 3 minutes and the water backwash is continued until the washwater is clear.

When air is introduced at the bottom of the filter, bubbles travel upward carrying some water and dirt particles as they pass through the bed and burst at the surface, where the maximum scouring action appears to be produced. Substantial agitation of the medium near the bed surface occurs and it is this action that provides the basis for air scour use in sand filters where minimum solids penetration occurs. Deeper agitation is also observed during the first minute or so of air scour. Thereafter, the bed begins to settle down, and the air becomes more channelled as it passes through the bed (Cleasby *et al.*, 1977).

One of the drawbacks of this system is the fact that the sand compacts during air scour and that the period of useful agitation is limited to a minute or two. New, automated backwash sequencing systems make it possible to partially eliminate this problem by sequencing two or even three backwash cycles instead of only one, in such a way that the same amount of or less backwash water is used than for a single cycle. This method allows consecutive air and water to approach the efficiency of simultaneous air and water.

12.3.5 SIMULTANEOUS AIR AND WATER

This system, sometimes called the "French" system, is also common in South Africa. During the first step of the backwash process, air scour (at 50 to 60 m/h) and water backwash (at 10 to 15 m/h) are used simultaneously for coarse sand single media (effective size typically larger than 0,9 mm).

This water rate is well below the fluidisation velocity of the sand. After a period of 5 to 10 minutes of simultaneous air and water backwash, the air flow is terminated. During the second step, the water continues to expel some of the air from the bed and to flush the remaining dirt from the water above the filter medium. The flow rate may be increased (up to 15 to 25 m/h) but should remain below the fluidisation velocity of the sand to avoid the loss of sand.

It has been demonstrated (Amirtharajah, 1993) that this method of backwashing is most effective when the ratio of the air and water flow rates is such that collapse-pulsing of the bed occurs. Collapse-pulsing conditions can be created at combinations of air and subfluidisation water flow rates where cavities larger than the media grains form and collapse. This process repeats itself throughout the depth of the bed. The greatest amount of abrasion among the media grains is achieved during collapse-pulsing conditions. This is the optimum condition for the removal of particles during the backwashing of media beds with water and air simultaneously.

12.3.6 COMPARISON OF BACKWASH SYSTEMS

Every commercial backwash system has its own unique practical features which make it more or less desirable. The emphasis of this section is on the inherent strengths and weaknesses of the different systems.

Cleasby *et al.* (1977) drew a comprehensive pilot-plant comparison between different backwashing systems, namely water alone, water enhanced by surface wash and consecutive air and water (below media fluidisation). Using municipal wastewater as a suspension, they ranked the systems as follows:

- The simultaneous air and water system was the most effective.
- Water enhanced by surface wash, and consecutive air and water were comparable in efficiency.
- Water alone was least efficient.

These findings do not imply that simultaneous air and water should necessarily be used for all applications. It very much depends on the nature of the particles captured in the filter bed. If media grains are easily cleaned, less efficient systems may be perfectly acceptable. Cleasby *et al.* (1977) concluded:

- When physical mechanisms are responsible for retaining floc particles in the bed (e.g. iron removal), water fluidisation alone can be as good as air scour or surface wash.
- If the raw-water quality is such that the film around the media grains is firmly attached (e.g. wastewater or lime-softened water), air scour or surface wash is essential. Synthetic polymers increase the "adhesiveness" of the film around the grains and in such instances a backwashing system with the best efficiency (backwashing with air and water simultaneously) would be advisable.

The implication of these findings for South Africa, where polymeric filtration aids are seldom used, is that consecutive air and water, as well as simultaneous air and water, can be considered on equal technical footing for most applications.

12.3.7 BACKWASH WATER MANAGEMENT

There is constant pressure on designers and operators to minimise water losses during treatment, especially since the steep increase in raw water tariffs in some parts of South Africa. As a result, more treatment plants are now incorporating systems for the recovery of filter backwash water. The following issues need to be considered before such a system is implemented:

- The total volume and flow rate of backwash water can only be calculated once the media backwash system is selected. It is not always as simple as assuming backwash water to be a constant fraction of the plant production. In many cases, especially during periods of low demand, filters are washed on time rather than head loss, which will cause a proportionally higher backwash fraction.
- Some media loss during washing is inevitable. Where filter sand is at a premium, a media trap should be incorporated into the backwash water recovery system.
- Where backwash water is recovered from lagoons, no chemical dosing is required. Where space and topography prohibit the use of lagoons, backwash water can also be recovered from much smaller settling tanks. At least one plant is known in South Africa where the recovery from such settling tanks has to be aided during certain times of year by the addition of coagulant.
- Backwash recovery systems do have the potential to increase the concentration of parasites in the raw water, where recovered water is returned directly to the head of works rather than to the raw water impoundment. No such evidence has been reported for South Africa, but in England, the use of slow sand filtration is advocated to remove parasites from the recovered backwash water.
- Direct return of recovered backwash water must be carefully and gradually controlled to ensure accurate chemical dosing.
- The capital cost of a backwash recovery system, along with the additional operational and maintenance burden, must be carefully weighed against the savings in raw water costs, and the costs that backwash water disposal would otherwise have incurred. The financial benefits can be substantial; for a large South African treatment plant with a capacity of 120 Mℓ/d, the payback period for the entire backwash recovery system, lagoons included, turned out to be only three years (Magalies Water, 1994).

12.4 RAPID GRAVITY FILTER DESIGN

The design of rapid gravity filters includes process considerations, layout, hydraulics, backwash and underfloor systems, automation and control.

12.4.1 PROCESS CONSIDERATIONS

Detailed design of a rapid filtration design begins with a number of process considerations. Specific questions that need to be addressed at this point include:

- What is the direction of flow? Almost all filtration systems in South Africa, with a few exceptions, use downflow filtration rather than upflow filtration.
- Will a single or multimedia bed be used? It is common to use a single sand layer for most applications, but in some cases a top layer of anthracite is used in conjunction with sand. This has the benefit of prolonging the filter runs when using direct filtration or combined flotation/filtration, but has the added complexity of specifying sand and anthracite which will work together efficiently.
- The option exists to use granular activated carbon as a top layer, or as the full media bed, when some sorptive capacity is required. Although not used in South Africa in this way, this option has been used elsewhere to contain pesticide spikes from agricultural runoff, for example.
- What is the maximum allowable clogging head loss? An empirical guideline of 1,5 m to 2,5 m is used in South Africa, but no data are available to assess the validity of this guideline. One would intuitively expect that the "softer" organic deposits from eutrophic waters would be sheared more easily from the media bed than inorganic clay deposits, but this point has yet to be verified.

12.4.2 FILTER BLOCK LAYOUT

The total filter area is determined by the design hydraulic loading. The many factors that influence the choice of hydraulic loading were discussed earlier. For various reasons, the hydraulic loadings in South Africa are conservatively fixed between about 5 m/h and 10 m/h.

With the total area fixed, the number of filters needs to be determined. The practical design considerations are:

- A minimum of three, preferably four filters are required to keep the variation in filtration rate within limits when one filter is backwashed. For a small treatment plant, individual filters often have to be made very small.
- A single filter has a maximum practical size of about 100 m^2, above which the backwash equipment becomes excessively large and expensive. For a very large treatment plant, there is no other option than using large filter galleries with a multitude of filters.
- The problem of expensive backwash equipment can be mitigated by placing the backwash trough in the middle of a filter and washing the filter one half at a time. This has the penalty of having two sets of backwash control valves.
- Some designers limit the distance between the backwash trough and the far filter walls to some arbitrary maximum to prevent "dead" spots next to the far wall. Right next to the far wall, there is very little transverse velocity above the media towards the backwash trough, and dirty backwash water takes a while to wash

away. This stagnant zone, however, is independent of the distance to the backwash trough. Whether the backwash trough is 2 m or 5 m away, the conditions next to the wall will be the same.

With the size and number of filters known, the horizontal layout of the filter block can be made. The following factors need to be considered:

- A single row of filters alongside each other will require a simpler flow distribution and collection system, but will require a longer gallery than two rows of filters alongside a central gallery.
- Backwash troughs can be placed parallel, or at right angles to the gallery. For smaller filters, they are usually placed on one side to leave a contiguous sand area, but for larger filters, they are centrally placed for reasons discussed earlier.
- Each filter necessarily has five connections to the rest of the filter gallery, namely raw water in, filtrate out, air in, washwater in and washwater out. Designers can make all connections on the gallery side of the filters, which enables the operator to control the backwash sequence from one point. Alternatively, some connections can also be made from the opposite end, to leave a less cluttered gallery layout, especially for smaller filters.

To determine the hydraulic gradient through the filtration system, a decision has to be arrived at on the method of flow control. Although the total head differential across the filtration system (the difference in water level between the inlet channel and the outlet box) is fixed by the clean bed media loss and the maximum clogging head, the relative height of the media bed has to be determined:

- For the upstream flow control system, there is no other option but to drop the top level of the media bed to below the water level in the outlet box. This will prevent the water level from dropping below the sand surface and the sand from drying out. Dry beds can lead to loss serious media loss if backwashed first before being put into operation.
- For the downstream flow control system, there is no such restriction on the vertical position of the media bed, since the rate controlling valve (if operating properly) will always maintain the water level above the sand level. This allows the designer to place the media bed higher than the outlet box, which leads to shallower filter boxes. On difficult terrain, this may have a significant cost benefit in terms of reduced excavation and filter wall materials.
- For declining rate filtration, the relative position of the media bed depends on extended hydraulic calculations, as detailed by Cleasby (1993).

12.4.3 HYDRAULICS

Detailed hydraulic calculations are an important part of rapid filtration design. Table 12.1 indicates the wide variety of typical calculations required. The hydraulics of porous media, which is a specialised area not normally covered by undergraduate engineering courses, were covered in detail in Section 12.2 of this chapter. With the exceptions of manifold theory and linearly increasing open channel flow, which will be discussed further on, most of the remaining analyses indicated in Table 12.1 require simple and well-known hydraulic theory taught at undergraduate level, and are accessible in elementary textbooks on fluid mechanics. Guideline values for actual flow velocities are given in Table 12.2.

TABLE 12.1
HYDRAULIC ANALYSES REQUIRED FOR RAPID FILTRATION SYSTEMS

Element	Flow rate	Remarks
Inlet distribution	filtration rate*	For even distribution of water to all filters. Distribution by pipe or channel. Weir flow into filter for upstream flow control systems.
Media head loss	filtration rate*	For determining clean bed head loss. Porous media hydraulics.
Media head loss	backwash rate	For determining the duty point of backwash pump(s). Porous media hydraulics.
Media expansion	backwash rate	For checking freeboard between media surface and backwash trough. Porous media hydraulics.
Nozzle head loss	filtration rate*	For determining clean bed head loss. Nozzle hydraulics.
Nozzle head loss	backwash rate	For determining the duty point of backwash pump(s). Nozzle hydraulics.
Overflow chamber	filtration rate*	For sizing overflow weir and chamber. Overflow depth vs. discharge required for electronic control systems. Weir hydraulics.
Filtrate collection	filtration rate*	For conveyance of filtered water to next step. Open channel or pipe.
Washwater supply	backwash rate	For determining the duty point of backwash pump(s). Supply pipe is often looped to ensure even distribution to all filters, regardless of position in gallery.
Washwater collection	backwash rate	For sizing of backwash trough. Linearly increasing open channel flow.
Washwater disposal	backwash rate	For conveyance of dirty backwash water to disposal/collection point. Open channel flow.

* The flow rate through each individual filter during filtration is normally based on one filter being washed, with the remaining filters sharing the additional hydraulic load evenly.

TABLE 12.2
GUIDELINE VALUES FOR FLOW VELOCITIES
(VAN DUUREN, 1992)

Pipe or conduit position	Flow velocity (m/s)
Raw water influent	1 to 2
Influent carrying flocculated water	0,3 to 0,6
Effluent conduits for filtered water	2 to 5
Washwater drainage	1,2 to 1,5
Clean washwater	2,5 to 3,5
Filter-to-waste connections	3,5 to 4,5
Wash air	12

12.4.3.1 Linearly increasing open channel flow

A backwash trough collects washwater over its full length, with the flow consequently increasing linearly towards the outlet. An expression can be derived (e.g. Reynolds, 1982) with the assumptions that a) the flow passes through critical depth at the outlet of the trough, i.e. there is a free fall at the outlet into the collector channel, and b) there is no wall friction:

$$D_o = \sqrt{d_c^2 + \frac{2 \cdot Q^2}{g \cdot b^2 \cdot d_c}} \quad \text{..........................} \; 12.13$$

with

D_o = water depth at upstream end of trough
d_c = critical water depth at downstream end of trough
Q = total flow entering trough
b = trough width

The critical depth is given by:

$$d_c = \sqrt[3]{\frac{Q^2}{g \cdot b^2}} \quad \text{..} \; 12.14$$

The water depth calculated in this way can be increased by between 6% and 16% to compensate for the fact that there are some frictional losses in practical systems (Reynolds, 1982).

12.4.3.2 Manifold flow

In filter underfloor systems, manifolds are pipes or ducts with the purpose of distributing water flow to uniformly spaced secondary fittings. In a pipe lateral, the secondary fittings would be the filter nozzles whilst in a header pipe the secondary fittings would be the lateral pipes. Hydraulically, they are analysed similarly. Perfectly even manifold distribution with uniform secondary fittings is impossible. Some pressure difference along the manifold between the first and the last secondary fittings will always be found - this has to be calculated.

The pressure in a manifold will be affected by two factors. Firstly, pressure energy will be lost due to wall friction, which will decrease the pressure as the water moves down the manifold. Secondly, the velocity head in the manifold will decrease in a downstream direction, as a part of the flow is lost to every secondary fitting. As the total energy consists of the sum of pressure and velocity head, the pressure will increase in a downstream direction. Friction losses are seldom a major factor in underfloor systems due to the short lengths and smooth walls involved. Moreover, as it has the opposite effect of the much more important velocity head effect, it can be safely ignored for practical computations (Hudson, 1981).

Hudson (1981) developed an iterative technique to estimate the flows in each lateral and orifice of a manifold system. The following empirical equation was adopted for the lateral entry, after review of data from a number of studies:

$$H_l = \left[\phi\left(\frac{v_m}{v_l}\right)^2 + \theta\right] \cdot \frac{v_l^2}{2 \cdot g} \quad \text{..........} \; 12.15$$

with

H_l = lateral entry loss (m)
v_l = average velocity in lateral (m/s)
v_m = average velocity in manifold (m/s)
g = gravitational acceleration (m/s^2)
ϕ = 0,9 for long laterals or 1,67 for short laterals
θ = 0,4 for long laterals or 0,9 for short laterals

(Laterals are considered short when the length of the lateral is smaller than three times the diameter of the lateral)

The final solution is obtained by assuming perfect distribution as a starting point, and adjusting the flows with each iteration until the required level of iterative closure is reached. This rather laborious method is fully detailed in Hudson (1981).

Chaudhry & Reis (1992) used the same mathematical model as Hudson (1981), but greatly simplified its application by rewriting the equations in dimensionless form in order to solve them directly with a forward difference solution method. An equation is finally derived which gives the discharge at each successive lateral/orifice:

$$Q'_k = Q'_{k-1} - \sqrt{\Delta H' - \frac{K_1}{K_2} \cdot Q'^2_{k-1}}$$

$$K_1 = \frac{\phi}{2 \cdot g \cdot A_m^2}$$

$$K_2 = \frac{1 + \theta}{2 \cdot g \cdot A_l^2} \quad \text{....................................} \; 12.16$$

$$Q'_{k-1} = \frac{Q_{k-1}}{Q_0}$$

$$\Delta H' = \frac{\Delta H}{K_2 \cdot Q_0^2}$$

with

$\Delta H'$ = dimensionless path head loss
Q' = dimensionless discharge in manifold
Q_o = total discharge entering manifold (m^3/s)
A_m = cross-sectional area of the manifold pipe (m^2)
A_l = cross-sectional area of the lateral pipe (m^2)

The definition of the symbols used in Equation (16) is indicated in Figure 12.3. The method is fully detailed in Chaudhry & Reis (1992).

For pipe lateral systems, the methods presented above must be applied twice; first to calculate the pressure differences along the main header pipe or channel (into which the pipe laterals connect), and secondly to calculate the pressure differences along the pipe laterals themselves. Equation 12.16 could be similarly applied to pipe laterals where the flow distribution is simply done through orifices in the wall of the pipe lateral (see Figure 12.3). The application of Equation 12.16 to filter nozzles protruding into a pipe lateral has, to the authors' knowledge, not yet been experimentally verified. Until such time, Equation 12.16 should be used with caution, and only for preliminary checking of adequate distribution amongst nozzles in pipe laterals.

Fair *et al.* (1958) recommended the following ratios as a rule of thumb for the design of filter manifold systems:

- Ratio of area of lateral to area of orifices served - (2 to 4) : 1
- Ratio of area of manifold to area of laterals served - (1,5 to 3) : 1.

These guidelines, in combination with Equation 12.16, was used to calculate the variation in flow between the first nozzle of the

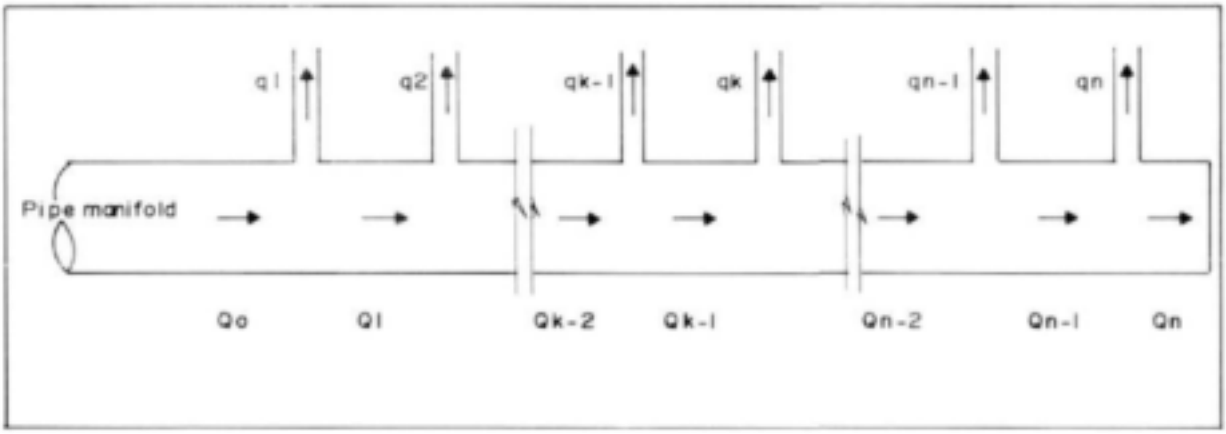


Figure 12.3
Filter pipe manifold analysis

first lateral and the last nozzle of the last lateral for a typical filter floor of average size (Lombard & Haarhoff, 1994). If the most conservative values of the above guidelines are used, the variation is about 3%; the highest values lead to a variation of 16%. The average guidelines will result in a flow variation of about 6%, which can therefore be accepted as typical design guideline, which corresponds with an earlier suggestion by Hudson (1963) of a maximum variation of 10%.

12.4.4 BACKWASH AND UNDERFLOOR SYSTEMS

The backwash procedure, the underfloor system and the filter nozzles are not uniquely designed for every project. A number of specialist contractors each have one or more standard systems or components, based on years of experience and continuous refinement. Based on each enquiry, these contractors will provide the most appropriate solution in terms of their own equipment. The challenge to the process designer in this case is to provide a specification with enough freedom to allow tenderers to offer their own equipment, but will also guarantee a solution which will comply with the client's requirements. Tenderers would naturally want maximum freedom to offer an entire system at their own discretion, while clients could have a preference for specific equipment which served them well in the past; the designer must therefore seek a compromise.

The following interactions are listed to demonstrate that filter nozzles, filter media, air scour and backwash rates are closely and mutually interdependent:

- The provision of coarse media support layers allows the use of filter nozzles with wide dome slots to lessen the risk of nozzle clogging when sand gets into the underfloor system. If the support layers are omitted, filter nozzles with much finer dome slots (finer than the smallest media grains) have to be used.
- The grading of a media bed which consists of coarse supporting layers and possibly an anthracite layer at the top must be such that there is no or little intermixing of the different layers.

- The media grading has a close relationship with the backwash rates used; finer media could be washed out by high backwash rates, while coarse media will be inadequately cleaned by low backwash rates.
- The backwash and air scour rates must exactly match the rates for which the filter nozzles have been designed. If not, water and air distribution will be inadequate.

Clearly, the backwash and underfloor systems must be designed as an integrated system. This point should also be recognised by operators; filter media or broken nozzles cannot simply be replaced from the nearest, cheapest source, but must conform closely to the original specification.

12.4.5 AUTOMATION AND CONTROL

Rapid filtration systems comprise numerous mechanical elements which require continuous or frequent movement. Automation or semi-automation of rapid filtration has, therefore, always been a goal to relieve operators of continuous filter attendance:

- During filtration, filters with downstream flow control require small incremental flow adjustments at short time intervals; for this, a number of ingenious flow regulation systems have been used in the past, based on floats, lever arms, overflow chambers, syphons and similar hydromechanical devices.
- During backwash, a minimum of 14 actions are required for each wash; five valves, one air blower and one backwash pump each requires two actions (open/close for valves, start/stop for electric motors). Where numerous filters have to be backwashed at a large plant, automatic sequencing of these actions, and automatic queueing and initiation of washes become indispensable. This sequencing is usually accomplished with an electrical system with timers and relays.

With the advent of the electronic computer, it became possible to accomplish both these control actions with a single programmable controller, which will control the flow control and backwash sequencing for all filters. Such systems have been put in use in South Africa, with great success, since the early eighties and are becoming commonplace for large filtration systems. Computer control, however, does not merely replace those control functions that were earlier performed by hydromechanical and electrical systems. There are additional benefits and possibilities, of which a few are enumerated:

- It becomes relatively simple to force filters to wash when maximum head loss *or* maximum filter time *or* maximum filtrate production is reached; moreover, these maximum limits can be adjusted easily through the system software.
- Detailed and accurate logging of each filter can be performed, such as head loss development, filtrate production and quality, and filter run times. Comparative analysis of these data will point out problems with any specific filter at an early stage.
- An automated system will eliminate operator error, such as not allowing enough time between the stop of the air blower and the start of the backwash pump (to allow the underfloor air to escape first), or not timing the air scour and backwash cycles properly, or starting equipment when the valves are not correctly set. Some of these errors previously could be prevented through the provision of electrical interlocks; it is not required when these actions are under computer control.

- Media loss at the onset of the air scour cycle can be prevented by allowing the filter to drain down to a distance *below* the backwash weir, so that no water (and media) is discharged into the backwash trough when the blower is started.
- It becomes a simple matter (and has indeed been implemented at a number of treatment plants) to allow for a double or triple wash, which leaves the filter beds much cleaner than a single wash.
- A backwashed filter can be eased back into operation by allowing a low initial flow rate at first to attenuate the initial turbidity peak associated with filter ripening.

Experience gained during the automation of a number of South African rapid filtration plants showed that it takes considerable time and effort to add automation features or changes once the system is commissioned. This places the responsibility on the designer to specify, at process design stage, in great detail *exactly* what is required in terms of control, number of valves, level sensing, data logging, sequencing, special features, to allow the electronic engineer to program and test the entire control system before he moves to site.

The decision to fully automate a rapid filtration system should not be taken lightly. To maintain such a system requires specialised skills not easy to come by in remote areas. To reset and recommission an automatic control system after a power failure, for example, may require special computer knowledge outside the experience of the operators. Even a simple operation such as changing the backwash time from three to four minutes, may now require a programmer with intimate knowledge of the software of the particular plant. It is therefore important that the designer and client thoroughly analyse the needs of a specific plant, in terms of process as well as operational and maintenance considerations, before the degree and type of automation are decided on.

12.5 SLOW SAND FILTER DESIGN

Slow sand filtration is a much simpler technological process than rapid sand filtration, which makes it an attractive option for communities where sophisticated construction and operation would be difficult to guarantee. The process has rather stringent raw water requirements, as is indicated in Chapter 3. Not many raw water sources in South Africa are suitable candidates for slow sand filtration.

The first key design decision is to fix the hydraulic loading, as it determines the total area required for the filtration plant. This area is substantial (compared to rapid sand filtration) and has a large influence on land selection and capital costs. Slow sand filtration rates commonly vary between 0,1 m/h and 0,4 m/h. The total filtration area is then divided into a number of cells. This decision depends on the available means for scraping a filter (a large cell with low scraping capacity will demand a longer filtration interruption for that cell) and on the degree of standby required.

The second key decision deals with the filter medium, which also has a significant influence on the capital costs. A general recommendation is that sand should have an effective size of 0,25 mm to 0,40 mm, with a uniformity coefficient of less than 2. The use of local sand would obviously be preferable, but may require testing or special preparation before it could be used. The media depth must be selected to allow a specified number of scrapings before the media bed must be replenished.

The hydraulics of the system is determined next, to allow even flow distribution to all cells while maintaining a layer of water on the filter sand during filtration. For scraping, filters need to be drained completely and quickly. Special attention should be paid to the underdrain filtrate collection system. A few ancillary items will be required to control the flow pattern through the system, to measure rudimentary flow and to indicate head loss.

Exact quantitative guidelines and procedures for slow sand filtration fall outside the scope of this handbook, because the process is not very commonly used in South Africa, and because the practical details of the process are adapted in most cases to allow the maximum use of local materials, skills and labour. There has been a resurgence in the international interest in slow sand filtration, and readers are advised to make use of the valuable sources that became available in the past few years, for example the guidelines of the American Society of Civil Engineers (1991) and the American Water Works Association Research Foundation (1991).

12.6 SERIES FILTRATION

Series filtration requires two filtration stages, one after the other. The final stage is always a conventional downflow rapid gravity filter. The first stage could be either an upflow filter (in which case the combination is also called the upflow-downflow system (Schulz & Okun, 1984), or a downflow filter (in which case the combination is called the dual-stage filtration system (Brigano *et al.*, 1994)). A summary of the application and performance of the upflow-downflow system (Schulz & Okun, 1984) shows that series filtration is a simple and economic treatment method for smaller treatment plants in developing communities, with examples from Brazil, Costa Rica and India. Recently, it was also demonstrated from a small community in the United States (Brigano *et al.*, 1994) that the dual-stage filtration system is not only highly effective, but also significantly cheaper than alternative treatment methods. In recent years, a number of these filters have also been constructed in South Africa (Van der Merwe *et al.*, 1995) and have proved to be successful and economical for South African conditions.

In all these reported cases, the first filter is functionally characterised as a *contact clarifier*, where flocculation takes place and a part of the sediment load is captured. The cost savings are therefore mainly due to the fact that conventional flocculation and settling facilities are replaced by the first filter. Because the treatment plants are small, a further cost saving is achieved by using prefabricated modules, typically vertically stacked concrete pipes, for constructing both filtration stages.

The series filters used in South Africa are of the upflow-downflow type. The upflow filters are designed for hydraulic loadings in the range of 5 to 7 m/h, with multiple layers of media, of which typical details are given in Table 12.3. The downflow filters are de-

TABLE 12.3
TYPICAL MEDIA SPECIFICATION FOR SERIES FILTRATION IN SOUTH AFRICA

	Media depth (m)	Media size (mm)
Upflow filter		
Layer 1 (bottom)	0,15	6 - 12
Layer 2	0,15	4,0 - 6,0
Layer 3	1,50	3,0 - 4,5
Layer 4	0,25	1,5 - 3,0
Layer 5 (top)	0,25	1,0 - 1,5
Downflow filter		
Layer 1	1,20	0,5 - 1,0

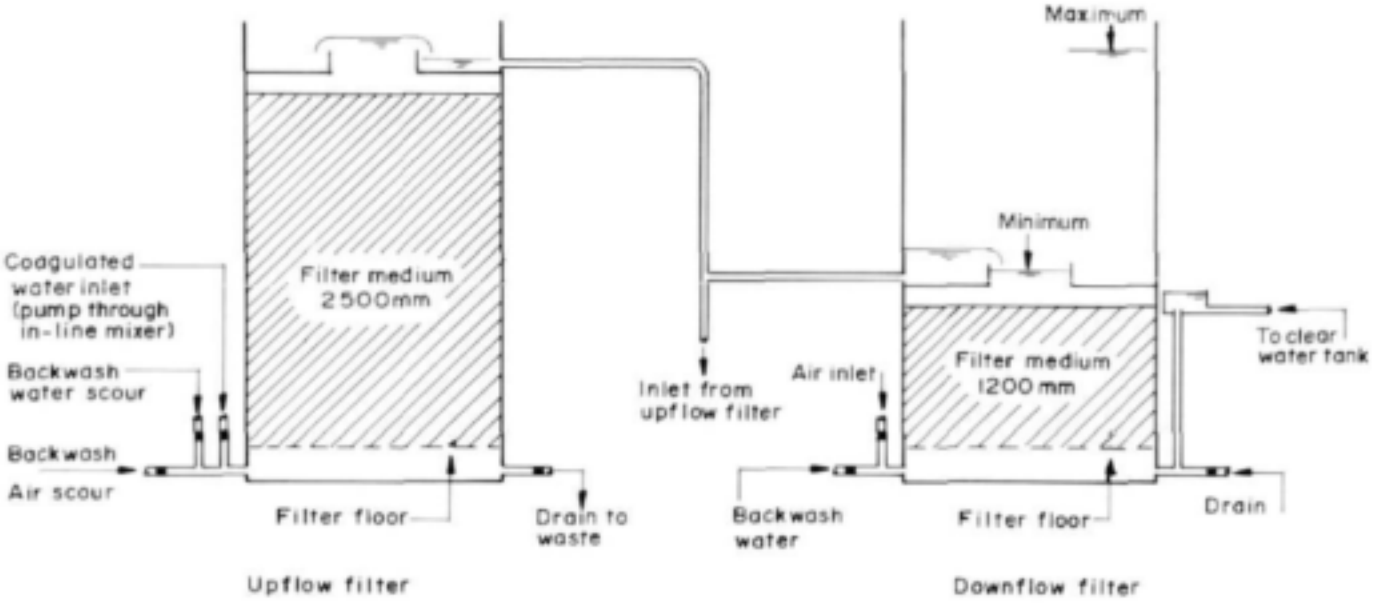


Figure 12.4
Hydraulic control of series filters

signed for higher hydraulic loadings, in the range of 7 to 14 m/h. To simplify construction, upflow and downflow filters have the same diameters; different upflow and downflow rates are achieved by connecting unequal numbers of upflow and downflow filters.

The hydraulic control of the filter series is shown in Figure 12.4. The total clogging head loss allowed for is typically 3,5 m. For the upflow filter, 2,0 m is allowed for, with the remaining 1,5 m for the downflow filter. The upflow filter is essentially a declining-rate filter; with a clean bed, the raw water pump will deliver more than when the bed is partially clogged. The fluctuations in pumping rate are kept to the minimum with careful selection of a raw water pump with appropriate H-Q characteristics.

Backwashing of the upflow filter is done with raw water in six steps. The first step consists of scouring with air at a rate of about 25 m/h for approximately 1 minute. The second step, an innovation developed in South Africa, is a *rapid* draining step. By providing adequately sized scour valves at the bottom of the upflow filters, and by rapidly opening them, the downward surge of water will slough off a significant fraction of the deposits in the media. In the third step, the water level in the filter is restored to the overflow weir. The fourth step is again an air scour step as before while the fifth step consists of backwashing with water at a rate of 70 m/h for about 5 minutes or until the water is clear. During the sixth and final step, upflow filtration is resumed, but at least one bed volume is filtered to waste to displace the uncoagulated washwater in the bed. (The last step is only required when the upflow filter is washed with raw water. If washed with final water, it could be omitted.)

The downflow filters are backwashed conventionally with air at a rate of 25 m/h for about 3 minutes first, followed by backwash with final water at a rate of 28 m/h for 5 minutes.

The series filtration system requires substantial valve operation during a backwash cycle, and correct manual operation asks for

TABLE 12.4
FULL-SCALE APPLICATION OF SERIES FILTRATION IN SOUTH AFRICA

Location	Source start-up	Year of (m/h)	Upflow (m/h)	Downflow (Mℓ/d)	Capacity
Magaliesburg	Blaauwbank River	1986	5	10	0,5
Marloth Park	Crocodile River	1987	5	7	0,8
Hectorspruit*	Crocodile River	1990	7	14	1,5
Burgersfort*	Spekboom River	1994	7	7	2,0

* Plant designed with backwash water recovery system

a level of skill and experience which is often not available in small, developing communities. Considerable effort has therefore gone into the development of a simple, robust electro-pneumatic sequencing and control system which will complete the entire backwashing operation once initiated by the operator. Chemical mixing is done immediately upstream of the upflow filter by an in-line static mixer.

The upflow-downflow system has been implemented at full scale at a number of locations in South Africa, of which a summary is given in Table 12.4. In all cases, the plants were constructed for the local government affairs council; a body charged with providing municipal services for some 50+ small communities in the northern provinces of South Africa. The capital costs of the plants were 20% to 50% lower than the estimated costs for conventional flocculation, settling and filtration.

13

Stabilisation

H Wiechers

Most waters from water purification works require some form of treatment before discharge to a distribution system. The chemical quality of the water entering the system will have some impact on the system, ranging from negligible to severe. Waters of appropriate quality have minimal impact, and the systems have a long life expectation; waters of inappropriate quality can have a major impact and can reduce the life expectation of the systems substantively. There are three major types of impact:

- The water may cause extensive **precipitation** of calcium and iron minerals on the walls of the conduits; the carrying capacity of the conduit may even be reduced severely.
- The water may dissolve the cement matrix in asbestos cement and concrete conduits and water retaining structures, causing destruction of the integrity of the material. This effect is termed **aggression**.
- Electro-chemical reactions between the water and metal components of the system may give rise to dissolution of the metal and precipitation of metal salts resulting in, for example, pitting, nodule formation, red water and finally destruction of the conduit. These effects are termed **corrosion**.

Economic costs in maintaining distribution systems have not been estimated countrywide in South Africa, but must be substantial. Heynike (1987) has reported on the economic effects of the salination of the Vaal River water and estimated that the additional cost that could be expected for all water users if the salt content of the water supply increased from 300 to 800 mg/ℓ would be R253 x 10^6/a. Loewenthal *et al.* (1986) have reported that aggression and corrosion, and to a lesser extent precipitation, in water distribution systems, are widespread. In many instances these effects can be nullified or minimised by relatively simple corrective chemical treatment.

Extensive research has been conducted into the chemistry of water in the treatment of water supplies. Possibly the most influential contribution has been that of Langelier (1936). He recognised the importance of the carbonate system in establishing pH in water, and the solubility of the calcium carbonate mineral in establishing a state of under- or oversaturation. He linked these states to aggressive/corrosive and non-aggressive/non-corrosive states, respectively. He proposed that the water be treated to a state of oversaturation (supersaturation) which would cause a layer of calcium carbonate to precipitate out on the walls containing the water, thereby providing a protective film. To assess the saturation state he developed an index, the Langelier Saturation Index (LSI), based on equilibrium chemistry in terms of the calcium and alkalinity concentrations and pH. He proposed criteria in terms of the Index to prevent corrosion and aggression. Chemical treatment to adjust water to a desired state of non-aggressiveness/non-corrosiveness is referred to as stabilisation.

Although nearly sixty years have passed since the pioneering work of Langelier, his criteria for the protection of distribution systems are still applied. Since Langelier's work extensive research studies have been conducted into the chemistry of the carbonate system, and the chemical response of water in contact with iron metals and cement materials. The influence of other factors, for example chloride and sulphate ions, total dissolved solids, pH, dissolved oxygen and flow velocity, has been delineated (Loewenthal *et al.*, 1986).

Basic information necessary for the design and operation of processes for the stabilisation of water prior to its leaving the water works and entering the distribution systemis presented here. A brief description of the chemistry of precipitation, aggression and corrosion is given, as well as criteria for ameliorating adverse effects. Quantitative procedures are described for estimating the dosages of chemicals to be applied to stabilise water.

13.1 AGGRESSION

Aggression is the designation given to the phenomenon where water contained in cement and concrete lined structures attacks the cement matrix and concrete aggregates. Two distinct aggressive processes have been identified, viz:

- When the water has chemical characteristics that cause dissolution of some of the minerals in the cement matrix. This form of aggression can be prevented by changing the chemical characteristics of the water.
- When the water contains an excessive amount of sulphate species (in excess of 350 mg/ℓ) which will react chemically with some of the minerals in the cement matrix to form products which cause physical deterioration of the cement matrix. To suppress this effect the sulphate content of the water has to be reduced, or, the structures in contact with this type of water need to be lined with a material which is not prone to sulphate attack. Alternatively, new structures can be constructed of special types of cement and high-density concrete.

13.1.1 CALCIUM CARBONATE DISSOLUTION AND PRECIPITATION

The terms undersaturation, supersaturation and saturation with respect to calcium carbonate describe chemical states in a water such that it respectively, dissolves solid calcium carbonate (undersaturation), precipitates solid calcium carbonate (supersaturation) or, neither precipitates nor dissolves calcium carbonate (saturation). Theoretically these states are identified by comparing the activity product of calcium, Ca^{2+}, and carbonate, CO_3^{2-}, species with the solubility product constant for calcium carbonate, K_{sp}. The activity product is given by the product of the active molar concentrations of the calcium and carbonate species, i.e.

$$\text{activity product} = (Ca^{2+})(CO_3^{2-})$$

Accordingly, for dissolution of calcium carbonate (undersaturation),

$$(Ca^{2+})(CO_3^{2-}) < K_{sp}$$

for precipitation of calcium carbonate (supersaturation),

$$(Ca^{2+})(CO_3^{2-}) > K_{sp}$$

and, at saturation,

$$(Ca^{2+})(CO_3^{2-}) = K_{sp}$$

K_{sp} is the thermodynamic solubility product of calcium carbonate. The value of this solubility product depends on temperature, pressure and the type of calcium carbonate mineral precipitated, usually calcite under conditions prevalent in water treatment plants.

13.1.2 SUPERSATURATION REQUIREMENTS FOR NON-AGGRESSION

When a water is distributed in a reticulation system comprised wholly of concrete and/ or asbestos cement pipes, then, if that water is saturated with respect to calcium carbonate, it will remain so, or it will become supersaturated (due to any dissolution of $Ca(OH)_2$ in the concrete) provided there is no generation of carbon dioxide from biological activity in the system. To guard against the development of undersaturated conditions, a slight degree of supersaturation is desirable, a calcium carbonate precipitation potential of 2 to 5 mg/ℓ is recommended (Loewenthal *et al.*, 1986). The calcium carbonate precipitation potential is defined as the mass concentration of $CaCO_3$ that can be precipitated by a supersaturated water to reach saturation.

Where reticulation systems include cast iron or mild steel conduits, it is insufficient to specify a precipitation potential as the sole criterion to guard against metal corrosion. Additional criteria, which include - minimum concentrations of the calcium and carbonate species, and oxygen, maximum concentrations of chloride and sulphate concentrations, and flow velocity, need to be specified. A precipitation potential of 2 to 5 mg/ℓ is recommended (Loewenthal *et al.*, 1986).

13.2 CORROSION AND PASSIVATION

The conduit material in municipal water reticulation systems when made of mild steel or cast iron will be subject to corrosion. This process is the result of oxidation and reduction reactions at sites on the metal-water interface, by the formation of electro-chemical cells each with an anodic and cathodic area. At the anode metal molecules lose their electrons to form metal ions which pass into solution. At the cathode the electrons pass from the metal to some chemical species in the water adjacent to the cathode - this is usually molecularly dissolved oxygen - if present. These reactions induce significant changes in the chemical composition of the water adjacent to the cathodic and anodic areas. The reactions may cause continuous dissolution of the metal at the anode (corrosion), or, may give rise to precipitation of minerals over the anode and cathode, thereby reducing the areas of the active electro-chemical sites, the rates of the reactions eventually stopping the corrosion completely (passivation of the surface).

It is beyond the scope of this chapter to give a detailed account of the mechanisms of corrosion and passivation. For detailed information on this topic the reader is referred to the literature (e.g. Loewenthal *et al.*, 1986; Campbell, 1980; Larson, 1960). However, it is possible to minimise corrosion by adjusting the chemical composition of the water. Below are set out **guidelines for minimising corrosion** proposed by Loewenthal *et al.* (1986):

- The bulk water should be saturated, or slightly supersaturated, with respect to calcium carbonate. Where the criteria for non-aggression for cement and metallic materials listed above are met, this guideline is automatically satisfied.

- Calcium and alkalinity values should each not be less than 50 mg/ℓ (as $CaCO_3$). (Note that the terminology "Alkalinity" is used in this chapter for the so-called total alkalinity or methyl orange alkalinity).
- The Corrosivity Ratio, i.e. $(Cl^- + SO_4^{2-})$/alkalinity in equivalents, must be less than 0,2 for passivation to be possible. This guideline has relatively little value in South Africa because so many of the inland waters exceed it. Therefore, the criterion of the Water Research Centre (1981) is of greater practical value, i.e. that waters be regarded as potentially corrosive when either the chloride or sulphate concentration exceeds 50 mg/ℓ.
- Design conduits in reticulation systems to maintain a water velocity in excess of 0,2 m/s (preferably > 1 m/s) and should be designed dead ends avoided. Where these conditions are not likely to be satisfied, utilise cement type or plastic pipes; if this is not possible, metal pipes lined with inert materials (e.g. paints, tars, epoxies or plastics) should be used.
- The dissolved oxygen concentration should be greater than about 4 mg/ℓ (as O_2).

13.3 WATER STABILISATION AND THE CARBONATE SYSTEM

Conditioning of water to stabilise it involves adjusting pH, alkalinity and/or acidity and calcium concentrations and the $CaCO_3$ saturation state. Calculations to estimate chemical dosages for stabilisation are carried out using equilibrium chemistry in the aqueous phase only. This is done even though it is desirable to supersaturate the water with respect to $CaCO_3$. For a slight degree of supersaturation, in the absence of high concentrations of precipitated slurry, the rate of precipitation will be slow so that one may ignore the precipitation reactions and consider only the single aqueous phase behaviour. However, precipitation from such waters may take place subsequently in the distribution system with corresponding changes in the chemical constitution of the water. Therefore it is necessary to determine, at the stabilisation stage, the expected mass concentration of $CaCO_3$ that can precipitate, and, the aqueous chemical state after precipitation. Such estimates necessarily require analyses involving solid-aqueous-phase equilibria.

The state of over- or undersaturation can be expressed in a number of ways, e.g. by means of the Langelier Saturation Index and the calcium carbonate precipitation potential.

13.3.1 LANGELIER SATURATION INDEX

Langelier (1936) developed a saturation index which, by its sign establishes the state of a water with respect to over- or undersaturation with respect to $CaCO_3$. The Langelier Saturation Index, LSI, is defined as follows:

$$LSI = pH_{actual} - pH_s$$

where

pH_{actual} = measured pH of the water

pH_s = theoretical pH for $CaCO_3$ saturation for the measured alkalinity and calcium concentration of the water, and, in the region $6 < pH < 9$

$pH_s = pK_2' - pK_s' + p[alkalinity] + p[Ca^{2+}]$

[ ] = molar concentration

K_2' and K_s' = apparent equilibrium constants (these "constants" are functions of temperature and the ionic strength of the water. Tables for these constants are published in *Standard Methods*, 1988).

For example, for a water having a pH of 9,00, a calcium concentration of 200 mg/ℓ (as $CaCO_3$), an alkalinity of 60 mg/ℓ (as $CaCO_3$), a temperature of 16°C and a total dissolved solids (TDS) of 650 mg/ℓ, this equation is solved as follows (refer to *Standard Methods* (1988) for the values of $_2'$ and K_s'):

$$pH_s = 2,20 + 9,88 + 2,30 - 1,78 = 8,00$$

and

$$LSI = pH_{actual} - pH_s = 9,00 - 8,00 = +1,00$$

that is, the water is oversaturated with respect to $CaCO_3$ and will have a scale-forming tendency. Langelier (1936) emphasised that the LSI must be considered only as a qualitative measure of over- or undersaturation.

Ryznar (1944) using Langelier's method of calculating pH_s, calculated the Ryzner Stability Index:

$$RSI = 2pH_s - pH_{actual}$$

For natural waters the Langelier Saturation Index can be positive or negative; the Ryzner Stability Index will be positive. For example, for the above water the RSI is 7,00. In either case the correlation of the indices with corrosion must be made by impartial determination of the accompanying corrosion.

13.3.2 $CaCO_3$ PRECIPITATION POTENTIAL

A quantitative measure of the degree of over- or undersaturation is given by the calcium carbonate precipitation/dissolution potential. This parameter defines the mass of $CaCO_3$ to be precipitated from, or dissolved into, a water to attain saturation with respect to $CaCO_3$. The calculation of this potential as well as the chemical dosages required for stabilisation, can be done by using graphical plots, i.e. by using the so-called Modified Caldwell-Lawrence diagrams (Loewenthal *et al.*, 1986) or by computer program, e.g. STASOFT III (Friend & Loewenthal, 1992).

13.3.3 MODIFIED CALDWELL-LAWRENCE (MCL) DIAGRAM

The MCL diagram is a multi-phase equilibrium diagram for carbonate species in the aqueous, solid ($CaCO_3$) and gaseous (carbon dioxide) phases. The diagram has co-ordinate parameters "acidity" and "alkalinity - calcium" and is made up of families of curves representing:

- pH and Alkalinity for equilibrium between species in the aqueous phase, and
- Ca^{2+} values at saturation with respect to $CaCO_3$ for equilibrium between species in the aqueous and solid phases.

The theory for constructing the MCL diagram is set out in detail by Loewenthal & Marais (1976).

An advantage of the MCL diagram is that lines representing the calcium concentration, to give saturation with respect to $CaCO_3$, are superimposed on the diagram. Consequently, the value of the Ca^{2+} line through the intersection point of lines representing measured pH and alkalinity (or pH and acidity, or alkalinity and acidity) defines the Ca^{2+} values for saturation. If the line representing measured Ca^{2+} does not cross the intersection point of the observed pH-alkalinity intersection a condition of either under- or oversaturation is defined from which the potential for dissolution or precipitation can be derived. Thus, in the MCL diagram one can deal with aqueous phase problems, aqueous-solid phase problems, and aqueous-solid-gas phase problems. Procedures for solving water stabilisation problems using MCL diagrams are detailed in *Softening and Stabilisation of Municipal Waters* by Loewenthal *et al.* (1986).

The MCL diagram procedure for solving water stabilisation problems has found practical application in the field. However, although these graphically aided procedures are relatively simple, their application still demands a fairly comprehensive understanding of the calcium-magnesium-carbonate system as found in municipal waters. Also, graphically aided procedures are relatively slow; finding an optimum solution often requires repeated calculations. In order to address these shortcomings of the MCL diagrams Loewenthal *et al.* (1988) developed an interactive user-friendly computer program STASOFT.

13.3.4 COMPUTER PROGRAMS FOR STABILISATION CALCULATIONS

A number of computer programs are available for water stabilisation calculations, e.g. STASOFT (Loewenthal *et al.*, 1988), STASOFT III (Friend & Loewenthal, 1992), AQUACHEM (Anon, 1989) and MINTEQA2 (US EPA, 1991). Since STASOFT III is well suited to the stabilisation calculations to be dealt with in this chapter, as well as being a user-friendly program which is freely available from the Water Research Commission, it will be the program utilised in the rest of this chapter. The use of the program STASOFT III is set out in detail in the Water Research publication entitled *STASOFT III: Computer Program for Chemical Conditioning of Low and Medium Salinity Waters* by Friend & Loewenthal (1992).

13.4 TYPICAL STABILISATION CALCULATIONS

In order to stabilise a water it is necessary to calculate the chemical dosages required to adjust the chemical state of the water to meet the guidelines set out above. To summarise:

(i) Alkalinity and calcium concentrations each to exceed about 50 mg/ℓ as $CaCO_3$, and a pH in the range $6,5 < pH < 9,5$.
(ii) The water to be supersaturated with respect to calcium carbonate with a precipitation potential of between 2 and 5 mg/ℓ as $CaCO_3$.
(iii) The Corrosivity Ratio $(Cl^- + SO_4^{2-})$/alkalinity < 0,2 (where species concentrations are expressed on an equivalent scale); and/or the concentrations of both the Cl^- and SO_4^{2-} species not to exceed 50 mg/ℓ (i.e. guideline for metallic corrosion protection).
(iv) The sulphate species concentration to be less than about 350 mg/ℓ (i.e. guideline for protection against aggression to cement and concrete).
(v) The dissolved oxygen concentration to be greater than 4 mg/ℓ.

All the guidelines can usually be satisfied by appropriate chemical addition and aeration. However, satisfaction of guideline (iii) may not be practicable if either or both of the Cl^- and SO_4^{2-} concentrations are high, since simple chemical conditioning cannot correct this situation. More complex treatment, e.g. desalination may be required. Alternatively, consideration could be given to lining the conduits and structures which will be in contact with this water.

Chemical stabilisation of water for general distribution will be discussed under four categories, i.e. those concerned with waters having :

- adequate calcium and alkalinity,
- calcium and carbonate species deficient,
- excessive calcium concentrations, and
- excessive dissolved carbon dioxide with high calcium concentrations.

The stabilisation of these categories of water will be discussed in detail, using the STASOFT III computer program both to assess the water quality and to select the most appropriate chemical treatment method and chemical dosage type(s). These are illustrated by worked examples.

Before an assessment can be carried out relating to the stabilisation of a water, it is necessary to determine by measurement (or prediction or design) the relevant physical and chemical characteristics of the water to be investigated. The following parameters need to be determined:

- Temperature: This is determined *in situ* or immediately after drawing a water sample; maximum and minimum annual temperatures are required.
- Total inorganic dissolved solids: Determined by standard test (*Standard Methods*, 1988) and used to estimate ionic strength approximately.
- Calcium concentration: Found by standard analytical technique (*Standard Methods*, 1988).
- Carbonate system parameters: Usually these are alkalinity and pH (*Standard Methods*, 1988).

13.4.1 STABILISATION OF WATER WITH ADEQUATE CALCIUM AND ALKALINITY

This category of waters has calcium and alkalinity values that satisfy the minimum criteria set out in Section 13.5, i.e. both the calcium and the alkalinity values are in excess of 50 mg/ℓ as $CaCO_3$; the pH is in the region acceptable for general distribution (i.e. $6,5 < pH < 9,5$). Provided that the corrosivity ratio $(Cl^- + SO_4^{2-})$/alkalinity is not a limiting factor and the water is well oxygenated, it is necessary only to ensure a slightly supersaturated state (with a $CaCO_3$ precipitation potential of about 4 mg/ℓ) to deliver a water satisfying all the criteria for stabilisation. Many of the inland water of South Africa falls into this category.

Two steps are necessary to solve problems of this nature:

- an assessment of the $CaCO_3$ saturation state, and
- if necessary, modification of this state by chemical conditioning to that of the prescribed saturation.

The solution to this type of problem will be demonstrated by various examples.

Example 1

A water with the following composition needs to be stabilised: Alkalinity - 80 mg/ℓ (as $CaCO_3$), Ca^{2+} - 100 mg/ℓ (as $CaCO_3$), pH - 8,6, TDS - 200 mg/ℓ and temperature 20°C.

Using STASOFT III (Friend & Loewenthal, 1992)(or alternatively using the Caldwell Lawrence Diagrams (Loewenthal & Marais, 1976)), select "Data Set" from the main menu, then select "New" from the submenu. Type in the relevant data. Note: date format is year/month/day; and H_2CO_3- Alkalinity is the Methyl Orange Alkalinity or Total Alkalinity. Use the F3 key (print screen) to retain a hard copy of input; then save for future use - give the file a name, say "STAB1.SFT". Now inspect the characteristics of the water, including the precipitation potential by selecting the "View" submenu. This particular water has a precipitation potential of 4,7 mg/ℓ (as $CaCO_3$), hence it meets the requirements as set out above for a stabilised water and hence does not require any further chemical conditioning.

Example 2

A water with the following composition needs to be stabilised. Alkalinity - 80 mg/ℓ (as $CaCO_3$), Ca^{2+} - 30 mg/ℓ (as $CaCO_3$), pH - 8,1, TDS - 200 mg/ℓ, Cl^- - 5 mg/ℓ, SO_4^{2-} - 8 mg/ℓ and temperature 20°C. Determine the saturation state of the water and the chemical treatment required to satisfy all the criteria listed in Section 13.4.

Using STASOFT III to determine the saturation state as set out in Example 1 above, this water is shown to be undersaturated with respect to $CaCO_3$ (i.e. it will dissolve calcium carbonate rather than precipitate it), and has a negative precipitation potential of -3 mg/ℓ (as $CaCO_3$). Before this water can be discharged to the potable water distribution system it is required to satisfy criterion (ii), i.e. the water has to be conditioned to a precipitation potential of 4 mg/ℓ (as $CaCO_3$). In practice the chemicals used to induce supersaturation, normally are lime, $Ca(OH)_2$ or soda ash, Na_2CO_3. Dosage calculations to adjust the saturation state are made using STASOFT III.

Select the menu entitled "Terrestrial Waters"; then select the submenu entitled "Stabilisation"; Enter the desired precipitation potential, i.e. 4,0 mg/ℓ (as $CaCO_3$). Various chemicals can be used for stabilisation, for example lime and soda ash. Start by using lime. The program calculates that 3,0 mg/ℓ of $Ca(OH)_2$ will be required. It also gives the new composition of the water, i.e. pH - 8,80 , alkalinity - 84 mg/ℓ (as $CaCO_3$), Ca^{2+} - 34 mg/ℓ (as $CaCO_3$) and a precipitation potential of 4,0 mg/ℓ (as $CaCO_3$).

Repeat the calculation using soda ash as the conditioning chemical. The program calculates that 8,9 mg/ℓ of soda ash would be required to achieve a precipitation potential of 4,0 mg/ℓ (as $CaCO_3$). The final state of the water will be: alkalinity - 88 mg/ℓ (as $CaCO_3$), Ca^{2+} - 30 mg/ℓ (as $CaCO_3$) and pH 8,89.

Example 3

Water from Hartebeespoort Dam is to be treated for municipal distribution. After treatment with alum and sulphuric acid the water has the following composition just prior to discharge to the distribution system: pH - 7,2, alkalinity - 96 mg/ℓ (as $CaCO_3$), Ca^{2+} - 108 mg/ℓ (as $CaCO_3$), temperature 20°C, TDS - 400 mg/ℓ, Cl^- - 56 mg/ℓ and SO_4^{2-} - 90 mg/ℓ. Establish the saturation state of the water, the dosage of lime required to produce a water with a precipitation potential of 4 mg/ℓ (as $CaCO_3$) and comment on the stability of the distribution water with regard to corrosiveness to cast iron pipes and fittings.

Using STASOFT III the initial state of the water can be calculated; the water is undersaturated and has a negative precipitation potential of -19,6 mg/ℓ (as $CaCO_3$). It will be very aggressive to cement type of materials. Use STASOFT III to calculate the lime dosage required to stabilise the water. It requires 9,5 mg/ℓ $Ca(OH)_2$ to give it a precipitation potential of 4 mg/ℓ (as $CaCO_3$) and will have a final pH of 8,2, alkalinity - 109 mg/ℓ (as $CaCO_3$) and Ca^{2+} - 121 mg/ℓ (as $CaCO_3$).

This water now satisfies the stabilisation criteria (i) and (ii) listed in Section 13.4. Now evaluate if it meets criterion (iii), i.e. is the corrosion ratio, $(Cl^- + SO_4^{2-})$ /alkalinity, less than 0,2, where the species is expressed on a $CaCO_3$ equivalent scale.

Chloride concentration
$= 56\ mg/\ell$ as Cl^-

$$= \frac{56 \times 50}{35}\ mg/\ell \text{ as } CaCO_3$$

$= 80\ mg/\ell$ as $CaCO_3$

Sulphate concentration
$= 90\ mg/\ell$ as SO_4^{2-}

$$= \frac{90 \times 50}{48}\ mg/\ell \text{ as } SO_4^{2-}$$

$= 94\ mg/\ell$ as SO_4^{2-}

i.e. $(Cl^- + SO_4^{2-})$
$= 80 + 94 = 174\ mg/\ell$ as $CaCO_3$

i.e. corrosivity ratio, $(Cl^- + S0_4^{2-})$/alkalinity
$= 174/109 = 1{,}60 > 0{,}2$

Hence the water is likely to be corrosive to mild steel and cast iron. Chemical conditioning to change this ratio to 0,2 would require an alkalinity of 580 mg/ℓ (as $CaCO_3$) which is impractical. Based on the chloride and sulphate concentration of the water, i.e. both in excess of 50 mg/ℓ, one would also expect the water to be potentially corrosive to mild steel and cast iron based on the Water Research Centre (1981) criteria for corrosiveness. Consequently, it would be advisable to use concrete, asbestos cement and/or plastic pipes, or lined metal pipes with inert plastic, resin or cement.

13.4.2 STABILISATION OF CALCIUM- AND CARBONATE-DEFICIENT WATERS

Calcium- and carbonate-deficient waters are identified as having low calcium (soft waters) and low alkalinity values, usually both less than about 20 mg/ℓ (as $CaCO_3$) and low pH, i.e. pH < 6. In South Africa calcium and carbonate species deficiency is found, for example, in waters derived from the Table Mountain sandstone regions on the eastern and southern seaboards (Loewenthal *et al.*, 1986). A typical example is the natural water feeding the Kloof Nek potable water treatment plant in Cape Town, which, after colour removal, has a Ca^{2+} concentration of about 1 mg/ℓ ($CaCO_3$), alkalinity ≈ 0 mg/ℓ (as $CaCO_3$) and pH ≈ 4,8.

Stabilisation of calcium and carbonate species deficient water requires increasing Ca^{2+} and alkalinity concentrations to values satisfying the minimum criteria to control corrosion and aggression, i.e. increasing both to greater than 50 mg/ℓ (as $CaCO_3$), and adjusting pH so that the final pH is in the range $6{,}5 < pH < 9{,}5$ and is supersaturated by about 4 mg/ℓ (as $CaCO_3$). Chemical conditioning to achieve these criteria involves the addition of $Ca(OH)_2$ for Ca^{2+} and alkalinity adjustment, and addition of CO_2 for pH adjustment.

Example 4
Stabilise a water with the following composition: Alkalinity - 1 mg/ℓ (as $CaCO_3$), Ca^{2+} - 5 mg/ℓ (as $CaCO_3$), pH - 5,8, TDS - 40 mg/ℓ and temperature 15°C. Determine the lime and carbon dioxide dosages required to stabilise the water.

Use STASOFT III to do the necessary calculations. Load the water composition, select the "Terrestrial Water" from the menu and then select the submenu "Ca^{2+} + Carbonate deficient water input". In order to have both Ca^{2+} and alkalinity ≥ 50 mg/ℓ, select a "Desired Alkalinity Value" of 50 mg/ℓ. The program calculates that a dosage of 36 mg/ℓ $Ca(OH)_2$ is required. The composition of the water after lime addition will be pH - 11,5, Alkalinity - 50 mg/ℓ (as $CaCO_3$), Ca^{2+} - 62 mg/ℓ (as $CaCO_3$). This water now needs to be stabilised by the addition of carbon dioxide. The program asks whether CO_2 is to be used and the desired precipitation potential. It calculates that the required carbon dioxide dosage is 37 mg/ℓ (as CO_2). The final state of the water is: pH - 9,06, alkalinity - 50 mg/ℓ (as $CaCO_3$) and Ca^{2+} - 62 mg/ℓ (as $CaCO_3$), with a precipitation potential of 4,0 mg/ℓ (as $CaCO_3$).

13.4.3 POST-SOFTENING STABILISATION

After lime-softening a water contains virtually no carbonate species and the pH is too high for potable use. The water can be stabilised by using carbon dioxide, this adds carbonate species and lowers the pH. In adding CO_2 to a high pH water $CaCO_3$ could precipitate from solution. However, if the CO_2 is dosed in a completely mixed reactor and the pH maintained at the lower stabilisation pH value (see example below), no $CaCO_3$ will precipitate.

Example 5
Stabilise the clarified supernatant from a lime-softening plant which has the following composition: Alkalinity - 76 mg/ℓ (as $CaCO_3$), Ca^{2+} - 60 mg/ℓ (as $CaCO_3$), pH - 11,3, TDS - 400 mg/ℓ and temperature 20°C. Determine the pH and CO_2 requirements to obtain a precipitation potential of 4,0 mg/ℓ (as $CaCO_3$) and a pH in the region $6,5 < pH < 9,5$.

Use STASOFT III to do the necessary calculations. Load the water composition, select "Terrestrial Water" from the menu and then the submenu "Stabilisation", enter a "Desired precipitation potential" of 4,0 mg/ℓ (as $CaCO_3$) and "Desired CO_2 dose". The program then calculates the required CO_2 dose and composition of the stabilised water: CO_2 dose - 73 mg/ℓ (as CO_2), precipitation potential - 4,0 mg/ℓ (as $CaCO_3$), pH - 8,71, alkalinity - 76 mg/ℓ (as $CaCO_3$), Ca^{2+} - 60 mg/ℓ (as $CaCO_3$).

13.4.4 STABILISATION OF WATERS WITH EXCESS DISSOLVED CARBON DIOXIDE

Expulsion of CO_2 from water to the air is of particular importance in water treatment when dealing with water derived from underground sources. For these waters the *in situ* dissolved CO_2 content usually is very high compared to waters in contact with the air. When pumped to surface and exposed to the air the difference in partial pressure between air and water creates a driving force for CO_2 expulsion from the water. Carbon dioxide expulsion causes the pH to increase, and, depending on the dissolved calcium content, the solubility product for $CaCO_3$ may be exceeded whereupon $CaCO_3$ precipitates. This may cause severe scaling and narrowing of pipe diameters, and turbid effluents with high $CaCO_3$ suspended solids content. Whereas equilibrium between species in the aqueous phase is virtually instantaneous, when two or more phases, aqueous, gas and solid, are present, the rate to equilibrium between phases is governed by physico-chemical factors in the water body and is usually relatively slow. For example, the rate to equilibrium between dissolved carbonate species and CO_2 in the air is dependent on the difference in partial pressure and mixing conditions; the rate of equilibrium attainment between the solid $CaCO_3$ and the aqueous Ca^{2+} and CO_3^{2-} ions is dependent on the solid surface area and the degree of supersaturation. In consequence, an underground water, when brought to the surface, will lose CO_2 to the air and precipitate $CaCO_3$ out of solution in varying degrees and rates depending upon the specific physical conditions.

- If the water is exposed to the air for a relatively brief period before entering a closed conduit system, CO_2 expulsion will be limited, and depending upon the Ca^{2+} content and the pH in the exposed water, a state of supersaturation will be induced. If the supersaturation is not too high, precipitation may not occur or be limited. If sufficiently high, precipitation as dispersed $CaCO_3$ solids may take place in the water or solid precipitant scale may form on the conduit walls. Once $CaCO_3$ saturation is re-established no further precipitation can take place even though the water is still supersaturated with respect to the CO_2 in the air because the water in the pipe is sealed from contact with the air.

- If the water is transported in an open channel, CO_2 exchange will take place until equilibrium with CO_2 to the air is established. The loss of CO_2 will raise the pH, and if sufficient Ca^{2+} is present, the apparent solubility product for $CaCO_3$ will be exceeded and $CaCO_3$ precipitated out of solution. The rate of precipitation will be determined by a complex set of interacting conditions between CO_2 loss, pH changes, $CaCO_3$ precipitation potential and crystal surface available. However, what is certain is that over a length of the channel eventually a three-phase equilibrium condition will be established whereupon precipitation will cease.

Example 6

Analysis of an underground water pumped to surface gives the following composition: Alkalinity - 320 mg/ℓ (as $CaCO_3$), Ca^{2+} - 210 mg/ℓ (as $CaCO_3$), pH - 7,4, TDS - 800 mg/ℓ and temperature 15°C. The water is exposed to the air. Determine the new chemical state of the water, the calcium carbonate precipitation potential and the mass of carbon dioxide expelled to the air if (a) CO_2 equilibrium is attained with the air and no $CaCO_3$ precipitation occurs during the CO_2 exchange but can occur after the water is isolated from the air, and (b) CO_2 equilibrium is attained relatively slowly and $CaCO_3$ precipitation takes place simultaneously.

Use STASOFT III to do the necessary calculations. Load the water composition, select the "Terrestrial Water" from the menu, and then the submenu "Underground Water".

- The program predicts that 38 mg/ℓ CO_2 will be expelled from the water; the chemical state of the water will be: pH - 8,93, alkalinity - 320 mg/ℓ (as $CaCO_3$), Ca^{2+} - 210 mg/ℓ (as $CaCO_3$) and this water will have a precipitation potential of 60 mg/ℓ (as $CaCO_3$);

- The program calculates the following final state: pH - 8,58, alkalinity - 137 mg/ℓ (as $CaCO_3$), Ca^{2+} - 27 mg/ℓ (as $CaCO_3$), precipitation potential 0 mg/ℓ (as $CaCO_3$), $CaCO_3$ precipitated - 183 mg/ℓ, CO_2 expelled - 245 mg/ℓ (as $CaCO_3$).

14

Disinfection

W A Pretorius

The major industrial development in South Africa occurred in the Pretoria, Witwatersrand and Vereeniging area, in Gauteng Province, at a relatively high elevation. Water for this region is augmented with good quality water imported over great distances (Department of Water Affairs, 1986). Effluents discharged in Gauteng eventually constitute a major component of the raw water sources for users downstream. The Department of Water Affairs and Forestry is committed to the theory of multiple barriers or multiple points of control between effluent discharge and a water supply intake.

The indirect reuse of a portion of treated wastewater relies on the implied, but unproven natural safeguards of dilution, time, and distance (Rogers & Lauer, 1986). The factors that militate against and diminish the effectiveness of these natural safeguards are: increased population, increased mobility of population, increased effluent discharge, and increased water use (White, 1992). Reliance on natural safeguards should be maintained as long as possible. Where the natural safeguards are eroding, increased emphasis and reliability requirements must be placed on the artificial barriers of treatment processes.

The purpose of water purification for potable use is to produce an aesthetically acceptable, chemically stable, safe and wholesome to health, water. Each process step to produce such a water can be regarded as conditioning the water for an effective and reliable disinfection (Water Research Centre, 1989). To minimise the health risks involved, the design engineer should always endeavour to select the raw water source with the highest sustainable chemical and biological quality (Schulz & Okun, 1984). To assist the design engineer in selecting the quality of a water source suitable for domestic use, the DWAF has compiled the essential water quality parameters in a comprehensive guideline (DWAF, 1993).

Consideration is given here to some general parameters affecting disinfection with special reference to the design of a disinfection facility using chlorine as disinfectant.

14.1 General Aspects of Disinfection

The design engineer should take cognizance of some important aspects pertaining to disinfection. These aspects include the type of pathogen and the purpose of indicator organisms.

14.1.1 Biological aspects

Microbiologically polluted water has long been associated with the transmission of infectious diseases such as gastroenteritis, amoebiasis, giardiasis, salmonellosis, dysentery, cholera, typhoid fever and hepatitis A (Craun, 1986). Infectious diseases occur as the result of interactions between pathogenic (disease-producing) micro-organisms and the host (Pelczar *et al.*, 1986). In the production of potable water, all water-borne organisms but especially water-borne pathogens are of concern. The majority of these patho-

TABLE 14.1
SOME COMMON WATER-BORNE DISEASES PREVENTED BY DISINFECTION

Bacterial	Viral	Parasitic
Typhoid fever Para-typhoid Childhood bacterial diarrhoeas Cholera	Hepatitis Rotavirus diarrhoea	Amoebiasis Giardiasis *Cryptosporidium*

gens affect the gastro-intestinal tract and can be members of bacteria, viruses, protozoa and sometimes fungi (Carlson, 1991). For a historical and present-day etiology of some of these pathogens, the reader is referred to White (1992).

Disinfection is the process of destroying or inactivating pathogenic organisms. It is not the same as sterilisation, which is the killing of all forms of life (Pelczar *et al.*, 1986). The sole purpose of disinfection of potable water is to destroy pathogenic organisms and thereby eliminate and prevent water-borne diseases (White, 1992). Some of the most common water-borne diseases prevented by disinfection are shown in Table 14.1 (USAID, 1992).

14.1.2 INDICATOR ORGANISMS

Given the diversity of potential water-borne pathogens, it is not practical to test for the absence of these organisms in potable water prior to its consumption. Testing for most pathogens requires specialised equipment, is expensive and time-consuming. The problem is complicated by the fact that various pathogens actually lead to an outbreak of disease only after long (14 days for typhoid fever) incubation (Carlson, 1991). It is therefore common practice to monitor and control microbiological water quality on the basis of numbers of indicator micro-organisms, rather than of pathogens themselves. A number of common indicator micro-organisms and their detection and uses, are listed in Table 14.2.

Traditionally a drinking water was declared to be microbiologically acceptable when *E. coli* or coliform bacteria were not detectable in 100 mℓ (Carlson, 1991). This guide has been extended to include the frequency of sampling and size of the community served (Tetlow & Hayes, 1988). Pieterse (1989) discussed the question of drinking-water quality criteria in South Africa and proposed a three-tier system for setting maximum concentration levels for determinands (pollutants) for no risks, insignificant risk and low risk to the consumer. To quantify the risk to the consumer, he proposes the microbiological criteria as shown in Table 14.3.

It should be noted that there is no observed relationship between the bacterial indicators and the presence of viruses, nor can enteroviruses or coliphages be used as virus indicators with our present knowledge (White, 1992). The same is true for protozoa. They form cysts that are more resistant to disinfection than bacterial indicators (Rubin *et al.*, 1983). *Cryptosporidium* and *Giardia lamblia* are the most common/problematic protozoa in drinking water. In the United States the inactivation of the protozoan *Giardia* has been used as indicator for measuring the efficacy of disinfection (Teefy and Singer, 1990).

TABLE 14.2
SOME COMMON INDICATOR BACTERIA, THEIR DETECTION AND USES IN MANAGEMENT (DEPARTMENT OF WATER AFFAIRS AND FORESTRY, 1993)

Indicator	Detection and use in water quality management
Standard plate count/ heterotrophic plate count organisms	Used for monitoring of efficiency of water treatment and disinfection processes, aftergrowth in water distribution systems, and the quality of swimming pool water
Total coliform bacteria (total coliforms)	Used to evaluate the general sanitary quality of drinking water and related waters, e.g. swimming pool water.
Faecal coliform bacteria (faecal coliforms)	Indicator of probable faecal pollution of water since this group is much more closely associated with faecal pollution than the broader total coliform group. Some faecal coliforms may not be of faecal origin. Used to evaluate the quality of wastewater effluents, river water, seawater at bathing beaches, raw water for drinking water supply and recreational waters.
Escherichia coli (E. coli)	Used to evaluate the possible faecal origin of total and faecal coliforms, usually when these are isolated from drinking water.
Faecal streptococci/ enterococci	Enterococci comprises a subgroup of faecal streptococci, being considered to include predominantly faecal streptococci of proven faecalorigin. Used in evaluation of treatment processes and recreational waters.
Clostridia (presumptive *Clostridium perfringens*)	These bacteria produce spores which are highly resistant to treatment processes and other unfavourable conditions. Used to indicate remote faecal pollution and to assess efficacy of treatment and disinfection processes.

TABLE 14.3
PROPOSED DRINKING-WATER CRITERIA FOR SOUTH AFRICA

Microbial determinant	Unit	A	B	C
Standard plate count	per 1 mℓ	< 100	1 000	10 000
Total coliform	per 100 mℓ	0	5	1001
Faecal coliform	per 100 mℓ	0	1	10
Clostridium perfringens	per 100 mℓ	0	10	100
Coliphages	per 100 mℓ	0	10	100
Enteric viruses	per 10 ℓ	0	1	10

A = Recommended limit (maximum limit for no risk)
B = Maximum permissible limit (maximum limit for insignificant risk)
C = Crisis limit (maximum limit for low risk)

14.1.3 DISINFECTION METHODS IN THE PRODUCTION OF POTABLE WATER

Micro-organisms can be removed, inhibited or killed by various physical processes, physical agents or chemical agents (Pelczar *et al.*, 1986).

- **Physical processes**: Includes gravity separation and filtration. Gravity separation (sedimentation and flotation) and filtration play a very important role in removing viruses (White, 1992), schistosomes (bilharziasis) (Cairncross & Feachem, 1983) and protozoa cysts. These processes are discussed elsewhere. Protozoan removal is very dependent upon the state and operation of sand filters.

- **Physical agents**: Includes heating (boiling) and irradiation. Boiling of water is practical only in small-scale applications. Ultraviolet irradiation as a disinfection method is gaining popularity in the potable water industry (Hiisvirta *et al.*, 1993; Kruithof *et al.*, 1992). For more details on the process the reader is referred to Potapchenko & Savluk (1991), Masschelein (1992) and White (1992).

- **Chemical agents**: These are by far the most popular disinfectant agents in the water industry and many alternatives, each with its particular application are available. The most commonly used chemicals are: chlorine gas (Cl_2), calcium hypochlorite [$Ca(OCl)_2$], sodium hypochlorite ($NaOCl$), chlorine dioxide (ClO_2), chloramines(NH_2Cl; $NHCl_2$), ozone (O_3), hydrogen peroxide (H_2O_2), potassium permanganate ($KMnO_4$), iodine (I_2) and bromine (Br_2). While chlorine and chlorine compounds have been historically the most popular chemical disinfection agents, the special properties of ozone have caused a rapid increase in its use world-wide (White, 1992). Most of these installations are primarily for taste and odour control, and colour removal. They are almost without exception backed up by chlorination.

Some of these chemical agents (disinfectants) have mainly been applied to large water works, and others to small water works. In small-scale applications, safety and ease of handling are considered more important than costs. The use of chemical disinfectants according to plant size is shown in Table 14.4 (USAID, 1992).

On large waterworks chlorination is automatically controlled. On small plants a variety of dosing methods is used, each with its unique application. For details on small plants the reader is referred to Water Research Centre (1989), USAID (1992) and DeMers & Renner (1993).

Of the four disinfectants used in large water works it is only chlorine that is gener-

TABLE 14.4
PLANT SIZE APPLICATION OF SOME DISINFECTIONS

Type of disinfectant	Application to:	
	Large plants	Small plants
Chlorine gas	X	
Chlorine dioxide	X	
Chloramine	X	
Ozone (in conjunction with a post-disinfectant)	X	
Hypochlorites		X
Iodine		X
Bromine		X
Mixed oxidant gases		X

ally generated off-site and then imported as a liquified gas (White, 1992). Chlorine dioxide, chloramines and ozone are quite unstable and are usually generated at the point of application (Masschelein, 1992; White, 1992).

14.1.4 HEALTH ASPECTS OF CHEMICAL DISINFECTION

Since the turn of this century, chlorination has been the universal disinfectant used for the potable water (White, 1992). The dominant role of chlorination has seriously been challenged since the discovery of the presence of organo-halogenated compounds in chlorinated drinking water (Rook, 1974; Bellar *et al.*, 1974). The best-known of these products are the trihalomethanes (THMs) of which chloroform ($CHCl_3$) has been found to be an animal carcinogen (Pieterse, 1988). This has lead to the abandonment of the use of chlorine in some industrialised countries (like the Netherlands) with a corresponding increase in the use of alternative disinfection methods and chemicals (Wondergem & Van Dijk-Looijaard, 1991; Kruithof *et al.*, 1992). In the Netherlands microbiological regrowth in the reticulation systems is being controlled by the elimination of growth-promoting substances in the water. According to Van der Kooij (1992) a low AOC (assimilable organic carbon) content can be obtained, resulting in the chances for regrowth being low.

None of these alternative methods are, however, entirely devoid of problems. Many studies have shown that potentially hazardous chemical by-products are formed with virtually every type of disinfectant method or chemical used (Jacangelo *et al.*, 1989; Lykins *et al.*, 1990; Lykins *et al.*, 1992). After a comprehensive review on the potential health risks of THMs, Pieterse (1988) concluded that the removal of microbiological contamination remains the most important consideration in ensuring the safety of potable water. Chlorination should not be phased out in preference to other methods until the potential dangers of alternative disinfectants are fully evaluated.

Considerable research has been done on the possible formation, types, and prevention of disinfection by-products (Stevens *et al.*, 1976; Jolley *et al.*, 1977; Krasner *et al.*, 1989; Duke *et al.*, 1980):

- **Formation**: Disinfection by-products are formed due to the action of the disinfection chemical on naturally occurring organic matter, such as fulvates and humates that constitute natural organic colour in surface waters.

- **Types**: The most common disinfection by-products include: trihalomethanes, haloacetonitriles, haloketones, haloacetic acids, chlorophenols, aldehydes (and many more). The latest scare is bromate formation when ozone is used (Kruithof & Meijers, 1994). This view has been superseded by the realisation that chlorination also results in the bromate formation.

- **Prevention**: The four approaches to minimising or preventing the formation of especially THMs are:
  - Removal of the THM formation potential (THMFP) by improving organic removal. In some cases it may be justified to lower the pH for better coagulation of organics. Biological removal of organic substances on filter or granular activated carbon (GAC) is increasingly used.
  - Control of the concentration and contact time of free chlorine. This may be achieved by altering the point of chlorine application - usually to after clarification. In the case of pre-chlorination,

the action of free chlorine can be ended by ammonification.

- Use alternative disinfectants such as chloramines, chlorine dioxide and ozone which do not form or form little THMs.
- Removal of THMFP as well as THMs by activated carbon. From the various methods investigated this method was the least cost-effective.

Bromate formation can be avoided at pH values < 7 and by means of ammonia. The latter is important when using ozone as it will not react with ammonia.

14.1.5 STATUS OF DISINFECTION PRACTICES

In a recent survey (Disinfection Committee, 1992) of the disinfection practices in the USA it was found that most water utilities were making concerted efforts towards reducing THMs in potable water. The most significant change by utilities was to alter the point of application of chlorine, the dosage of chlorine used, and the addition of ammonia. It should be noted that although chloramines are recommended as a primary disinfectant (Longley & Roberts, 1982) and for controlling bacterial regrowth in distribution systems (Neden *et al.*, 1992), it could affect kidney dialysis patients and aquarium fish (White, 1992).

Ozonation is particularly useful and is increasingly used because of its benefits other than disinfection. Ozone is the only disinfectant which is successful in combatting chlorine-resistant protozoa.

In a world-wide survey on disinfection practices (Hiisvirta, 1993) it was concluded that:

- Water-borne epidemic outbreaks still occur today - even in developed countries.
- Outbreaks in developed countries were due to poor disinfection due to fear of chemical by-product formation.
- Chlorination is still the dominant disinfection method throughout the world, with the use of chloramination, ozonation and UV irradiation being on the increase.
- Chlorine dioxide use as a substitute for chlorine is decreasing rather than increasing.

Ozonation is particularly useful and is increasingly being used because of its benefits other than disinfection. Ozone is the only disinfectant which is successful in combating chlorine-resistant protozoan vectors.

From the above it is clear that chlorine is still the most universally used disinfectant for the production of potable water. However, the designer should take note of the developments regarding the potential health hazard of disinfection by-products, and how to minimise or avoid it, as well as the advantages and disadvantages of alternative disinfectants.

14.1.6 KINETICS OF DISINFECTION

The inactivation rate of a bacterial population due to different chemical disinfection concentrations is described by the classic and well-known Chick-Watson law (Fair *et al.*, 1968):

$$\ln \frac{N}{N_0} = k^{'}C^{n}t$$

where:

N_0 = number of bacteria at time, t = 0
N = number of bacteria at time t
$k^{'}$ = rate constant

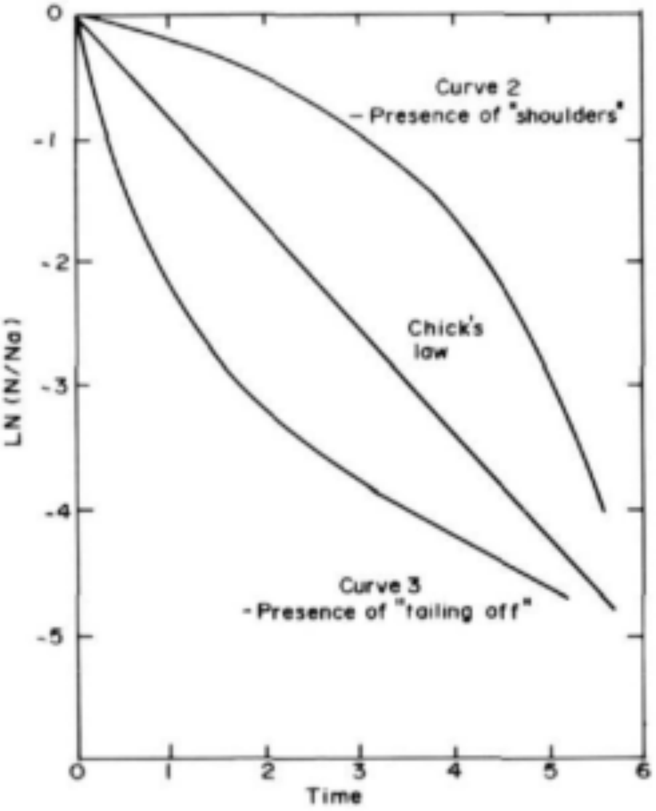


Figure 14.1

The Chick-Watson law and deviations (Haas, 1990)

C = concentration of disinfectant
n = empirical constant

Although the Chick-Watson equation plots a straight line when ln N/N_o is plotted against time, experimental data show more deviations than confirmations of the law (Rubin *et al.*, 1983). Two common types of deviations occur: the presence of "shoulders" or lags until the onset of disinfection action and the exhibition of "tailing" wherein the rate of disinfection progressively decreases (Haas, 1990). These deviations of the Chick-Watson law for constant disinfectant concentration are graphically shown in Figure 14.1.

Severin *et al.* (1984) has proposed a "series-event" kinetic model for analysing these types of inactivation curve.

Disinfection, like all rate processes, is temperature-dependent. This dependency can be represented by the Arrhenius relationship (Haas, 1990):

$$k = k_0 \exp(-E/RT)$$

where:

k = Chick-Watson k' value
T = absolute temperature, °K
R = gas constant, 8,134 J/mol. °K
k_o = frequency factor
E = activation energy, J/mol

The values of E and k_o may be determined from the rates of inactivation obtained as a function of temperature.

14.1.7 THE CONCENTRATION-TIME (C.T) FACTOR

Even if deviations from the Chick-Watson law are observed, plotting combinations of disinfectant concentration and time to produce a fixed percent inactivation, plot linearly on a log-log scale. Such plots tend to follow the relationship $C^n.t$ = constant, where n is a function of the organism, pH, temperature, form of disinfectant, and extent of inactivation. If $n > 1$, a proportionate change in disinfection concentration produces a greater effect than a proportionate change in time.

Potable water may be contaminated by a variety of pathogenic organisms. Owing to this dilemma, it is extremely difficult to choose a test organism, which is easily identifiable, and which has an inactivation rate equal to that of the most resistant pathogen.

The Environmental Protection Agency (EPA) in the USA has developed the so-called "Surface Water Treatment Rules"

(SWTR) for the disinfection of surface water (Teefy & Singer, 1990). The SWTR contains tables of disinfectant-C.t values that have been demonstrated to achieve the required degree of inactivation of viruses and *Giardia* cysts. The methods by which C and t values for a particular application are calculated are very important since factors like pH, temperature and flow pattern in the contact chamber, each plays a role (Rubin *et at.*, 1983; Teefy & Singer, 1990; White, 1992).

The possible adoption of a C.t standard for the disinfection of potable water in South Africa would have a tremendous effect on the proper design of cost-effective disinfection systems (Teefy & Singer, 1990).

14.2 DISINFECTION WITH CHLORINE

14.2.1 DISSOLUTION OF CHLORINE GAS IN WATER

Most of the following section was compiled from White (1992) who made a comprehensive study of chlorination.

When chlorine is dissolved in water, it hydrolyses rapidly according to the following equation:

$$Cl_2 + H_2O \rightarrow HOCl + H^+ + Cl^-$$

The resulting solution of chlorine in a chlorinator discharge is limited by design to 3 500 mg/ℓ which gives a pH <3.

The next most important reaction is the dissociation of hypochlorous acid:

$$HOCl \leftrightarrow H^+ + OCl^-$$

In waters with pH between 6.5 and 8.5 the reaction is incomplete and both species are present to some degree. The extent of this reaction is given by:

$$K_i = \frac{[H^+][OCl^-]}{[HOCl]}$$

where K_i, the dissociation constant and [ ] represents chemical species concentration in moles per litre.

The values of K_i at different temperatures (degrees Kelvin; T) may be calculated from the acid dissociation constant, $pK_a = - \log [K_i]$ and

$$pK_a = \frac{3000,0}{T} - 10,0686 + 0,0253\ T$$

and T = degrees Kelvin

The percentage distribution of OCl^- ion and undissociated hypochlorous acid can be calculated at various pH values as follows:

$$\frac{[HOCl]}{[OCl] + [OCl^-]} = \frac{1}{1 + \frac{[OCl^-]}{[HOCl]}} = \frac{1}{1 + \frac{K_i}{[H^+]}}$$

As in any solution, the activity of ions like OCl^- and H^+ is influenced by the ionic strength of the solution. The activity coefficients of the ions in question can be calculated from a chemical analysis of the water or approximated from the electrical conductivity using the Debye-Hückel or other equations (Benefield *et al.*, 1982). The actual ionisation constant K_i for hypochlorous acid is then given by:

$$K_i = \frac{f_H + [H^+] . f_{OCl^-} [OCl^-]}{[HOCl]}$$

where

f = activity coefficient, and

( ) = activity (moles/ℓ)

The effect of pH and ionic strength on the dissociation of HOCl at 5°C is shown graphically in Figure 14.2 (White, 1992).

The interest in the HOCl : OCl^- lies therein that for various reasons the undissociated HOCl has at the same concentration a substantially higher microbial inactivation rate than the corresponding OCl^- ion. The quantity of HOCl and OCl^- is called the *free available chlorine.*

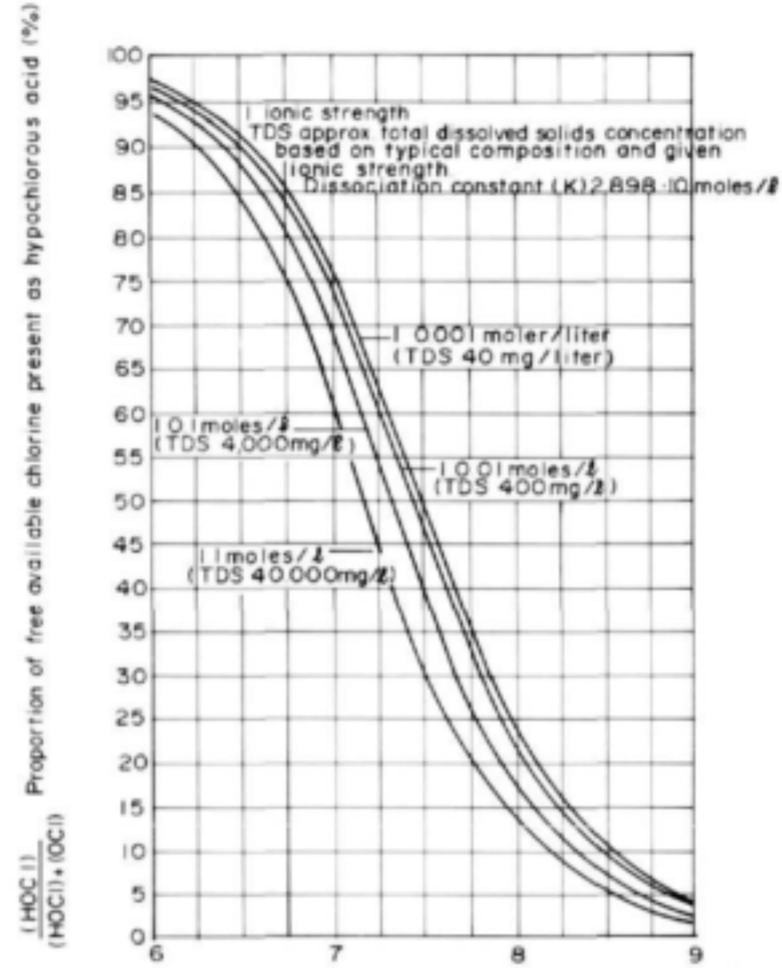


Figure 14.2

Effect of pH and ionic strength on the dissociation of HOCl at 5°C

14.2.2 BREAKPOINT CHLORINATION

In dilute aqueous solutions, the reaction between ammonia and chlorine forms three types of chloramines:

$HOCl + NH_3 \leftrightarrow NH_2Cl$
(monochloramine) + H_2O
$NH_2Cl + HOCl \leftrightarrow NHCl_2$
(dichloramine) + H_2O
$NHCl_2 + HOCl \leftrightarrow NCl_3$
(trichloramine) + H_2O

These reactions are dependent on pH, temperature, initial ratio of chlorine to ammonia and time. For example, all free chlorine will be converted to monochloramine at pH 7 to 8 when the ratio of chlorine to ammonia-nitrogen (NH_3-N) is equimolar (5 : 1 by weight) or less. At 25°C it takes about 420 seconds and at pH 8,3 less than 0,1 seconds to convert 99% of HOCl to NH_2Cl.

The two species that predominate are NH_2Cl and $NHCl_2$. The chlorine in these compounds is called *combined available chlorine.*

In water works practice the maintenance of a residual chlorine (free or combined) for the purpose of disinfection is complicated by the fact that chlorine not only reacts with ammonia. All readily oxidisable substances such as Fe^{2+}, Mn^{2+}, H_2S and organic matter reduce it to the chloride (Cl^-) ion. The reactions with these compounds and ammonia are shown graphically in Figure 14.3 (Metcalf & Eddy, 1979):

Continued addition of chlorine past the breakpoint as shown in Figure 14.3, results in a directly proportional increase in free available chlorine. The main reason for adding enough chlorine to get a free residual is that usually satisfactory disinfection can be

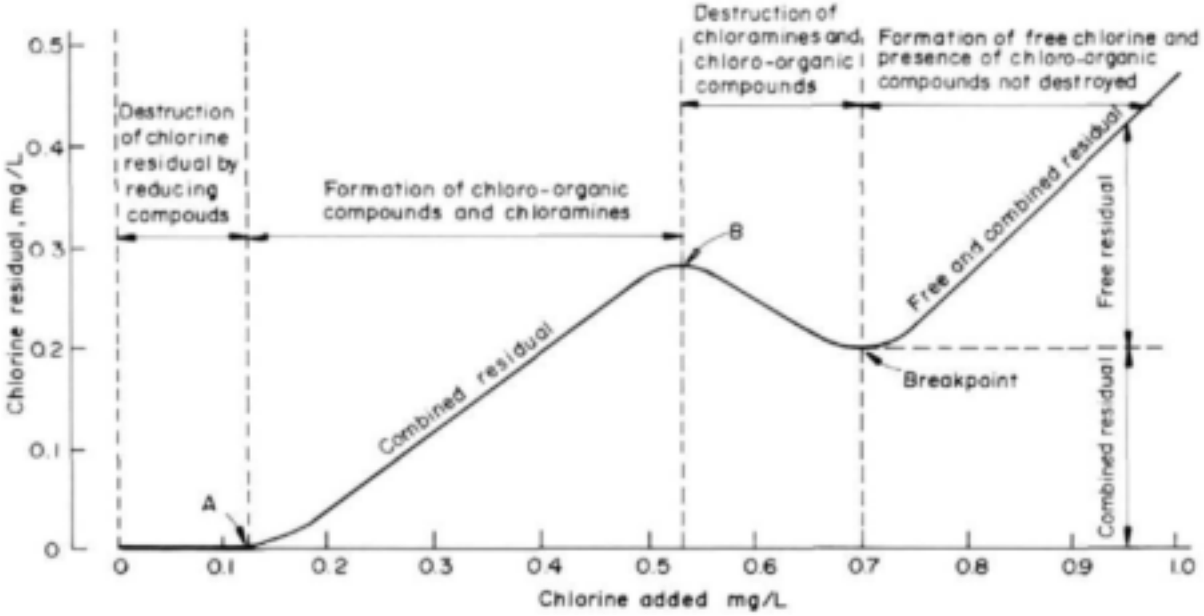


Figure 14.3
Generalised curve obtained during breakpoint chlorination

assured in the contact time designed for.

When chlorine gas is used for breakpoint chlorination, alkalinity is used up. Stoichiometrically, 14,3 mg/ℓ of alkalinity, expressed as $CaCO_3$, will be destroyed for each 1,0 mg/ℓ (NH_3-N) that is oxidised in the chlorination process. The efficiency of the various chlorine forms as disinfectants differs, and thus, the concentration of available chlorine is insufficient to characterise process performance. The inactivation potency of $HOCl : OCl^- : NH_2Cl : NHCl_2$ is approximately 1 : 0,0125 : 0,005 : 0,0166 (Haas, 1990).

Although the immediate disinfection efficacy of chloramines is less than that of chlorine, it has a relatively long half-life (approximately 100 h in distilled water). Because of this property, chloramines are often used in a post-chlorination step to prevent secondary microbial growth in water distribution networks.

14.2.3 Closure

Chlorine, like most of the other chemical disinfectants, is also a strong oxidising agent. A lot of published information exists with regards to its other beneficial properties like taste and odour control, removal of unwanted organics, iron, manganese, and much more. A description of this is beyond the purpose of this book. For more details the reader is referred to White (1992).

14.3 Design of Chlorination Facilities

Chlorination is in essence a chemical process and the laws that govern chemical processes are also applicable in the design of chlorination systems.

According to the Chick-Watson law of disinfection, three major parameters govern chemical disinfection, namely the number of pathogens that must be inactivated, the disinfectant concentration, and the reaction or contact time.

Number of pathogens: The only control that the design engineer has on the number of pathogens is that the engineer should choose the best possible quality of raw water. The higher the number of pathogens, the greater C.t required. Coagulation and flocculation followed by clarification remove a large fraction of micro-organisms.

Disinfectant concentration: The disinfectant concentration (usually measured at the end of contact time) depends on the number of pathogens to be inactivated, the chemicals that can react with the disinfectant, the type of disinfectant, and the contact time. The initial complete mixing of disinfectant and water is imperative.

Contact time: This is probably the most important design parameter since it controls the degree of disinfection achieved, as well as the formation of disinfection by-products.

In the potable water industry there are no established standards or codes of practice for the design of chlorination systems in general and the contact tanks in particular. In most water works the main function of contact tanks is to give temporary storage to treated water, as a buffer between the steady output of the works and the fluctuating demand of the distribution system (Water Research Centre, 1989).

The nearest approach to a general standard for disinfection is the WHO Guideline which recommends that the treated water should have a free chlorine residual of at least 0,2 to 0,5 mg/ℓ after a contact time of 30 minutes (Water Research Centre, 1989). In the USA more attention has recently been given to the control of contact times since the adoption of the "Surface Water Treatment Rule", where C.t values were provided (Teefy & Singer, 1990; Disinfection Committee, 1992).

Most of our knowledge on the design aspects of the disinfection of large quantities of water has come from research on the disinfection of wastewater (Louie & Fohrman, 1968; Marske & Boyle, 1973; Hart, 1979; Sepp, 1981; Haung *et al.*, 1985). Two aspects of disinfection which have a bearing on design are: the effect of initial homogeneous distribution of the disinfectant and especially the approximation of plug flow conditions in the contact tank.

14.3.1 DESIGN CONSIDERATIONS

The primary purpose of disinfection of potable water is to reduce the potential health risk associated with pathogens for the user. Although health risks due to pathogen-contaminated water are probably the most important, risks due to other water quality parameters (see **Water Quality Criteria** elsewhere) should be used as a guide to the degree of treatment required. For the design of chlorine-based disinfection facilities, the criteria shown in Table 14.3 for indicator organisms should be used as basis for determining the required C.t values. Since the time of the C.t factor is a function of the residence time distribution in the chlorine contact tank, reliable C.t factors may only be calculated once the flow pattern in the contact tank is known (USEPA, 1989). The two main additional parameters to be taken in account are:

- pH: HOCl is more effective than OCl (see Figure 14.2).
- Temperature: The lower the temperature, the lower the disinfection efficiency. C.t values determined at 5°C are recommended as reference (USEPA, 1989).

A chlorination system for disinfection of potable water consists of:

- chlorine storage and supply;
- chlorine solution mixing;
- provision of contact time, and
- chlorine dosing control.

14.3.1.1 Chlorine storage and supply

Some important design and safety considerations of chlorine storage and supply are given by Qasim (1985):

- Chlorine storage and chlorinator equipment must be housed in a separate building. If not, it should be accessible from an outside door.
- Adequate exhaust ventilation at floor level should be provided because chlorine gas is heavier than air. Fan control and gas masks should be located at room entrance.
- The temperature in the chlorine supply area should not be allowed to drop below 10°C.
- Dry chlorine liquid and gas can be handled in wrought-iron, SM steel, Hasteloy, Monel alloys and appropriate stainless steel (AISI 318-322); wet chlorine gas in glass, silver, hard rubber (Masschelein, 1992) and plastic. Valves and pipe fittings should be specially designed for chlorine use.
- Chlorine cylinders in use should be set on platform scales set flush with the floor, and loss of weight should be used for record-keeping of chlorine dosages.
- Storage should be provided for at least a 30-day supply.

14.3.1.2 Chlorine solution mixing

One of the most important aspects of disinfection with chlorine is the initial mixing of the relatively small, high chlorine concentration stream with the bulk of the clarified water. The effective initial mixing of the chlorine solution can be accomplished in a number of ways (White, 1992; Harnby *et al.*, 1992):

(a) diffusers;
(b) mechanical mixers;
(c) hydraulic mixers including static mixers, and hydraulic jumps.

Mixing intensities, measured as the velocity gradient, G, should be > 500 s^{-1} (White, 1992).

14.3.1.3 Contact tanks

The chlorine contact tank must be designed in such a way as to provide the optimum distribution of residence time for contact between the disinfectant and the pathogens. The distribution of residence time may differ appreciably in tanks of different geometrical configuration, although tank volumes and flow rates are identical. For disinfection the ideal contact tank should have a configuration that encourages 'plug-flow' characteristics. The flow characteristics of a contact tank can be determined by tracer tests which have become necessary in the USA where the C.t standard has become mandatory (Teefy & Singer, 1990). With the proper use of tracer information it is possible to rationalise (Trussel & Chao, 1977) and optimise (Tikhe, 1976) the design of chlorination systems. For design details of chlorine contact tanks the reader is referred to White (1992).

14.3.1.4 Chlorine dosing equipment

Chlorine is supplied as a liquified gas under high pressure in containers varying in size from 68 kg to 909 kg. Chlorine can be abstracted directly from the gas phase of the pressurised container with a chlorinator. The most widely used chlorinators are those using vacuum-feed devices. In each of these systems, the chlorine injector is the basic component. The injector is used to create the vacuum that is used to draw the chlorine gas from the storage supply through the chlorine regulator, which serves as a metering device, and into the injector. At the injector, the chlorine dissolves in the injector water to form hypochlorous acid. From the injector the hypochlorous acid solution flows to the point where it is to be injected into the clarified water. At liquid chlorine temperatures of 10°C, about 10 kg chlorine/h can be evaporated per 909 kg container (Degrémont, 1991). If higher rates are required, more containers can be used in parallel, or specially heated evaporators installed. A chlorine concentration of 2,5 g chlorine/ℓ (or higher) at the injector can be obtained in this way. For a complete discussion on the equipment available, design calculations and physical layout, the reader is referred to White (1992).

14.3.2 PROPERTIES OF CHLORINE

In its normal state chlorine is a greenish-yellow gas with the following physical constants (Degrémont, 1991):

- Density in relation to air : 2,491
- Relative atomic mass: Cl = 35,45
- Specific mass : 3,214 g/ℓ at 0°C, 760 mm Hg
- At 15°C, 760 mm Hg, 1 kg of chlorine gives 314 ℓ of chlorine gas and 1 ℓ of liquid chlorine corresponds to 456 ℓ of gas.

It liquifies on cooling and compression at a pressure varying with temperature:

- 10 Bars at 40°C or 5 bars at 18°C, 760 mm Hg
- Liquefaction point (at 1 bar): - 34,1°C
- Freezing point: - 102°C
- Critical temperature: 144°C
- Critical pressure: 77,1 bars
- Heat capacity of gas: 0,518 kJ/kg from 15° to 100°C at 1 bar
- Heat capacity of liquid: 0,92 kJ/kg

Solubility and latent heat of evaporation of chlorine varies with temperature. At liquid chlorine temperatures of 10°C, about 10 kg chlorine/h can be evaporated from a 900 kg container.

15

Sludge and Washwater

G E Rencken

In water purification, one of the principal objectives is to remove suspended solids from the water to purify it. "Suspended solids" is a general term and includes clay and fine particulate matter, colloids, organic and inorganic precipitates, as well as algae, bacteria and viruses which are present in the raw water feed.

In a conventional water purification works, suspended solids are removed in clarifiers or sedimentation tanks and sand filters after prior coagulation and flocculation. Filter washwater contains suspended matter removed by the filters.

In South Africa, sludge from clarifiers and washwater from sand filters have conventionally either been disposed of by releasing the sludge into the nearest stream or water course or into evaporation ponds. While evaporation ponds are still a favoured option for many water works in South Africa, because of high evaporation rates and availability of land, disposal of sludges and filter washwater to rivers and water courses is prohibited by law. While it may be argued that by discharging sludge to a river, one is merely returning solids back to a river, it must be borne in mind that the sludge contains coagulants and other chemicals which may be harmful to aquatic life. Sludges and washwater released into water courses may also have a negative aesthetic impact such as localised build-up of sludge deposits and increased turbidity in receiving waters.

The following aspects of sludge and washwater management are discussed:

- properties and quantities of sludges and washwater;
- washwater handling;
- sludge volume minimisation and conditioning;
- sludge thickening;
- sludge dewatering techniques;
- ultimate disposal and potential commercial uses of sludges.

A schematic diagram showing the various treatment options and ultimate disposal methods for sludge and washwater, which are discussed in this chapter, is shown in Figure 15.1.

15.1 PROPERTIES AND QUANTITIES OF SLUDGE AND FILTER WASHWATER

The properties and quantities of sludges and filter washwater produced in a water purification works are a function of the raw water quality and the type of coagulant and flocculant used. An understanding of the properties and quantities of sludges is fundamental to designing facilities to handle or treat the sludge and washwater.

In a conventional water works suspended solids are collected in the clarifiers and captured on the filters after coagulation and flocculation. The properties of the sludge collected in clarifiers and filters depend on the water quality, type and dose of coagulant or flocculant used, efficiency of operation, plant design and other factors. For typi-

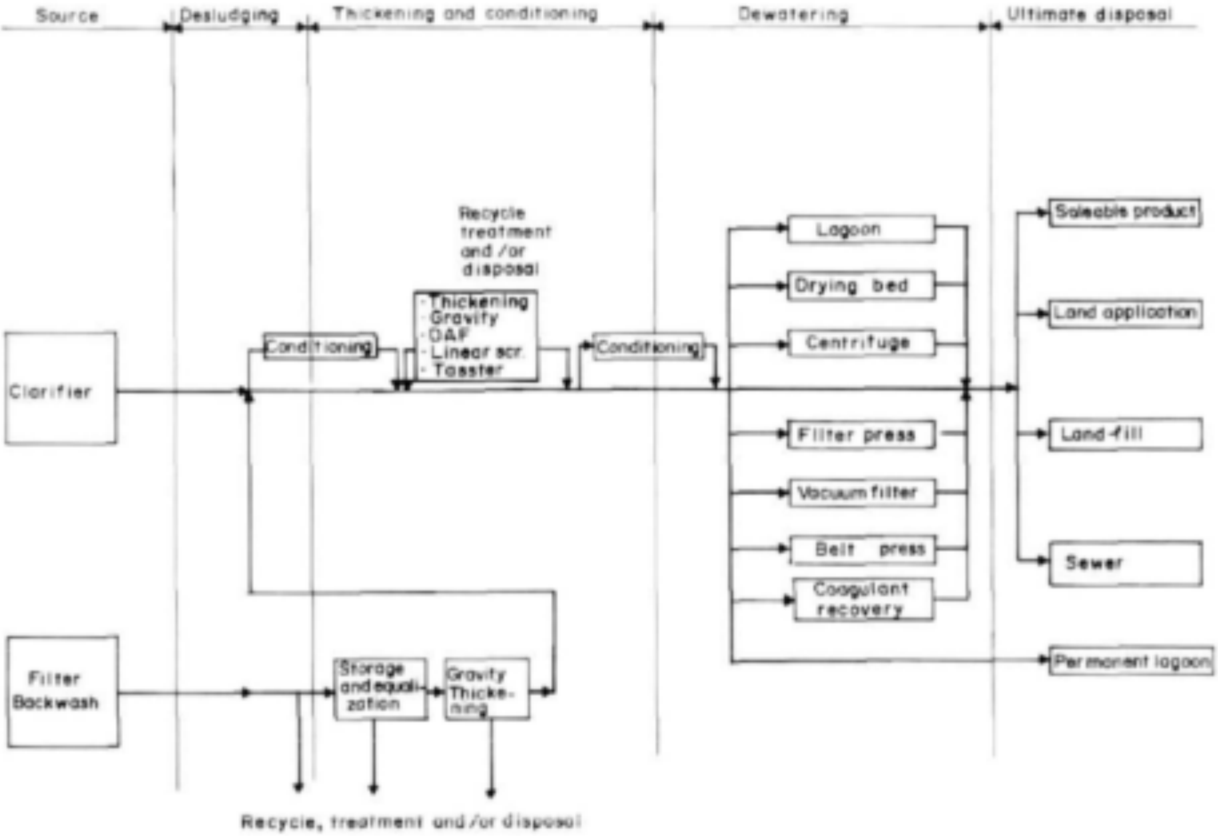


Figure 15.1

Sludge and washwater treatment options and ultimate disposal methods

cal plants using alum as the coagulant, between 60% and 90% of the total sludge will be collected in the clarifiers with the remainder in the filters (Montgomery, 1985).

15.1.1 QUANTITIES OF SLUDGE AND FILTER WASHWATER PRODUCED

For design purposes, the amount of suspended solids produced in a works can be estimated on the basis of the type of coagulant dose rate and raw water quality.

The total suspended solids which will be collected in the sludge from the clarifiers and the filter washwater, may be estimated from the following equation:

$$\begin{pmatrix}\text{Total suspended}\\ \text{solids in clarifier}\\ \text{sludge and}\\ \text{washwater}\end{pmatrix} = \begin{pmatrix}\text{Total suspended}\\ \text{solids due to}\\ \text{natural turbidity}\\ \text{in raw water}\end{pmatrix} + \begin{pmatrix}\text{"Additional"}\\ \text{suspended}\\ \text{solids}\\ \text{from addition}\\ \text{of coagulant}\\ \text{or flocculant}\end{pmatrix} \quad ...15.1$$

The "additional" suspended solids produced by the addition of coagulant or flocculant in Equation (15.1) depend on the type of coagulant or flocculant.

For coagulation with alum or ferric chloride, the "additional" suspended solids consist of precipitated coagulant in the form of aluminium or iron hydroxide, respectively. In the case of alum sludge an empirical formula for the aluminium hydroxide fraction of the "additional" sludge has been determined to be $Al(OH)_3.1,25H_2O$ (Montgomery *et al.*, 1985). Using this formula it can be calculated that a total of 0,33 g of "additional" sludge suspended solids will be produced for each g of alum [$Al_2(SO_4)_3.14H_2O$] added. If a similar proportion of bound water is assumed for iron hydroxide sludges, the value for ferric sulphate [$Fe_2(SO_4)_3$] would be 0,59 g for every g of coagulant (flocculant) added and 0,48 g for ferric chloride [$FeCl_3$].

Geldenhuys (1992) determined the following factors (g of "additional" sludge suspended solids for each g of coagulant added) for "additional" sludge for alum, ferric chloride and ferric sulphate experimentally:

[$Al_2(SO_4)_3.14H_2O$; factor: 0,922;
$FeCl_3$; factor: 0,65;
$Fe_2(SO_4)_3$; factor: 0,54]

For a polyelectrolyte coagulant or flocculant, the amount dosed into the raw water will contribute directly to the "additional" fraction of suspended solids, i.e. for every g of polyelectrolyte added to the raw water a total of 1 g of suspended solids in the sludge (washwater) will be produced.

The determination of the amount of additional sludge produced where hydrated lime is used is more complicated as softening, solubility of calcium carbonate produced and unreactive material all have a significant effect. For Vaal Dam water, approximately 1,873 mg/ℓ of $CaCO_3$ residue is produced from each mg/ℓ of CaO added (Geldenhuys, 1995).

Another useful aid when estimating sludge solids produced by a water treatment process, is that for raw water turbidities < 100 NTU, the suspended solids in mg/ℓ have been shown to be approximately equal to the turbidity in turbidity units (Montgomery, 1985). Caution should, however, be exercised when using this guideline for design purposes. For Vaal Dam water, for example, it was found that the relationship between turbidity as NTU and mg/ℓ of suspended matter is 1:1,25 to 1:1,35 (Geldenhuys, 1995).

The dosing of supplementary chemicals, such as bentonite for raw water conditioning and powdered activated carbon (PAC) for adsorption of taste and odour compounds, also contribute to the suspended solids collected from the clarifiers and filters. For design purposes, it is assumed that for each g of bentonite or PAC dosed, a total of 1 g of sludge solids is produced.

15.1.2 EXAMPLE 1

The raw water flow rate into a water works is 100 Mℓ/d, while the turbidity of the raw water is 50 NTU. Alum is used as a coagulant and is dosed at a rate of 40 mg/ℓ. Powered activated carbon is dosed at 5 mg/ℓ to remove tastes and odours from the raw water. Estimate the total dry solids production in the sludge (washwater) from the clarifiers and filters on a daily basis.

Solution

From the raw water turbidity we may estimate the suspended solids concentration in the raw water to be 50 mg/ℓ. (This is only an estimate. For an accurate design, it is essential to determine the raw water suspended solids from a laboratory analysis).

The total sludge produced due to "natural" turbidity or suspended solids in the raw water (Montgomery, 1985) = 50 kg/Mℓ x 100 Mℓ/d = 5000 kg/d. (As a comparison the estimate for Vaal Dam water = 50 kg/Mℓ x 100 Mℓ/d x 1,35 = 6750 kg/d).

The dose rate of alum is 40 mg/ℓ or 40 kg/Mℓ.

The total "additional" sludge (suspended solids) produced due to alum dosing according to Montgomery (1985):

= 40 kg/Mℓ x 0,33 x 100 Mℓ/d
= 1 320 kg/d

As a comparison the total "additional" sludge (suspended solids) produced due to alum dosing according to Geldenhuys (1992):

= 40 kg/Mℓ x 0,922 x 100 Mℓ/d
= 3 688 kg/d

The sludge due to the dosing of powdered activated carbon:

= 5 kg/Mℓ x 100 Mℓ/d
= 500 kg/d

Therefore the total daily sludge produced on a dry solids basis is estimated to be 5000 + 1320 + 500 = 6820 kg/d = 6,82 t/d (according to Montgomery (1985)) and 6750 + 3688 + 500 = 10938 kg/d = 10,938 t/d (according to Geldenhuys (1995)).

This example demonstrates that caution should be exercised when estimating sludge quantities on the basis of "factors".

15.2 FILTER WASHWATER

As mentioned in Section 15.1, in a conventional water purification works with clarifiers and filters, the bulk of the suspended solids is removed in the clarifiers and a relatively small fraction by the filters. The filter washwater is also dilute (0,1 g/ℓ to 1 g/ℓ of dry solids concentration depending on type of filter).

The current practice in South Africa for the handling of filter washwater, is either:

- to pump the washwater directly to the head of the water treatment plant when a filter is backwashed, or,
- to capture the filter washwater in a holding tank of sufficient capacity. The suspended solids are allowed to settle in the washwater holding tank, which serves as a batch settling or thickening tank. When the level of washwater is high, the supernatant is pumped to the head of the water treatment plant. The settled solids at the bottom of the holding tank are pumped to the sludge thickening, dewatering or disposal facility on a regular basis, together with the sludge from the clarifiers.

The disadvantages of the first option are:

- sporadic inflows of large volumes of washwater with high suspended solids concentrations at the head of the water treatment plant, affect coagulation and flocculation processes;
- the potential recycling of *Cryptosporidium* and *Giardia* cysts captured in the sand filters.

For the second option, a fraction of the *Cryptosporidium* and *Giardia* cysts in the filter washwater is released with the thickened sludge from the washwater holding tank to the sludge dewatering/disposal facility.

15.3 SLUDGE AND WASHWATER STORAGE

It is normal practice to store the sludge, which is released from the clarifiers and the washwater holding tank, in a sludge holding tank. The purpose of the sludge-holding tank is to smooth out the peaks and to ensure a constant and consistent feed to the thickening/sludge dewatering plant as well as to facilitate sludge conditioning prior to the thickening/dewatering plant. Sludge holding tanks are normally agitated with mechanical or diffused air agitators to "homogenise" the feed sludge to the thickening/dewatering plant. It is essential to minimise sludge volume and necessary to condition it before dewatering.

15.3.1 VOLUME MINIMISATION

It is beneficial to minimise the volumes of clarifier/washwater sludge, which are released to the dewatering/disposal facility. There are two ways of minimising the volume of sludge:

- optimise coagulation/flocculation;
- optimise desludging from the clarifiers or washwater recovery tank.

As shown in Section 15.1, the addition of coagulants/flocculants to raw water, contributes to the daily sludge dry solids production rate. The quantity of sludge dry solids produced on a daily basis is influenced by both the **type** and **dosing rate** of coagulants and flocculants, e.g. the use of polyelectrolyte instead of alum as coagulant will significantly reduce the daily sludge dry solids production rate. The dosing of coagulants/flocculants should be optimised by doing regular jar tests, for example.

As shown in Chapter 10, there are numerous techniques for releasing sludge from a clarifier (desludging). The volumes of sludge (and therefore solids concentration of sludge) are greatly influenced by the **type** of clarifier and the desludging strategy employed for a specific clarifier. Desludging should be optimised and monitored regularly.

15.3.2 SLUDGE CONDITIONING

It is normally beneficial to condition the sludge prior to thickening/dewatering to facilitate solid/liquid separation and to enhance the dewaterability of the sludge.

Sludge conditioning entails the dosing of flocculant(s) to the sludge under controlled conditions. Flocculants which may be added to the sludge include polyelectrolytes, lime, aluminium sulphate and ferric chloride. Tests to select a suitable flocculant and to optimise flocculant dosing are performed in the laboratory. Selection may be done on the basis of a single parameter, or combinations of the following parameters:

- Settling rate of flocculated material.
- Clarity of supernatant.
- Compaction of the flocculated sludge.
- Dewaterability of sludge - as tested by the resistance to filtration (see CST test, Section 15.5.2.7).

15.4 SLUDGE THICKENING

It is economically advantageous to reduce the volume of the clarifier/washwater sludge before it is further dewatered to a cake or sent to a disposal facility. The preferred process to achieve sludge volume reduction is thickening, since it is a relatively inexpensive process, which relies on gravity as the driving force for solids/liquid separation.

Four sludge thickening techniques are described in this section:

- conventional batch and continuous gravity thickening;
- dissolved air flotation;
- linear screen;
- Tasster sludge compactor.

15.4.1 BATCH AND CONTINUOUS GRAVITY THICKENERS

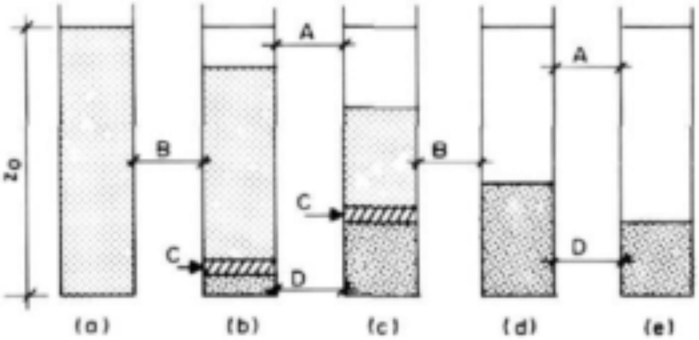


Figure 15.2
Batch sedimentation

If a flocculated suspension of uniform solids concentration is allowed to settle in a settling cylinder, the suspension will settle as is shown is Figure 15.2 (a) to (e).

Initially the total depth of the suspension is Z_0. After a period of time a lower zone D of settled particles, which rest lightly on one another, will form (see Figure 15.2 (b)). Above zone D another layer, zone C, forms. Zone C is a transition layer and the solids concentration varies from that in the original suspension to that in zone D. Above zone C is zone B, which has a uniform solids concentration equal to that of the original suspension. The particles in zone B adhere together and zone settling occurs. Zone A is a clear liquid layer.

As settling continues, the depths of zones D and A increase, that of zone C remains constant (McCabe *et al.*, 1985) and that of zone B decreases (see Figure 15.2 (c)). After further settling, zones B and C disappear and all the solids are in zone D (see Figure 15.2 (d)). Then a new effect called compression, begins. The point at which compression is first evident is called the critical point. During compression, a portion of the liquid which has accompanied the flocs to the compression zone D is expelled when the mass of the deposit breaks down the structure of the flocs.

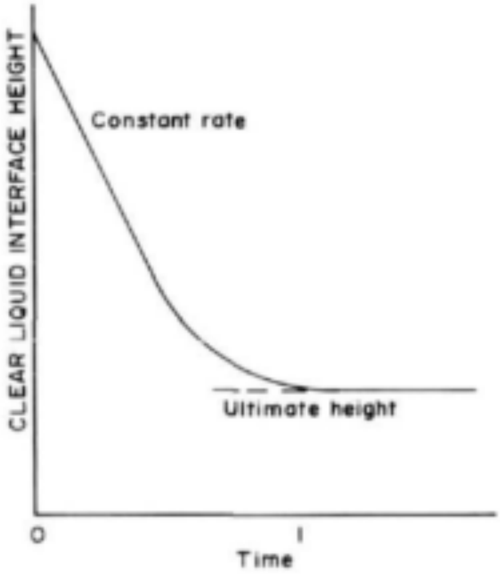


Figure 15.3
Batch sedimentation curve

Finally, when the mass of the solids is in equilibrium with the compressive strength of the flocs, the settling process stops, as is shown in Figure 15.2 (e). At this point, the sludge has reached its ultimate height.

A typical plot of the height of the sludge/clear liquid interface (the boundary between zones A and B) versus time is shown in Figure 15.3. During the early stages of settling the settling velocity is constant. As solids accumulate in zone D, the rate of settling decreases and steadily drops until the ultimate height is reached.

Batch settling tests as described above are used for the design of both batch and continuous thickeners.

15.4.1.1 Batch gravity thickeners

Batch gravity thickeners are relatively easy to design, since a settling curve as in Figure 15.3 is used directly to determine settling

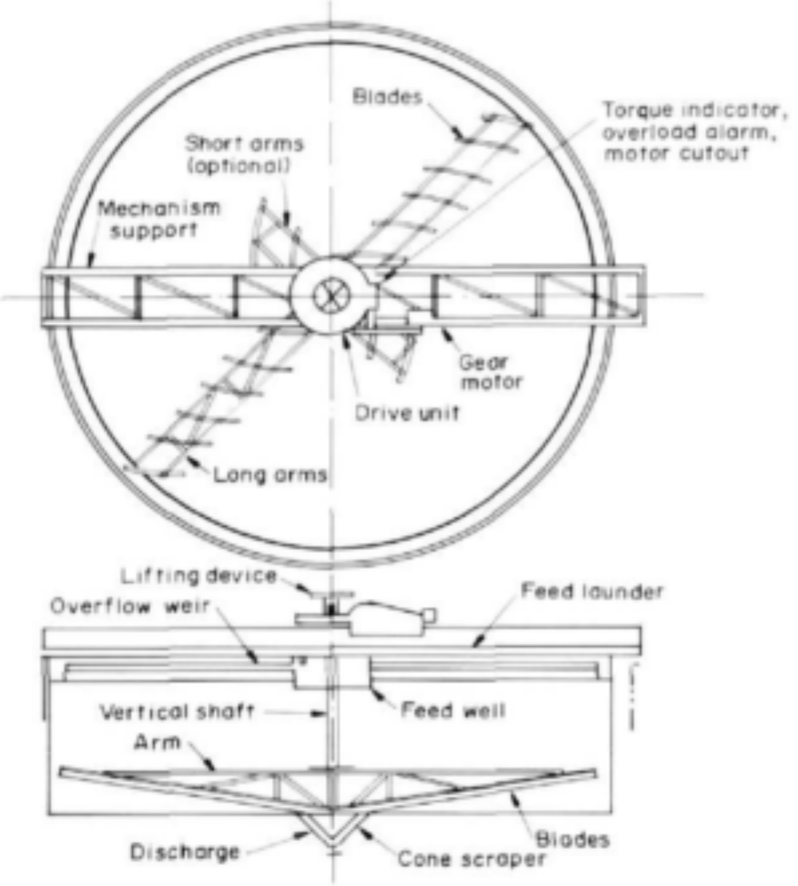


Figure 15.4(a)
Continuous gravity thickener (Eimco Corp.)

rates and interface heights.

15.4.1.2 Continuous gravity thickeners

A typical continuous thickener is shown in Figure 15.4 (a) Most continuous thickeners are relatively shallow tanks (2 to 3 m deep) with slowly revolving (1 to 3 r/min) radial rakes driven from a central shaft. The bottom may be flat or a shallow cone. Dilute feed slurry flows from an inclined trough or launder into the centre of the thickener. Sludge flows radially at a constantly decreasing velocity, allowing solids to settle to the bottom of the tank. Clear water spills over the edge of the tank into a launder. The arms gently agitate the sludge to release interstitial water from the sludge.

Thickened sludge is released continuously from the bottom of the thickener.

The concentration of solids within a continuously operated thickener is as is shown in Figure 15.4(b). The clear zone on top consists of liquid that eventually flows over the top weir into a launder. The next zone is called the feed zone; although it is not necessarily at the same concentration as the feed solids it is characterised by a uniform solids concentration. Below this zone the thickening zone is characterised by an increasing solids concentration to the point of sludge discharge.

15.4.1.3 Design of continuous gravity thickeners

The design of continuous gravity thickeners is based on batch settling tests and Kynch's settling theory.

If uniformly mixed sludge of sufficiently high solids concentration is left to settle in a transparent measuring cylinder, the sludge/clear liquid interface will settle.

The height of the sludge/clear liquid interface can be measured with time (as is shown in Figure 15.4 (c)) and the settling velocity of the sludge/clear liquid interface can be determined as shown in Figure 15.4 (c).

For the design of a thickener (see example in Section 15.4.1.3.2) the settling velocity as a function of solids concentration has to be determined. The solids concentration that is used in conjunction with the settling velocity of the sludge/clear liquid interface is found from the following equation (Foust *et al.*, 1960):

$$C = \frac{C_0 Z_0}{Z_i} \quad .... \; 15.2$$

where

- C = solids concentration, (kg/m^3)
- C_o = initial solids concentration in measuring cylinder, (kg/m^3)
- Z_i = Y axis intercept of tangent to interface height versus time curve as shown in Figure 15.4 (c), (m)
- Z_o = initial height of sludge in measuring cylinder, (m)

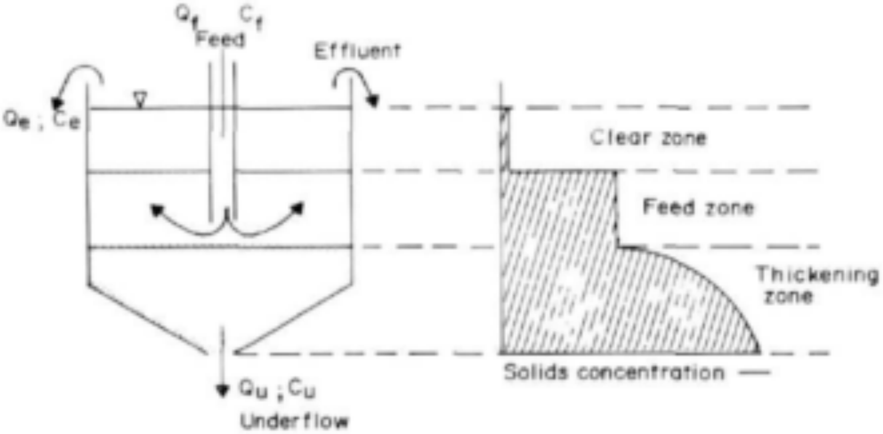


Figure 15.4(b)

Solids concentration profile in continuous gravity thickener

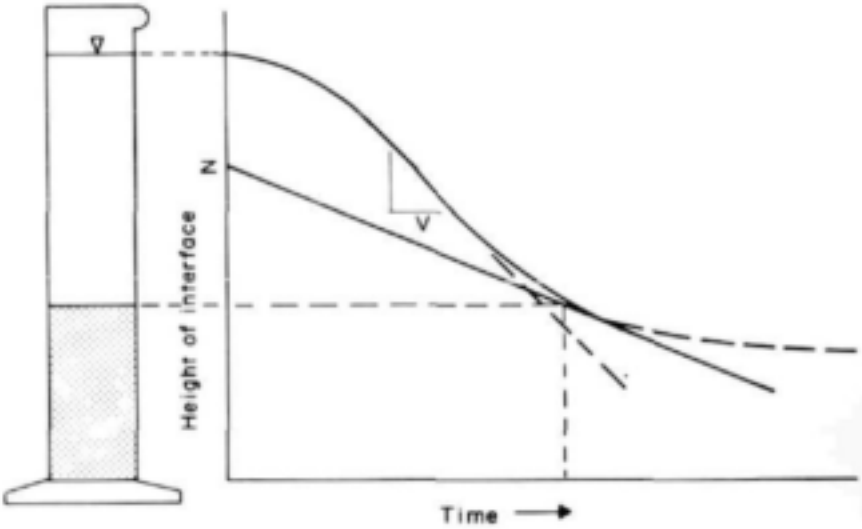


Figure 15.4(c)

Thickening test in a cylinder with resulting interface height vs. time curve

15.4.1.3.1 Thickener area

A method to calculate the area of a thickener is derived below. The symbols used in the derivation are defined as follows:

- A = cross-sectional area of thickener, (m^2)
- A_{min} = minimum cross-sectional area of thickener, (m^2)
- C = solids concentration (at a certain point in thickening zone), (kg/m^3)
- C_e = solids concentration in effluent, (kg/m^3)
- C_f = solids concentration in feed, (kg/m^3)
- C_L = limiting solids concentration in thickening zone, (kg/m^3)
- C_u = solids concentration in underflow, (kg/m^3)
- G = solids flux, (kg/m^2.s)
- Q_e = volumetric effluent flow rate,

(m^3/s)

Q_f = volumetric feed flow rate, (m^3/s)

Q_L = volumetric downflow rate of sludge at concentration C_L in thickening zone, (m^3/s)

Q_u = volumetric underflow flow rate, (m^3/s)

t = time, (s)

V = upflow velocity of effluent, (m/s)

v = settling velocity of concentration layer with solids concentration C in thickening zone, (m/s)

v_L = settling velocity of concentration layer with solids concentration C_L in thickening zone, (m/s)

ρ_{av} = average density of sludge in thickening zone, (kg/m^3)

ρ_f = density of feed sludge, (kg/m^3)

ρ_u = density of underflow sludge, (kg/m^3)

ρ_w = density of thickener overflow effluent (water), (kg/m^3)

From a solids balance over the thickener [see Figure 15.4 (b)].

$$Q_fC_f = Q_uC_u \qquad 15.3$$

and from a liquid balance:

$$Q_f(\rho_f - C_f) = Q_e\rho_w + Q_u(\rho_u - C_u) \qquad 15.4$$

(for an efficient thickener $C_e = 0$)

Equation 15.4 may also be written as:

$$Q_f(\rho_f - C_f) = VA_w + Q_u(\rho_u - C_u) \qquad 15.5$$

From Equation 15.3:

$$Q_u = \frac{Q_0C_0}{C_u} \qquad 15.6$$

Hence:

$$Q_f(\rho_f - C_f) = VA + \frac{Q_fC_f}{C_u}(\rho_u - C_u) \qquad 15.7$$

From Equation 15.7:

$$Q_f\rho_f - \frac{Q_fC_f\rho_u}{C_u} = VA\rho_w \qquad 15.8$$

$$\text{i.e. } Q_fC_f\left(\frac{\rho_f}{C_f} - \frac{C_0\rho_u}{C_u}\right) = VA\rho_w \qquad 15.9$$

From Equation 15.9:

$$V = \frac{Q_fC_f}{A}\left(\frac{\rho_f}{C_f} - \frac{\rho_u}{C_u}\right)\frac{1}{\rho_w} \qquad 15.10$$

Using ρ_{av} as the average sludge density for the thickened sludge in the thickening zone, Equation 15.10 becomes:

$$V = \frac{Q_fC_f}{A}\left(\frac{1}{C_f} - \frac{1}{C_u}\right)\frac{\rho_{av}}{\rho_w} \qquad 15.11$$

However:

$$G = \frac{Q_fC_f}{A} \qquad 15.12$$

Therefore from Equation 15.11:

$$V = G\left(\frac{1}{C_f} - \frac{1}{C_u}\right)\frac{\rho_{av}}{\rho_w} \qquad 15.13$$

In order to prevent solids from overflowing, the upward velocity of the liquid effluent must be equal to or less than the settling velocity of the solids. In Equation 15.13 V may therefore be replaced by v.

$$G = \frac{v\rho_w}{\left(\frac{1}{C_f} - \frac{1}{C_u}\right)\rho_{av}} \qquad 15.14$$

As shown in Figure 15.4 (b), inside a thickener there is an increase in solids concentration from the feed zone to the bottom of the thickening zone. At steady state the solids flux in the feed and thickening zones is constant. However, at a certain limiting solids concentration, C_L, there is a minimum thickener area required to accommodate the solids flux (Foust *et al.*, 1960).

From Equations 15.12 and 15.14 the minimum thickener area required is given by:

$$A_{min} = \frac{Q_L C_L \left(\frac{1}{C_L} - \frac{1}{C_u} \right) \rho_{av}}{V_L \rho_w} \qquad \text{..............} \; 15.15$$

The method to determine the minimum thickener area is shown in Section 15.4.1.3.2, Example 2. This example has been taken from Foust *et al.* (1960).

15.4.1.3.2 Example 2

A single batch settling test was done for a limestone sludge. The interface between clear liquid and suspended solids was observed as a function of time and the results are tabulated below. The test was done using a limestone concentration of 236 g/ℓ.

(a) Prepare a curve showing the relationship between settling rate and solids concentration.

(b) A limestone/water sludge is fed to a thickener at a rate of 50 tonne/h to produce a thickened sludge with a limestone concentration of 550 g/ℓ. Specify the thickener area required (assume $\rho_{av} = \rho_w$).

Solution

(a) Using the test data, the height of the interface (Z) is plotted as a function of time (t) as shown in Figure 15.4 (d).

From the solids concentration of the initial slurry:

$$C_o Z_o = 236 \times 36 = 8\,500 \text{ g/cm.ℓ}$$

From Equation 15.2:

$$C = \frac{8500}{Z_i} \text{ g/ℓ}$$

The tangent to the curve at t = 2 h, is found to have an intercept of Z_i = 20 cm [see Figure 15.4 (d)]. The settling velocity at that time

TEST DATA

Time (h)	Height of interface (cm)
0	36,0
0,25	32,4
0,5	28,6
1,0	21,0
1,75	14,7
3,0	12,3
4,75	11,55
12,0	9,8
20,0	8,8

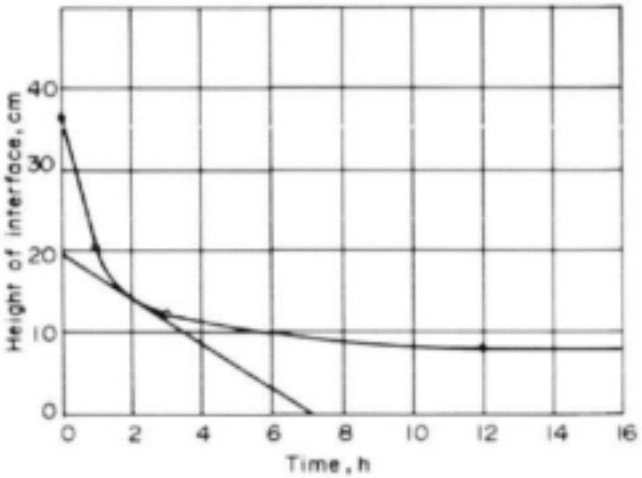


Figure 15.4(d)
Height of interface as a function of time for Example 2

TABLE 15.1
TABLE SHOWING SETTLING VELOCITY AS A FUNCTION OF SOLIDS CONCENTRATION

t (h)	Z_i (cm)	v (cm/h)	C (g/ℓ)
0,5	36	15,65	236
1,0	36	15,65	236
1,5	23,8	5,0	357
2,0	20	2,78	425
3,0	16,2	1,27	525
4,0	14,2	0,65	599
8,0	11,9	0,16	714

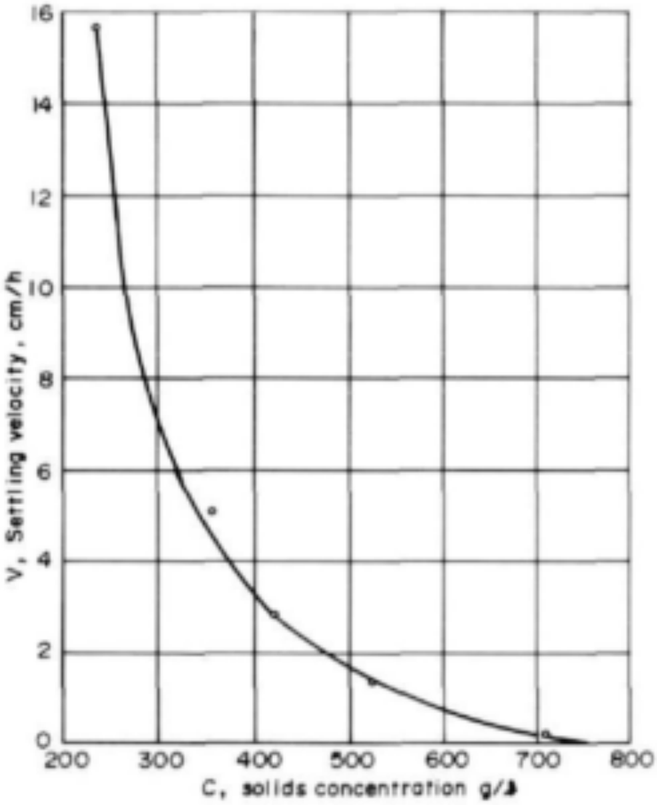


Figure 15.4(e)
Settling rate - concentration relationship for Example 2

is the slope of the curve = dZ/dt = v = 2,78 cm/h and C = 425 g/ℓ.

Other points, which are tabulated in Table 15.1 and plotted in Figure 15.4 (e) are obtained in the same way.

(b) Using Equation 15.15 and Figure 15.4 (e), Table 15.2 is prepared.

Since no solids leave in the overflow, a solids mass balance gives:

$$Q_fC_f = Q_LC_L = Q_uC_u = 50\,000 \text{ kg/h}$$
$$= 13{,}89 \text{ kg/s}$$

From Equation (15.15):

$$A_{min} = Q_LC_L \frac{\left(\frac{1}{C_L} - \frac{1}{C_u}\right)}{V_L}$$

$$= 13{,}89 \frac{\left(\frac{1}{C_L} - \frac{1}{C_u}\right)}{V_L}$$

However, C_L and V_L are not known. In order to determine A_{min}, we have to find the maximum value for $\frac{\left(\frac{1}{C} - \frac{1}{C_u}\right)}{V}$

As shown in Table 15.2 the maximum value of $\frac{\left(\frac{1}{C} - \frac{1}{C_u}\right)}{V}$ is approximately $2{,}11 \times 10^{-4}$ cm²/h.g

$$= 7{,}60 \text{ m}^2/\text{s.kg}$$

TABLE 15.2
TABLE SHOWING CALCULATIONS TO DETERMINE THE THICKENER AREA

v (cm/h)	C (g/ℓ)	(ℓ/g)	(ℓ/g)	(cm²/h.g)
10	265	0,00377	0,00195	0,000195
8	285	0,00351	0,00169	0,000211
6	325	0,00308	0,00126	0,000210
3	415	0,00241	0,00059	0,000197
2	465	0,00215	0,00033	0,000168
1	550	0,00182	0	0

therefore $A_{min} = 13{,}89 \times 7{,}60 \text{ m}^2$
$= 105{,}56 \text{ m}^2$
(diameter = 11,6 m)

15.4.1.3.2 Depth of thickener

The required depth of the thickening zone is given by Coulson & Richardson (1978):

$$d_t = \frac{W_t t_r}{A\rho_s} (1 + \frac{\rho_s}{\rho_w} X) \text{ .... } 15.16$$

where:

d_t = depth of thickening zone (m)
t_r = required residence time of solids (s)
W_t = mass rate of feed of solids to the thickener (kg/s)
X = average value of mass ratio of liquid to solids (-)
= density of solids (kg/m³)

The method of design is only approximate and therefore, in a large tank, about 1 m should be added to the calculated depth as a safety margin (Coulson & Richardson, 1978).

As an example, the process parameters for a thickener at Rand Water are given in Table 15.3.

15.4.2 DISSOLVED AIR FLOTATION

Dissolved air flotation (DAF) for water clarification is described fully in Chapter 11 and the principles of DAF will therefore not be repeated here.

In a survey in 1993, Haarhoff & Van Vuuren (1993) found that there was only one case in South Africa where DAF was used to thicken the sludge of a drinking-water treatment plant in South Africa, namely Umgeni Water's Wiggins water treatment plant in Durban.

The design procedure for a DAF plant has been given in Chapter 11 (see also Haarhoff & Van Vuuren, 1993) and only the main parameters for the DAF at Wiggins water treatment plant are shown in Table 15.4.

15.4.3 LINEAR SCREEN

The operation of the linear screen or moving belt gravity thickener is similar to that of the free drainage area of a belt press (see Section 15.5.2.3), the difference being that the linear screen has only one belt and does not have pressing rollers as the belt press.

In the same manner as the belt press, the linear screen also relies on "superflocculation" with a polyelectrolyte.

TABLE 15.3
EXAMPLE OF PROCESS PARAMETERS FOR A THICKENER AT RAND WATER (GELDENHUYS, 1995)

Shape	Circular (round)
Depth at circumference	3,0 m
Depth at centre	4,9 m
Diameter	25 m
Nominal inlet flow	1 458 m^3/h
Nominal hydraulic duty (rise rate)	2,97 m/h
Maximum inlet flow	1 895 m^3/h
Maximum hydraulic duty (rise rate)	3,86 m/h
Nominal solids capacity	23 t/h
Nominal solids duty	46 $kg/m^2.h$
Maximum solids duty	55 $kg/m^2.h$
Raking system	Variable speed: 1,0 - 0,5 r/min
Incoming sludge concentration	0,5 - 2,0% (m/m)
Underflow concentration	16 - 20% (m/m)
Sludge conditioning dry solids	Anionic polyelectrolyte 0,5 - 1,0 kg/t

Rand Water has installed two linear screens for thickening of waste clarifier sludge Geldenhuys (1992). The main parameters are shown in Table 15.5.

For the operation of the moving belt gravity thickeners, the following should receive special attention:

- adequate flocculation;
- appropriate forward belt speed;
- belt cleansing.

During normal operation the filtrate from the first section on the belt could contain high concentrations of suspended matter which should be returned to the flocculator. Filtrate will also contain excess flocculant and by recycling will reduce the initial flocculant consumption.

15.4.4 TASSTER SLUDGE COMPACTOR

The Tasster sludge compactor is a deep cone thickener with mechanically driven rakes. It is used to further thicken sludge which has already been thickened by a conventional thickener. This is achieved by further conditioning of the feed sludge by dosing additional flocculant and a well-designed mechanically driven raking system inside a deep thickening cone, for maximum release of interstitial water.

A full-scale plant has been installed at the Western Transvaal Regional Water Company. In this plant, sludge containing some 80% of organic matter by mass is thickened to a concentration of 10 to 12% solids content. Rand Water has conducted many tests on pilot scale, with very good results (Geldenhuys, 1994). Sludge rich in calcium carbonate and containing mostly natural clay material is thickened to a solids concentration of up to 35% by mass.

15.5 SLUDGE DEWATERING

15.5.1 CENTRIFUGATION

In South Africa **horizontal solid-bowl** centrifuges are extensively used for water treatment plant sludge dewatering. Three of Umgeni Water's water treatment plants, namely, Wiggins, Durban Heights and Amanzimtoti water treatment plants have been equipped with centrifuges to dewater clarifier sludge. Centrifuges have *inter alia* also been installed by George and Cape Town municipalities.

The advantages of these centrifuges are:

- compact design;
- high throughput;
- relative simplicity of operation.

The main disadvantages are:

- high energy requirements compared to filtration;
- relatively high flocculant requirements for flocculation prior to the centrifuges.

The most commonly used centrifuges for dewatering of clarifier sludge in South Africa are the continuous counter-flow decanter centrifuges as shown in Figure 15.5.

As shown in Figure 15.5, the centrifuge consists of an outside (truncated cone-shaped) rotating bowl and an inner rotating scroll. The internal scroll fits closely inside the outer bowl and conveys solids to the cake discharge end.

The outside bowl typically rotates at 1 500 to 2 500 r/min. There is a rotating speed differential between the outer bowl and the inner scroll. Typical differential speeds are in the range of 50 to 150 r/min.

TABLE 15.4
MAIN PARAMETERS FOR DAF AT WIGGINS WATER TREATMENT PLANT

Shape	circular
Depth	3,5 m
Outer diameter	10 m
Flotation area	77,8 m^2
Contact zone	9,5 m^3
Design feed rate	160 m^3/h
Design recycle rate	100 m^3/h
Saturation pressure	500 kPa
Scraper speed	2 revolutions per hour
Feed solids concentration	0,5% to 0,75% (m/m)
Solids concentration of float	5-8% (m/m)
Saturator	unpacked, horizontal, efficiency: 50-60%
Sludge conditioning	polyelectrolyte; dose 0,5 to 1 kg/ton dry solids

TABLE 15.5
MAIN PARAMETERS FOR LINEAR SCREENS AT RAND WATER

Belt dimensions	5 m wide x 6 m long
Belt speed	0,5 to 1,0 m/min
Liquid duty	71,4 m^3/m belt width.h
Dry solids loading	2500 kg/m belt width.h
Feed sludge solids concentration	3,5% (m/m)
Thickened sludge solids concentration	20% to 23% (m/m)
Filtrate turbidity	50 NTU
Flocculant consumption	0,8 to 1,2 kg/t dry solids

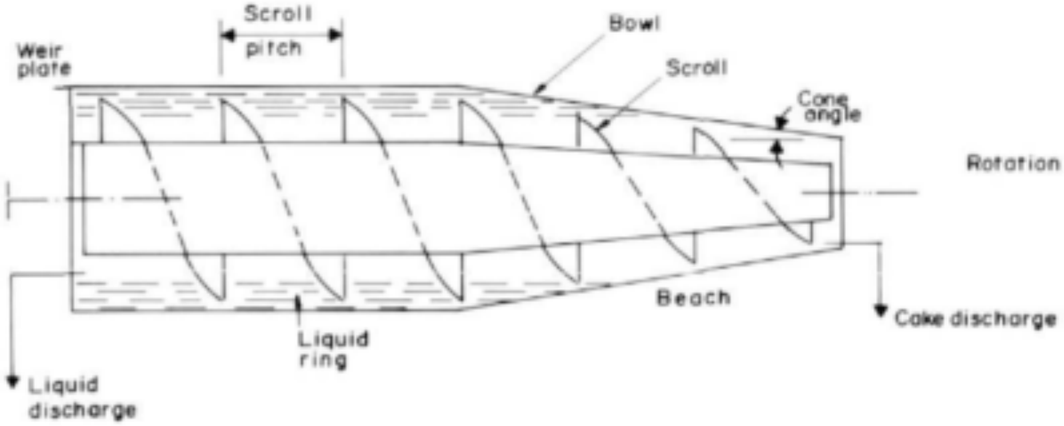


Figure 15.5
Horizontal bowl decanter centrifuge

In operation, feed is admitted through a central pipe in the scroll and is discharged in the centre of the bowl. The centrifugal action forces both liquid and solid phases to the walls of the cone and down the cone to the large (or centrate discharge) end. The solids, being more dense, concentrate against the cone walls and along the bottom of the pool of liquid held in the bowl by the weir plates. Because of the speed differential between the scroll and bowl, the scroll has a net rotation towards the small end of the bowl and scrapes the solids from the walls of the cone towards the small or cake discharge end. As the solids are scraped out of the pool along the conical section (the so-called beach) free water drains from the solids into the pool. The depth of the pool (and therefore the volume of liquid in the centrifuge) can be varied by adjusting the position of the weir plates.

A washwater system is normally included to provide a means of flushing the machine when it is to be shut down. This prevents the accumulation of solids that may cause vibrations because of imbalance.

15.5.1.1 Centrifugal theory

A very good description of centrifuge theory is given by Foust *et al.* (1960). The theory will not be repeated here, since the design engineer generally does not face the problem of designing a centrifuge, but rather has to size and specify a centrifuge. For this he has to refer to manufacturers' specifications and also has to understand the effect of various design and operational variables.

15.5.1.2 Design variables

There are several design variables to be considered when choosing a centrifuge for a specific dewatering application, the most important being bowl design, rotational speed and scroll speed.

15.5.1.2.1 Bowl design

As the bowl length increases, retention time increases, resulting in drier cake and clearer centrates. However, power requirements also increase, since they are proportional to the volume contained in the centrifuge. As is true for bowl length, increasing the diameter of a centrifuge results in drier cakes and clearer filtrate at the same feed rate.

15.5.1.2.2 Bowl speed

Higher bowl speeds produce drier cakes and clearer centrates, but also increase the machine wear. When centrifuging abrasive materials, low speed centrifuges are preferred.

As the bowl/scroll speed differential increases, the throughput increases; however, the penalty is a wetter cake and deteriorating centrate quality.

15.5.1.2.3 Cone angle

In the conical section of the bowl, the sediment is subjected to back-pressure, which becomes weaker as the cone angle decreases. This back-pressure must not destroy the cohesion of the thickened sludge, otherwise extraction will be poor and a substantial amount of cake will be re-slurried.

15.5.1.3 Operational variables

For sludge dewatering, the following variables must be balanced to obtain the desired results.

15.5.1.3.1 Sludge flow

The sludge feed rate is one of the more important variables. The rate of sludge feed affects both clarity and cake dryness. As sludge flow rate increases, centrate quality and cake dryness deteriorate. Extremely high flows cause machine flooding. Very low flow rates produce extremely dry cakes causing increased wear of the scroll. Sludge should be pumped to the machine at the highest rate that will not cause excessive carry-over of fines in the centrate.

15.5.1.3.2 Chemical conditioning

Polyelectrolytes are used to condition the sludge prior to entry into the scroll. Polyelectrolyte addition has a significant effect on cake dryness and centrate quality. The variables that need to be considered with polyelectrolyte addition are:

- polyelectrolyte dose rate;
- injection point (normally just before sludge entry into the centrifuge).

Polyelectrolyte is normally diluted with water to allow good polyelectrolyte distribution in the sludge.

15.5.1.3.3 Pool depth

Pool depth determines pool volume and affects both clarification and cake dryness. Decreasing the pool depth exposes more beach area (for drainage of cake) and consequently increases the cake dewatering time on the beach, thereby leading to a drier cake. Conversely, increasing pool depth increases clarification (i.e., improves centrate quality) by increasing retention time. If pool depths are too great, however, particles do not reach the sedimentation zone and are discharged with the centrate. On the other hand, if pool depths are too shallow, the linear liquid velocities in the liquid ring towards the centrate discharge side will be too high and this will lead to re-suspension of fines. Consequently "medium" pool depths are used in practice (Nalco Chemical Company, 1979).

15.5.2 FILTRATION PROCESSES

While filters are not generally used in South Africa for the dewatering of sludges from drinking water works, a description is given of several types of filters, which may be used for sludge dewatering purposes. The following filters are described:

- vacuum filter;
- belt press;
- plate and frame filter press;
- tubular filter press;
- coagulant recovery sludge dewatering system (CRSDS) and rapid sludge dewatering system (RSDS).

In order to facilitate understanding of the various filters, it is necessary to give a brief summary of filtration theory.

15.5.2.1 Filtration theory

The conventional planar filtration equation for both a compressible and an incompressible filter cake is:

$$\frac{dv}{dt} = \frac{P}{\mu_f(\alpha_{av}\ W_c + R_m)} \quad \text{.........................} \ 15.17$$

where:

P = applied filtration pressure, (Pa)
R_m = resistance of medium, (m^{-1})
t = time, (s)
v = volume of filtrate per unit medium area, (m^3/m^2)
W_c = total mass of cake dry solids deposited per unit medium area, (kg/m^2)
= average specific filtration resistance of cake, (m/kg)
$_f$ = viscosity of filtrate, (Pa/s)

Equation (15.17) can be written as:

$$\frac{dt}{dv} = \frac{\mu_f(\alpha_{av}\ W_c + R_m)}{P} \quad \text{.........................} \ 15.18$$

However:

$$W_c = cv \quad \text{...} \ 15.19$$

where:

c = mass of cake dry solids per unit volume of filtrate (kg/m^3)

Substituting Equation 15.19 in Equation 15.18 gives:

$$\frac{dt}{dv} = \frac{\mu_f(\alpha_{av}\ cv + R_m)}{P} \quad \text{.........................} \ 15.20$$

For constant pressure filtration and assuming c is constant (Leu, 1981), Equation 15.20 may be integrated to give:

$$t = \frac{\mu_f \alpha_{av}\ cv^2}{2P} + \frac{\mu_f R_m v}{Pc} \quad \text{................} \ 15.21$$

From Equation 15.21, the time to produce a required volume of filtrate may be calculated, for **constant pressure** filtration.

The above equations may be further manipulated to calculate the time to deposit any cake mass, W_c, for a given value of c:

$$t_d = \frac{\mu_f \alpha_{av}\ W_c^2}{2Pc} + \frac{\mu_f R_m W_c}{Pc} \quad \text{................} \ 15.22$$

where:

t_d = time to deposit cake mass W_c (s)

Another mode of filtration is constant rate filtration. For **constant rate** filtration i.e. ${}^{dv}/_{dt}$ is constant, Equations 15.17 and 15.20 may be used to calculate a relationship between P and W_c or P and v, respectively.

15.5.2.2 Vacuum filtration

Vacuum filters have been used extensively in the USA and Europe to dewater sludges from water works. In South Africa the application of vacuum filtration systems for water works sludge dewatering has been limited to the Coagulant Recovery Sludge Dewatering System (CRSDS) and the Rapid Sludge Dewatering System (RSDS) (see Section 15.5.2.6). However, for completeness rotary vacuum filters will be described in this section.

As shown in Figure 15.6, a rotary drum vacuum filter consists of a perforated cylindrical drum rotating in a tank or bath containing the sludge to be dewatered. A subatmospheric pressure is applied to the interior and solids/liquid separation is achieved by "sucking" the sludge through a filter medium. The cake is deposited on the medium, while the filtrate passes into a collection cylinder. The vacuum filtration process

TABLE 15.6
OPERATING VARIABLES FOR A ROTARY VACUUM FILTER

Operating variable	Effect
Sludge conditioning by dosing chemicals such as polyelectrolytes, ferric chloride or lime	Formation of larger particles - higher filtration rates
Agitation in tank	Eliminate settling in tank
Cycle time of drum	Higher speed results in thinner, drier cake
Filter media	Coarser material gives drier cakes, but inferior filtrate quality
Particle size of solids in sludge	High solids capture of all sizes is important otherwise fines accumulation leads to blinding of medium
Vacuum	Higher vacuum in filtration stage will normally lead to higher filtration rates, while higher vacuum in cake drying stage results in drier cakes

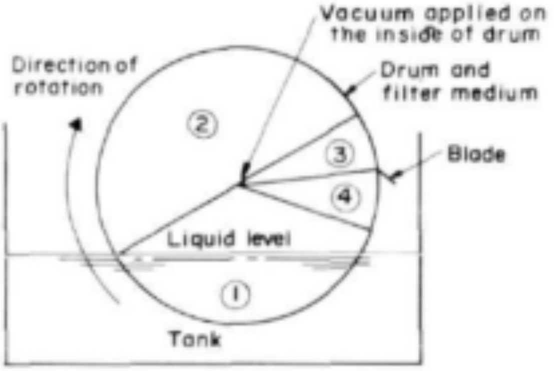


Figure 15.6
Operating sequence of rotary-drum vacuum filter

is continuous, with the filter moving through the following sequence:

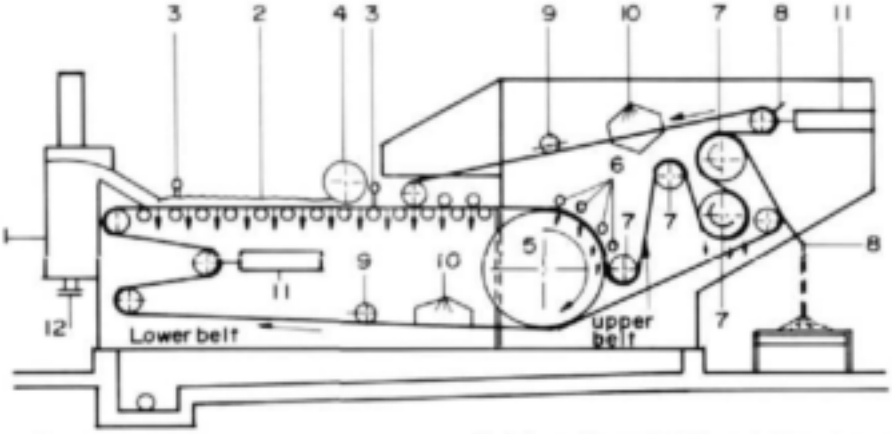
- cake filtration (stage 1 in Figure 15.6);
- liquid extraction or drying (stage 2 in Figure 15.6);
- filter cake removal (stage 3 in Figure 15.6); the filter cake is normally removed from the filter medium by a "doctor blade";
- media washing and cleaning - high-pressure water sprays are normally used to dislodge any particles that could accumulate and clog the medium.

15.5.2.2.1 *Output capacity of a rotary vacuum filter*

Rotary vacuum filters are normally operated at constant "vacuum" of 40 to 80 kPa (Degremont, 1991).

In order to calculate the amount of cake deposited on the medium of a rotary vacuum filter during the immersion period in the tank, Equation 15.22 may be solved for W_c:

$$W_c = \frac{-\mu_f R_m + \sqrt{(\mu_f R_m)^2 + 2\mu_f \alpha_{av} Pct_d}}{\mu_f \alpha_{av} T} \qquad 15.23$$



1. Flocculator/mixer
2. Drainage zone
3. Combing rakes (movable)
4. Levelling roller (movable)
5. Filter drum
6. Additional pressing rollers controlled by pneumatic jacks
7. Idler rollers of different diameters and drive rollers
8. Scrapers
9. Pneumatic cloth centring device
10. Enclosed washing system
11. Pneumatic cloth tensioning jacks

Figure 15.7
Belt press

For a rotary vacuum filter the filtration time or time to deposit a cake may be expressed as a fraction of the total time for one revolution of the filter (Degremont, 1991), i.e.

$$t_d = nT \qquad 15.24$$

where:
- n = immersion fraction of the drum in the tank (-)
- T = total time for one revolution of the filter, (s)

By combining Equations 15.23 and 15.24, the output capacity of a rotary drum filter may be calculated as:

$$W_c = \frac{-\mu_f R_m + \sqrt{(\mu_f R_m)^2 + 2\mu_f \alpha_{av} PcnT}}{\mu_f \alpha_{av} T} \qquad 15.25$$

where:
- W = mass of dry solids of cake produced per unit area and unit time (kg/m^2.h)

15.5.2.2.2 *Operating variables for a rotary vacuum filter*

There are several operating variables which have to be balanced to operate a vacuum filter efficiently. Since each sludge, which is filtered, has its own dewatering characteristics, the variables must be tuned to give the desired results.

Operating variables and their effects are shown in Table 15.6.

15.5.2.3 Belt press

The fundamental condition for the application of belt presses to sludge dewatering is the phenomenon of "superflocculation", whereby an overdose of appropriate polyelectrolytes provides large flocs in clear interstitial water. This means that unless an appropriate polyelectrolyte can be found to achieve "superflocculation" for a specific sludge, a belt press will not be suitable for the dewatering of that sludge.

As mentioned in Section 15.4.3 the same applies to the linear screen used for sludge thickening. The operation of the linear screen is similar to that of the belt press, the difference being that a belt press produces a cake instead of a thickened sludge.

There are many types of belt press available, but most of them have four phases of operation, namely:

- sludge conditioning to achieve "super-flocculation";
- free drainage on the lower belt;
- progressive pressing between the lower and upper belt;
- cake shear or deformation.

A schematic diagram of a belt press is shown in Figure 15.7.

TABLE 15.7
OPERATING VARIABLES FOR A BELT PRESS

Operating variables	Effect
Polyelectrolyte dosing	Dose rate too low - wet cake and low solids capture resulting in poor filtrate quality Dose rate too high - gelatinous cake
Linear velocity of the two belts	Lower velocity - lower output, higher cake solids concentration Higher velocity - higher output, lower cake solids concentration
Belt pressure	Higher belt pressure - higher cake solids concentration, higher cake creep

Both upper and lower belts are permeable. During the free drainage phase, free interstitial water drains through the lower belt, while the solids remain on the belt. This natural drainage significantly increases the dry solids content of the sludge, giving a sludge of sufficient consistency and cohesion to withstand progressively increasing pressure. Belt presses are normally equipped with combing rakes in the free drainage zone, to facilitate the release of interstitial water.

During the pressing phase, increasing pressure is applied to the thickened sludge as it is squeezed between the upper and lower belts. The pressure applied is limited by the risk of sludge creep, as the pressing zone is not sealed at the sides of the moving belts. The maximum pressure is limited to approximately 100 kPa. Increased pressure is achieved by tensioning of the cloth using pneumatic jacks and pressing rollers, which are also controlled by pneumatic jacks (see Figure 15.7).

After pressing, additional cake dewatering is achieved by cake deformation or shear. This is achieved by "bending" of the upper and lower belts over rollers of various sizes (see Figure 15.7). The deformation of the cake, while the belts roll over the rollers gives rise to shear stresses which release interstitial water and gives rise to new flow channels in the cake.

The cake is scraped off the belts by scrapers and the upper and lower belts are subsequently washed (see Figure 15.7).

The output capacities of belt presses are expressed in kg of dry solids per metre of belt width per hour.

The performance of a belt press is influenced by the variables shown in Table 15.7.

Apart from pilot-scale tests by Rand Water, belt presses have not been used in South Africa for the dewatering of sludges from water works.

Rand Water conducted trials on a belt press with a capacity of 25 t/d of dry solids. For sludges containing high concentrations of calcium carbonate (up to 75% by mass) cakes with 50% or more solids concentration were achieved. Sludges with lower calcium carbonate (30 to 40% by mass) were more

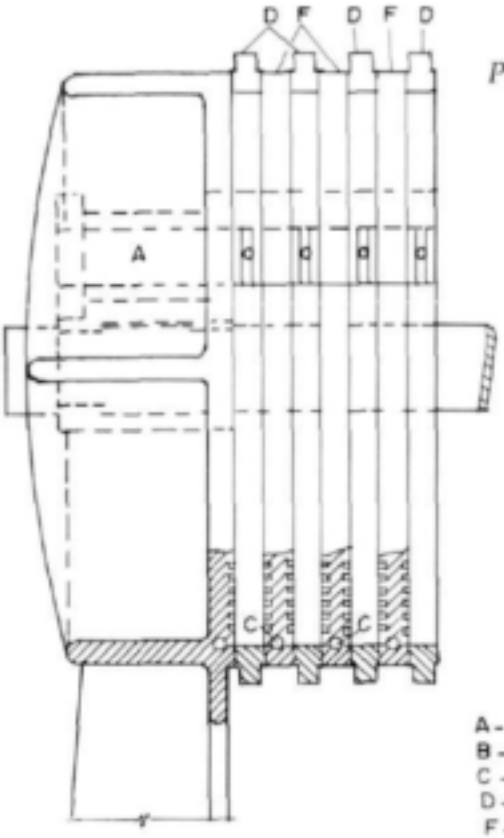


Figure 15.8
Plate and frame filter press

difficult to dewater and produced cakes with lower solids concentrations of 30 to 40%. Blinding of the belts occurred more readily with this sludge (Le-Patourel, 1993).

15.5.2.4 Plate and frame filter press

A plate and frame filter press consists of a set of vertically, recessed plates supported in a common frame. During the filtration cycle, the frames are pressed together either electromechanically or hydraulically between a fixed and a moving end. A filter cloth is mounted on the face of each plate (see Figure 15.8).

The filtration cycle consists of four phases:

- **Filling** - Once the frames have been pressed together, sludge is pumped into the spaces formed between adjacent plates. This phase is relatively brief (5 to 15 minutes) (Degremont, 1991). Filling time depends on the capacity of the feed pump.

- **Filtration** - Once the chambers are filled, continuous introduction of the sludge to be dewatered and the increasing resistance due to the increasing thickness of the filter cake, cause an increase in pressure inside the chamber.

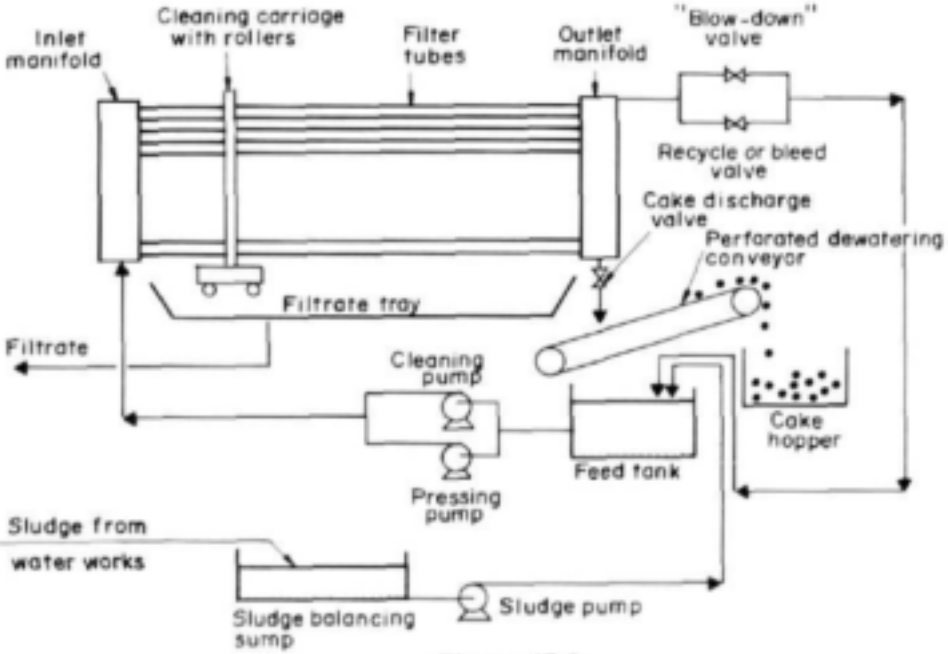


Figure 15.9
Process schematic for tubular filter press process

A filter press may be operated either at constant pressure or at constant rate.

- **Filter opening** - In this operation the filter is "opened"; the plates and frames are separated from each other and the cakes formed between pairs of plates are removed. Although the filtration cycle on most modern filter presses has been automated, manual labour is required to ensure that all cakes are removed.
- **Cleaning** - This operation is carried out occasionally only. It entails rinsing and brushing to prevent blinding of the filter cloths.

TABLE 15.8
OPERATING VARIABLES FOR A PLATE AND FRAME FILTER PRESS

Operating variable	**Effect**
Sludge conditioning	Lowers a_{av}; increases output capacity
Filtration pressure	High filtration pressure gives higher solids concentration in cake. Also, higher filtration pressure increases filter press output; although for very compressible cakes the effect is only marginal.
Filtration time	It can be shown that for constant pressure filtration the maximum output capacity of the filter press is achieved when $t_d = t_f + t_c + t_{ci}$ (Coulson & Richardson, 1978) For constant rate filtration the maximum output capacity of the filter press is achieved when t_d is the filtration time to reach the maximum filtration pressure, P

For constant pressure filtration the solids deposited in the filter cake (per unit area of filter medium), W_c, may be calculated from Equation 15.22, while for constant rate filtration, W_c (for a final filtration pressure, P) may be calculated from Equation 15.17.

The output capacity of a filter press for constant pressure and constant rate filtration is:

$$W = \frac{W_c}{t_f + t_d + t_c + t_{ci}} \quad \text{.........................} \; 15.26$$

where

t_f = time required for filling of filter press (s)

t_c = time required for opening of filter press and cake removal (s)

t_{ci} = time required for cleaning of filter cloths (s)

The performance of a plate and frame filter press is influenced by the operating variables shown in Table 15.8.

Apart from pilot-scale tests by Rand Water, plate and frame filter presses have not been used in South Africa for the dewatering of sludges from water works.

Plate and frame filter presses have, however, been used extensively in Europe for the dewatering of water works sludges (Degremont, 1991).

15.5.2.5 Tubular filter press

The tubular filter press is a novel tubular configured filter press, which was developed by the Pollution Research Group of the University of Natal under a Water Research Commission grant (Treffry-Goatley & Buckley, 1987). A schematic of the tubular filter press process is shown in Figure 15.9. The process consists of a sludge balancing or collection sump, a feed tank, a filtrate collection tray, a pressing pump, a tube flushing or cleaning pump, the filter tube array and cleaning rollers mounted on a carriage.

The tube array is made up of filter tube curtains. Each tube curtain consists of a number of filter tubes manifolded in parallel, between an inlet and a reject manifold. The filter tubes are flexible and collapsible and are constructed from woven polyester fabric (Downing & Squires, 1986).

The sludge pump supplies thickened sludge on demand from the balancing sump to the feed tank. The sludge is pumped into the tube array by the pressure pump. When the cake discharge pump is closed, the pressure in the tube array increases and the filtrate permeates through the tube walls. The sludge solids are simultaneously deposited as a cake on the inner walls of the tubes. As the thickness of the cake increases the filtrate flow rate decreases. The cake thickness is limited to a maximum in order to prevent tube blockages.

When it is estimated that the desired cake thickness has been achieved, the cake discharge valve is opened, the cleaning pump is started (to pump sludge from the feed tank through the tube curtains), the rollers are engaged and the cake is dislodged from the inner tube surface by the action of the rollers moving along the outer tube surface. The rollers create a restriction in the tubes. The high fluid velocity, reduced pressure and turbulence created at the point of restriction are sufficient to dislodge the cake from the tube surface. The cake is simultaneously hydraulically conveyed out of the tubes and is separated from the bulk fluid on a perforated conveyor belt.

Not all the deposited cake is recovered during the cake removal phase. Some cake is re-slurried or re-dispersed by the action of the flowing fluid and the moving rollers. This leads to an increase in the solids concentration of the feed sludge in the feed tank.

The entire cyclical process is micro-processor controlled and requires a minimum of operator attention.

The major advantages of the tubular filter press are that:

- No plates and frames are required as for a conventional plate and frame filter press. This makes the equipment less bulky and decreases costs relative to a plate and frame filter press.

- The cleaning cycle is short and the cleaning operation relatively simple. The filter is particularly suitable for sludges which produce cakes with a high specific filtration resistance since the rapid and simple cleaning allows the deposition and removal of relatively thin cakes. The rapid cyclic operation leads to significantly higher overall filtration rates.

- The process is modular because of the concept of tube curtains. This facilitates maintenance and allows for inexpensive and easy upgrading of the works.

The tubular filter press process has been modelled by Rencken (1992). Since the model is complex, it will not be repeated here, but the interested reader is advised to consult the referenced text.

A prototype unit of the tubular filter press was constructed at Umgeni Water's HD Hill water treatment plant in Pietermaritzburg in 1987. The prototype performed well, but tube blockage problems were experienced during periods when high doses of bentonite were added to the raw water, especially in winter.

Further development work on the tubular filter press is currently being undertaken by Umgeni Water under a Water Research Commission grant.

15.5.2.6 The coagulant recovery sludge dewatering system (CRSDS) and rapid sludge dewatering system (RSDS)

A Coagulant Recovery Sludge Dewatering System (CRSDS) was installed at Umgeni Water's Hazelmere water treatment plant in 1987. The objective of this system was not only to dewater the aluminium sulphate/polyelectrolyte sludge (derived from the clarifiers) to produce a cake, but also to recover aluminium sulphate from the sludge. The recovered alum solution was dosed into the raw water at the head of the works in order to save on commercial alum.

Both the CRSDS and the Rapid Sludge Dewatering System (RSDS) rely on a vacuum-assisted bed of porous plates for sludge dewatering. The difference between CRSDS and RSDS is that RSDS is a sludge-dewatering process without coagulant recovery.

15.5.2.6.1 The basic concept of alum recovery

When alum sludge is sufficiently acidified with sulphuric acid, aluminium is dissolved from the sludge in the form of dilute liquid alum. The stoichiometry of alum recovery is given by the following relationship:

$$2Al(OH)_3.3H_2O+3H_2SO_4+2H_2O \rightarrow Al_2(SO_4)_3.14H_2O \quad \text{.......} \quad 15.27$$

The liquid alum can be recovered and reused. According to Bishop *et al.* (1987), the sludge that remains after alum recovery is generally less voluminous, more concentrated and may be more easily dewatered. Overall cost savings can be achieved, because less commercial alum has to be purchased and the size of the sludge handling facilities can be reduced.

At this point it should be mentioned that ferric chloride may also be recovered from sludge when ferric chloride is used as a coagulant. This is achieved by using hydrochloric instead of sulphuric acid.

15.5.2.6.2 Process description of CRSDS

The process schematic diagram is given in Figure 15.10. The plant operation is a cyclic batch process, while the duration of a cycle is approximately 24 hours.

The CRSDS consists of a gravity thickener, a sulphuric acid dosing system, a reaction vessel, a polyelectrolyte dosing system, the CRSDS bed with vacuum chamber and recovered alum storage tanks with dosing facilities.

Waste sludge, which is discharged from the clarifiers, is fed into the gravity thickener, where the sludge is allowed to settle and supernatant water may be decanted and returned to the head of works if desired.

Before a run, the thickened sludge is homogenised by means of a coarse bubble aeration system in the thickener. The homogenised sludge is pumped from the thickener to the reaction vessel and concentrated sulphuric acid (98% m/m) is dosed at the inlet to the reaction vessel so that the pH in the reaction vessel is 2,0. The reaction vessel is a Continuously Stirred Tank Reactor (CSTR) with a sludge residence time of 15 minutes.

After the reaction vessel, polyelectrolyte is dosed into the acidified sludge (for enhanced dewatering) and the sludge gravitates onto the CRSDS bed through a series

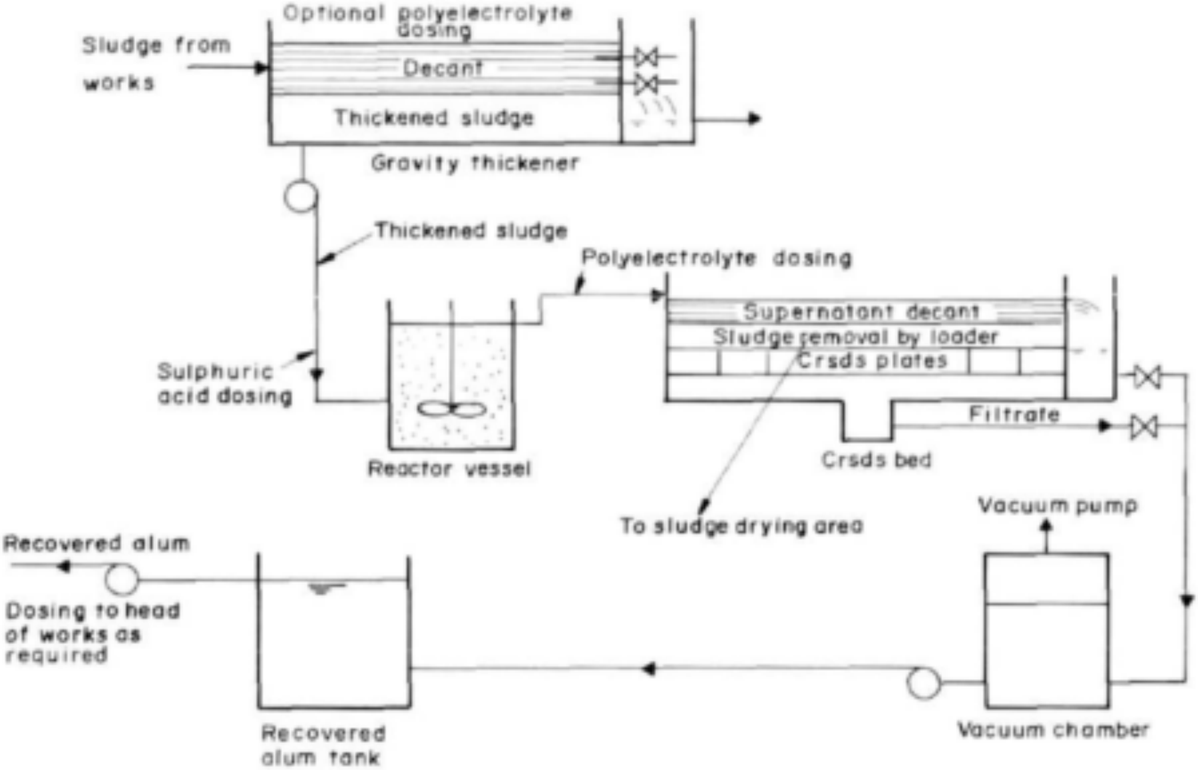


Figure 15.10
CRSDS process schematic

of nozzles.

Sludge is loaded onto the CRSDS bed to whatever level is required for the run. Supernatant alum liquor is decanted into the vacuum chamber soon after the bed is loaded and the sludge solids have settled onto the porous CRSDS filter plates. The recovered alum liquor is pumped to either of the two recovered alum storage tanks.

After decanting, the process is automatically controlled and consists of applying a vacuum to the underside of the filter plates, thus drawing filtrate through the sludge cake and the porous filter plates. The vacuum is gradually increased until the sludge cake cracks, thus completing the vacuum cycle which typically lasts 12 hours.

During the vacuum cycle any recovered alum liquor, which drains from the cake into the vacuum chamber is pumped to the selected recovered alum storage tank.

Filter cake is removed from the bed using a front-end loader and is transferred to a nearby area where further drying takes place.

After the cake has been removed from the filter plates, the plates are cleaned manually by high-pressure water nozzles to prevent any subsequent blocking of the porous plates.

Recovered alum is dosed at the head of the works, together with commercial alum and polyelectrolyte.

15.5.2.6.3 Results obtained at Hazelmere water purification works with CRSDS and RSDS

The CRSDS was operated at Hazelmere water purification works from 1987 to 1989. From the results obtained it was found that coagulant recovery was not economically viable for the application at Hazelmere. From 1989 the CRSDS was operated in the RSDS mode, i.e. as a sludge dewatering plant without alum recovery.

Detailed results obtained for the CRSDS at Hazelmere water treatment plant have been given by Rencken *et al.* (1989). Only the most important results will be quoted here.

15.5.2.6.4 Bed solids loading and cake solids concentration

The design solids loading of the bed was 14,7 kg dry solids/m². The effect of raw water turbidity on bed solids loading is shown in Figure 15.11.

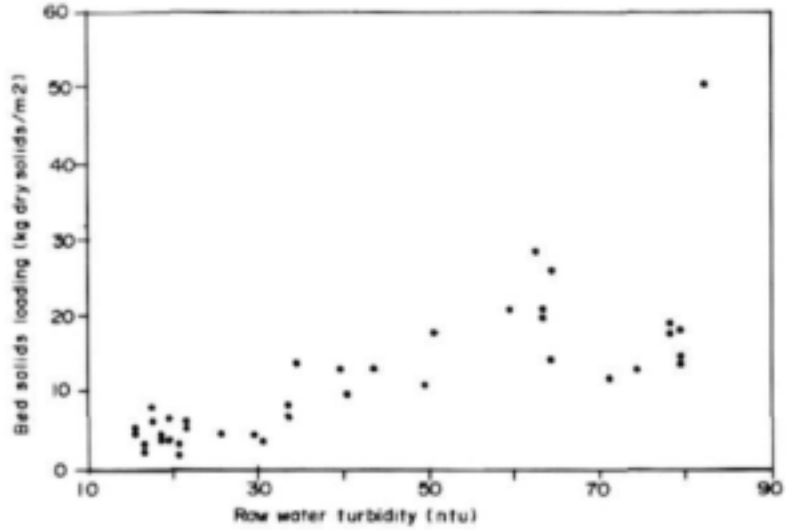


Figure 15.11
Graph showing effect of raw water turbidity on bed solids loading

One of the alleged advantages of coagulant recovery is that it leads to an increase in cake solids concentration. The effect of alum recovery on cake solids concentration is shown in (Figure 15.12). There is no visible improvement in cake solids concentration as alum recovery increases. The average cake solids concentration achieved over the test period was approximately 25%. It should also be noted that after air drying for 14 days the cake solids concentration was in excess of 50%.

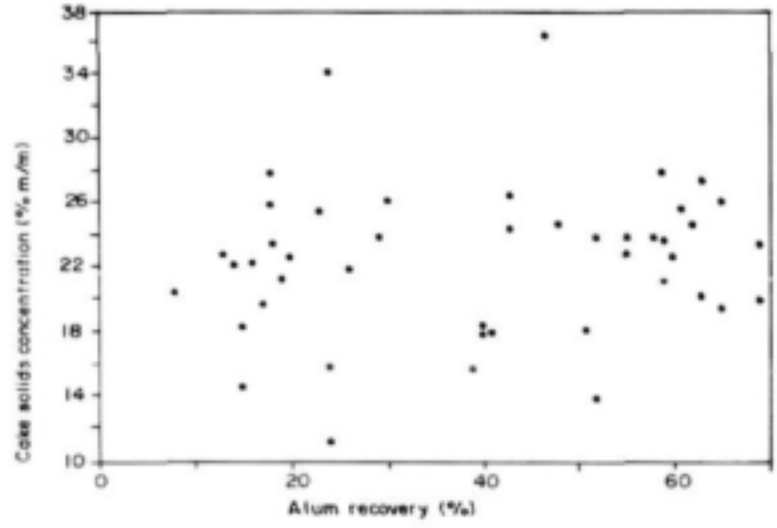


Figure 15.12
Graph showing effect of alum recovery on cake solids concentration

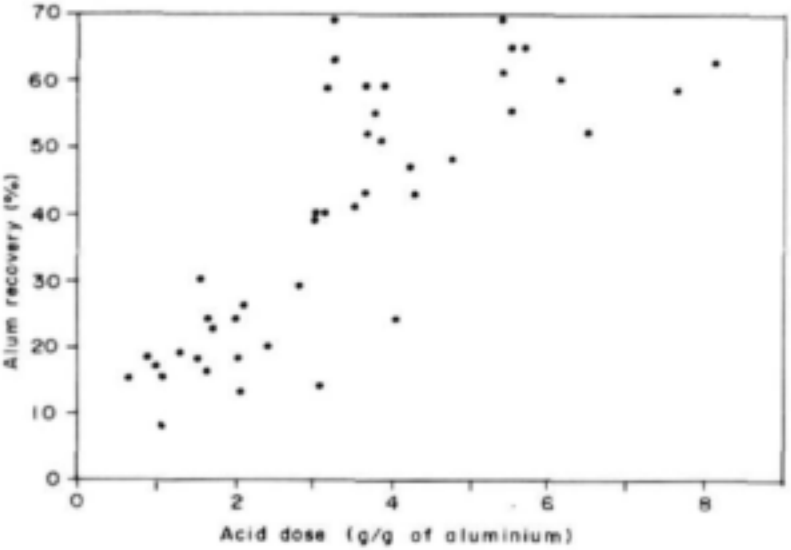


Figure 15.13
Graph showing effect of acid dose on alum recovery

15.5.2.6.5 *Alum recovery*

From Equation 15.27, the theoretical acid dose to dissolve all aluminium is 5,44 g of acid/g of aluminium.

As shown in Figure 15.13 alum recovery increases almost proportionally to acid dose i.e. for an acid dose of 2 g/g of aluminium the recovery is 20%, while for an acid dose of 6 g/g of aluminium the recovery is 60%.

Also, from Figure 15.13 it is evident that at very high acid doses (7 to 8 g/g of aluminium), the alum recovery does not increase significantly above 60%. The optimum acid dose seems to be around 6 g/g of aluminium.

15.5.2.6.6 *Results obtained by Rand Water*

Rand Water also evaluated RSDS on pilot scale (Geldenhuys, 1992). The bed had an area of 50 m^2 and a maximum solids loading of 6 t dry solids per day. The dry solids loading on the bed was 120 kg/m^2, compared to 14,7 kg/m^2 for Hazelmere water treatment plant. On the Rand Water plant cake solids concentrations of up to 40% were achieved.

15.5.2.6.7 *Advantages and disadvantages of CRSDS/RSDS*

From the pilot evaluations of Umgeni Water and Rand Water (Rencken *et al.*, 1989 and

TABLE 15.9
ADVANTAGES AND DISADVANTAGES OF CRSDS/RSDS

Advantages	Disadvantages
(i) mechanically simple; (ii) simple control for RSDS; (control for CRSDS more sophisticated).	(i) blinding of porous plates after some time - reduced output, cleaning difficult; some time - reduced output, cleaning difficult; (ii) cake removal labour intensive; (iii) uneven cake distribution on bed premature cracking and loss of vacuum, resulting in relatively low cake solids concentrations.

Geldenhuys, 1992) it was concluded that the main advantages and disadvantages of CRSDS/RSDS were as shown in Table 15.9.

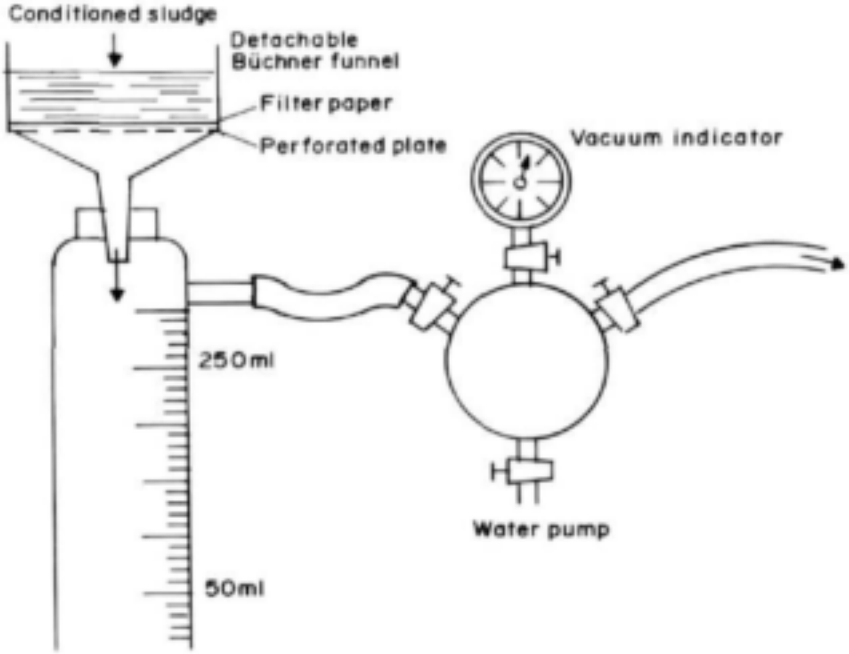


Figure 15.14
Büchner funnel

15.5.2.7 Laboratory tests to assist in design and optimisation of sludge dewatering plants

There are four basic laboratory tests to assist in the design and optimisation of sludge dewatering plants:

- vacuum filterability test with a Büchner funnel;
- pressure filterability test using a pressure filtration cell;
- compression - permeability (C-P) cell test;
- capillary suction time (CST) test.

The first three tests apply specifically to the determination of specific cake resistance, for the design and reagent dosing optimisation for a filter, while the CST test is an operational tool and is used mainly to determine optimum dosing rates for flocculants and conditioning chemicals for the operation of a sludge dewatering plant.

Since space limitations do not allow a detailed description of each method, a very broad overview of each method will be given.

The vacuum filterability test with a Büchner funnel is designed to assist with the design of a vacuum filter such as the rotary drum vacuum filter. It can also be used to determine the optimum doses of reagents for filters in general. The test is described fully in the Degremont Water Treatment Handbook (1991). As shown in Figure 15.14, a Büchner funnel is filled with the sludge to be filtered and a constant "vacuum" is applied to the filtrate collection flask. The volumes of filtrate at different times are noted.

The specific cake resistance, , and medium resistance, R_m, are determined by plotting a graph of ${}^{t}/_{v}$ vs. v using Equation 15.21.

The pressure filterability test using a pressure filtration cell is designed to determine R_m for pressure filters and it can also be used to determine the optimum reagent dosing. The test is described in the Degremont Water Treatment Handbook (1979). It is almost identical to the Büchner funnel vacuum test, except that the sludge is poured into a pressure cell and a positive constant pressure is applied to the cell (see Figure 15.15).

A C-P cell is used to accurately determine the degree of compressibility of a filter cake.

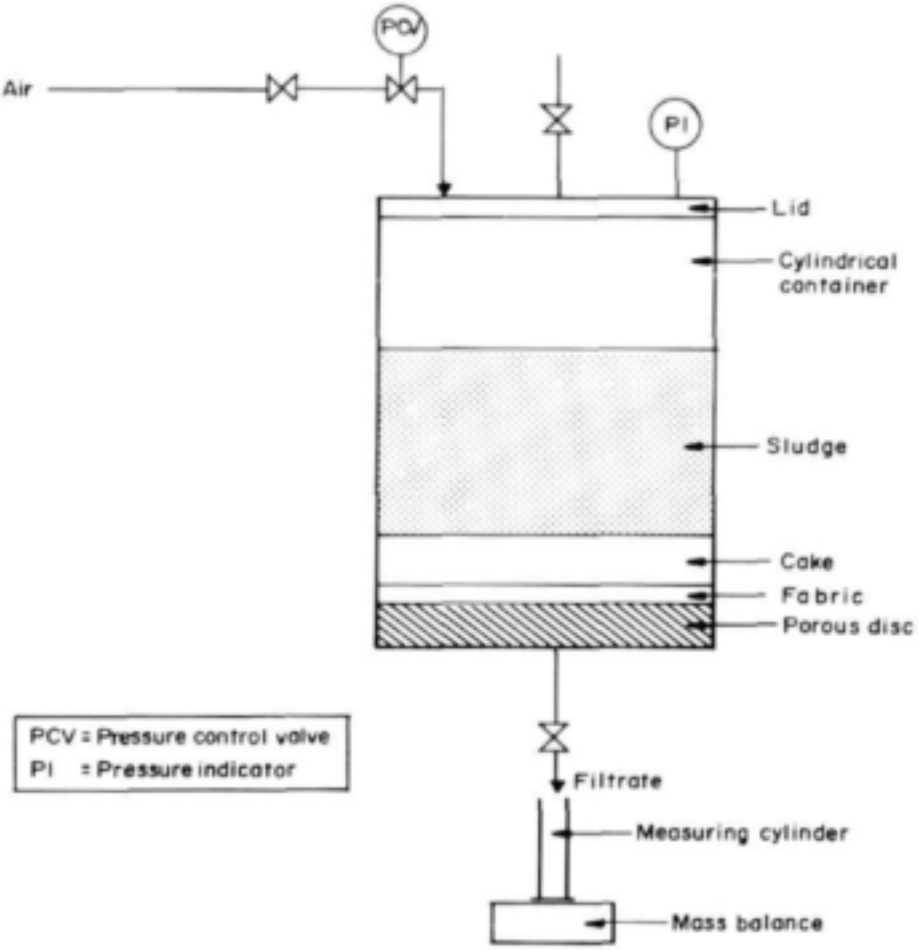


Figure 15.15
Pressure filtration cell

The C-P cell test was described by Rowe *et al.* (1966). It is a sophisticated test and can be used for accurate modelling of pressure filtration operations. Rencken (1992) used the test to model the tubular filter press (see Section 15.5.2.5).

The advantage of the CST test compared to the three methods described above, is that it is a rapid evaluation test. The principle of the test is simple. A sludge sample is poured into a bottomless cylinder resting on absorbent filter paper, which absorbs the filtrate by capillary suction. The Capillary Suction Time (CST) is the number of seconds taken by the filtrate ring to travel 1 cm. The higher the filterability of the sludge, the shorter this time.

The test is particularly suitable for the optimisation of chemical conditioning and flocculant dosing.

15.5.3
EVAPORATION PROCESSES

Evaporation processes which are commonly used in South Africa for water works sludge dewatering are:

- drying beds;
- sludge lagoons.

15.5.3.1 Sludge lagoons

Sludge lagoons are used where land and labour are available. Since lagoons are simple to operate, they are suitable for sludge dewatering and disposal for unsophisticated environments.

A sludge lagoon system is presently being operated by Rand Water at its Panfontein sludge disposal site, where sludge produced at its Zuikerbosch and Vereeniging treatment works is disposed of. The sludge lagoons were built as an interim disposal system while the mechanical thickening plant was built and now forms an integral part of the sludge disposal system (disposal of some 1 000 t of dry solids per day). The six sludge lagoons with a total capacity of 2 million m^3

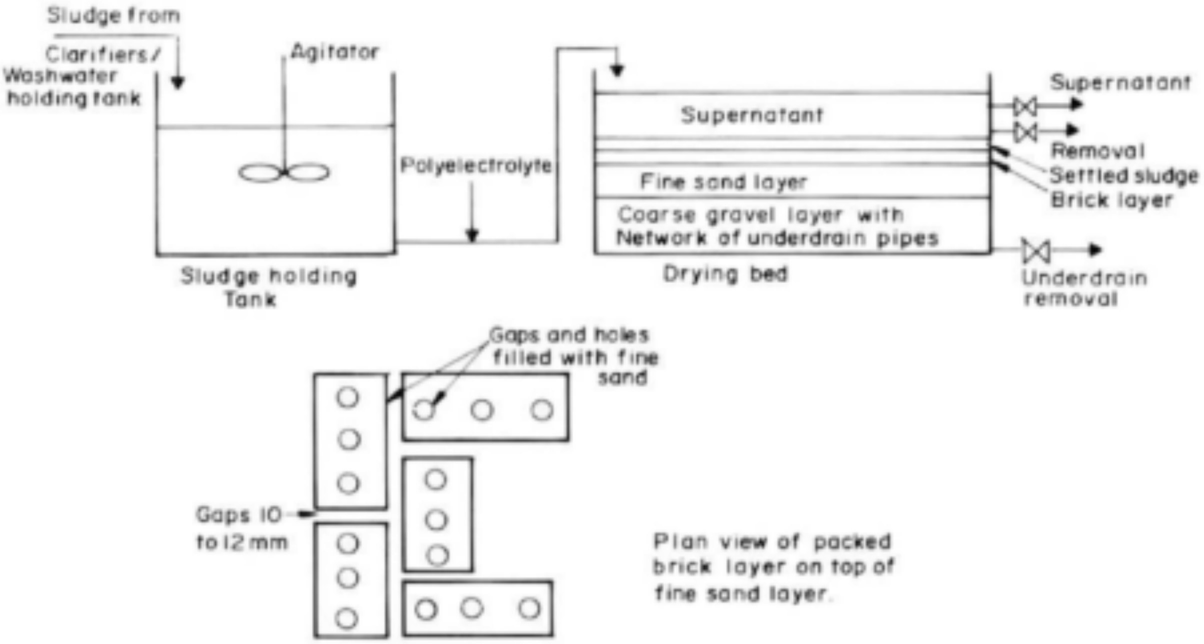


Figure 15.16
Conventional drying bed system

are also used for emergency storage while the thickening plant is out of commission or the mass of sludge produced exceeds the capacity of the thickening plant.

Each lagoon has been equipped with three inlet points for sludge and a single decant tower with adjustable penstocks for the controlled release of the clear supernatant, which flows under gravity into the Vaal Dam - Zuikerbosch canal.

In order to enhance phase separation and settling in the lagoons, sludge pumped into the lagoons is flocculated with polyelectrolyte, which is dosed upstream of the appropriate inlet chute.

Each lagoon is filled for about 5 days to the required level. The sludge is then left to settle for another 3 days before the clear supernatant is decanted.

The operation of the lagoons has proved relatively trouble-free and the solids concentration of sludge in the lagoons is on average about 35% m/m.

15.5.3.2 Drying beds

Drying beds are used where land and labour are available. Since drying beds are mechanically simple to operate, they are the preferred method of sludge dewatering for unsophisticated environments.

As shown in Figure 15.16 a well-designed drying bed system has a balancing sump or holding tank to store sludge which is discharged from the clarifiers, before it is discharged onto the drying beds. The capacity of the holding tank is normally designed so that it can accommodate one day's sludge production from the clarifiers. The sludge holding tank is normally agitated to keep the feed sludge to the drying beds homogeneous.

Sludge is pumped from the holding tank onto a preselected drying bed. In order to effect better dewatering it is advantageous to dose a polyelectrolyte into the sludge as it is pumped onto a drying bed.

It is important to have nozzles distributed around the perimeter of a drying bed in order to prevent localised accumulation of solids on certain parts of the bed.

Once a bed is loaded, sludge solids settle onto the sand bed (see Figure 15.16). Some water flows through the sand bed and is removed through a drain valve, while the supernatant on the bed is decanted through pipes and valves at predetermined levels in one wall of the drying bed.

Once the maximum amount of water has been removed by decanting and draining, the thickened sludge on the sand bed is allowed to dry.

The drying rate depends on climatic conditions. The A-pan evaporation rate (expressed as evaporation rate of water in mm/day) for a certain area may be obtained from the Weather Bureau. For the Pietermaritzburg area, for example, the A-pan evaporation rate is 1 mm/d in winter and 3 to 4 mm/day in summer (Hodgkinson, 1994).

Once the cake on the drying bed has cracked and is sufficiently dry it may be removed by manual labour using spades and then transported to a final disposal site.

Drying beds have been installed at two of Umgeni Water's water treatment plants, namely, Mill Falls near Howick and Umlaas Road near Pietermaritzburg. The cake at these two plants is removed when the solids concentration is 18 to 20% (Hodgkinson, 1994). Since significant sand losses were experienced at Mill Falls when cake was removed by manual labour, a layer of bricks was packed onto the drying beds as shown in Figure 15.16. Bricks have also been placed on the beds at the Umlaas Road water treatment plant. During the cake removal stage, dried cake is scraped off the top of the brick layer.

The addition of a brick layer onto the sand bed also allows mechanisation of the cake removal process by means of a front-end loader with a hydraulically operated shovel.

After having used sludge lagoons for sludge dewatering and disposal, Rand Water commissioned a system of drying beds in 1992 (Le Patourel, 1993). Tests on air drying of sludge demonstrated that the sludge could be dried to form an inert mass which, when dried, would not absorb water and reconstitute as a slurry. Pre-thickening of the sludge to a minimum of 16% solids concentration was required and the maximum layer depth that could be successfully air-dried was 200 mm.

Based on the experimental findings, a scheme was designed whereby sludge of a solids concentration of 2 to 3% is thickened to a minimum solids concentration of 16% and then pumped to adjacent drying beds for *in-situ* air drying.

Continuous thickeners and linear screens (see Section 15.4.3) are used for sludge thickening.

Forty drying beds with dimensions of 300 m x 100 m (depth of 2 m) each, covering a total area of 140 ha have been constructed, allowing a daily deposition of 200 mm of sludge per bed in a 40 day cycle.

At an average A-pan evaporation rate of 4 to 5 mm per day, 40 days are required to dry a 200 mm thick layer of sludge (Geldenhuys, 1992).

Decant structures and drains in the drying bed walls ensure that there is minimal freestanding water to impede drying.

After drying to a solids concentration of greater than 80%, the depth of the sludge layer decreases to approximately 30 mm and further layers of sludge are subsequently deposited on the (dried) sludge layers for air drying. Sludge is pumped to the head of each drying bed in a 200 mm diameter main and branch lines down the centre of each bed. Hydrant take off points are provided at approximately 40 m centres and 30 mm diameter high-pressure irrigation sprinklers are used to spread the sludge evenly.

Provision is also made for both harrowing of the partially dried sludge to facilitate drying, as well as mechanical compaction of the dried sludge (Le Patourel, 1993).

15.5.3.2.1 Example 3

A water treatment plant generates a thickened sludge from the bottom of the clarifiers. The maximum daily sludge production in terms of dry solids is 2,5 t of dry solids per day.

A series of drying beds is to be installed to dewater the sludge. Laboratory results have shown that after decanting and draining of underdrain water the solids concentration of the sludge on the drying bed is expected to be 8% with a specific gravity of 1,2.

The A-pan evaporation rate for the site is 1 mm of water/day in winter and 3 mm of water/day in summer. If the dry solids loading rate on the bed is 1 kg/m^2 and the time required for loading of the bed and decanting of the supernatant is 8 hours, calculate the minimum number of beds and the total drying bed area required.

Solution

The sludge is loaded at 1 kg dry solids/m^2.

The mass of wet cake (8% solids concentration):

$$= \frac{1}{0{,}08}\ \text{kg/m}^2 = 12{,}5\ \text{kg/m}^2$$

The thickness of the wet cake after decanting (at 8% solids concentration):

$$\frac{12{,}5\ \text{kg/m}^2}{1\,200\ \text{kg/m}^3} = 10{,}4\ \text{mm}$$

Number of days required for drying of the wet cake:

$$= \frac{\text{Cake thickness}}{\text{A-pan evaporation rate}}$$

$$= \frac{10{,}4\ \text{mm}}{1\ \text{mm/day}}$$

$$= 10{,}4\ \text{days (in winter)}$$

Total time for drying and loading

$$= 10{,}4 + 0{,}33\ \text{days}$$
$$= 10{,}73\ \text{days (say 11 days)}$$

Therefore a minimum of 11 beds is required.

The area of each bed

$$= \frac{2\,500\ \text{kg/day}}{1\ \text{kg/m}^2}$$

$$= 2\,500\ \text{m}^2$$

The total drying bed area required

$$= 2\,500 \times 11 = 27\,500\ \text{m}^2$$

15.6 ULTIMATE DISPOSAL AND POTENTIAL COMMERCIAL USE OF SLUDGES/CAKES

At many water authorities in South Africa, sludges are ultimately stored in sludge lagoons or evaporation ponds. At both the Western Transvaal Regional Water Company and the OFS Goldfields Water Board, for example, thickened sludge is stored in lagoons or ponds for ultimate disposal.

Apart from storing sludges in lagoons, current practice for ultimate disposal of cake produced by the dewatering of sludges from water treatment plants is co-disposal at municipal land fill sites. Since there is no land available for on-site dumping at Umgeni Water's Durban Heights and Wiggins water treatment plants in Durban, for example, a waste disposal contractor has been commissioned to transport the sludge to the nearest municipal dam site.

At Umgeni Water's DV Harris water treatment plant in Pietermaritzburg, the least expensive option was to release thickened sludge in a controlled manner into the municipal sewer. The sludge is then hydraulically transported to the municipal biological wastewater treatment plant, together with sewage.

One of the commercial options which has been considered by Rand Water is the use of dried cake as a source of agricultural lime. However, the calcium carbonate concentration is unpredictable, varying from as low as 30 per cent to as high as 70 per cent by mass, therefore, making the sludge unsuitable for this application.

Tests conducted by Rand Water have shown that stock bricks of comparable quality to clay bricks can be produced from the thickened sludge. However, unlike sludges from biological wastewater treatment plants, there is no calorific advantage in using "inorganic"sludge and therefore some form of carbonaceous material must be added. A distinct advantage of this material is that firing can be done at 700°C instead of 1 100°C making it potentially economically attractive (Geldenhuys, 1995).

16

Treatment Processes

G Juby

The objective of water purification is to economically produce an adequate quantity of potable water, by considering the raw water quality and quantity available, together with various treatment processes and operations.

Although the health and hygienic aspects of potable water are of primary importance, the water should also be aesthetically pleasing, and be non-corrosive or scale-forming in nature. Most water authorities require that a number of water quality criteria have to be met. Water treatment therefore, involves modification or alteration of one or more of the quality parameters of the water by the use of treatment processes for a particular purpose.

The water treatment processes dealt with in this chapter are those that, although widely used, are not usually encountered in conventional water works; however, as the standards for drinking water become more stringent, these processes are becoming more commonly used. For the most part these processes will be required to address the specific challenges arising from particular areas and their problematic waters.

In many cases the treatment processes are fairly specialised and to deal with them in detail is beyond the scope of this manual. However, sufficient information will be given to illustrate their potential and to serve as a preliminary guide to their use.

The following processes are included in this chapter:

- Adsorption
- Desalination/demineralisation
- Fluoridation
- Defluoridation
- Nitrate and nitrite removal
- Water softening
- Aeration
- Iron and manganese removal
- Colour removal.

16.1 ADSORPTION

Adsorption as represented by the use of activated carbon for the removal of impurities that cause tastes and odours in drinking water is a well-established treatment technology. Ion-exchange can also be regarded as an adsorption process, but it is dealt with under the **Demineralisation** heading. Similarly, defluoridation by means of activated alumina contact is dealt with separately.

Among the many problems that are encountered in water purification, particularly for domestic and certain demanding industrial applications, is the presence of constituents that give rise to odours, taste and colour. These include hydrogen sulphide, chlorophenols, trihalomethane (THM) precursors, geosmin and methylisoborneol (MIB), breakdown products of certain algae, humic and fulvic acids and iron and aluminium salts. These constituents may be substantially removed from water by conventional treatment methods such as clarification and disinfection, but some are persistent and require special treatment. One such process option is adsorption (AWWA, 1990).

Adsorption is the process whereby materials, known as *adsorbents*, are able to retain on their surfaces, substances with which they are brought into contact, such as gaseous molecules, or metallic ions, organic molecules and various colloidal particles in solution or suspension. These retained substances are known as the *adsorbates*.

16.1.1 ADSORPTION EQUILIBRIUM

Generally the process of adsorption is reversible and the adsorption-desorption process can be represented as a chemical reaction as follows:

$$A + B \leftrightarrow A.B$$

Where A represents the adsorbate, B the adsorbent and A.B the adsorbation compound. The reversibility of the reaction implies that the adsorbent can be regenerated and reused.

Activated carbon, activated alumina, iron, aluminium and magnesium hydroxides, calcium carbonate and ion exchange resins are the main adsorbents that are of interest in water treatment.

16.1.1.1 Isotherm equations

The constant-temperature equilibrium relationship between the quantity of adsorbate per unit of adsorbent q_e and the equilibrium concentration of the adsorbent in the solution C_e is called the adsorption isotherm.

Several equations are available that describe this function but only two, the Freundlich and the Langmuir are presented here. A third, the Branauer, Emmet, Teller (BET) adsorption model is valid for isotherms reflecting apparent multilayer adsorption; it is more generally applicable than the Langmuir model to which it however reduces when the level of adsorption is a monolayer. This concept is fully described in Weber (1972) and will not be dealt with here. It should be noted that studies have shown that the Langmuir isotherm is applicable to coagulation (Van Duuren, 1967). In practice the BET model is not used much in drinking water applications, but is more useful for waste water treatment studies:

- The Freundlich isotherm is an empirical equation and is useful because it accurately describes much adsorption data. It has the form:

  $$q_e = KC_e^{\ 1/n} \quad \text{.......................................} \quad 16.1$$

  and can be linearised as follows:

  $$\log q_e = \log K + 1/n. \log C_e \quad \text{.........} \quad 16.2$$

The parameters q_e (with units of *mass adsorbate/mass adsorbent* or *moles adsorbate/moles adsorbent*) and C_e (with units of *mass/volume* or *moles/volume)* are the equilibrium surface and solution concentrations respectively. The terms K and n are constants, and the units of K are determined by the units that are selected for q_e and C_e.

The constant K is related primarily to the capacity of the adsorbent for the adsorbate, and 1/n is a function of the strength of adsorption. For fixed values of C_e and 1/n the larger the value of K, the larger the capacity q_e is. For fixed values of K and C_e ,the smaller the value of 1/n, the stronger the adsorption bond is.

As 1/n, becomes very small, the capacity tends to be independent of C_e and the isotherm plot approaches the horizontal; the value of q_e then is essentially constant, and the isotherm is called irreversible. If the value of 1/n, is large, the adsorption bond is weak and the value of q_e changes markedly with small changes in C_e .

However, the Freundlich equation cannot apply to all values of C_e, for example, as C_e increases, q_e increases [in accordance with Equation (16.1)] only until the adsorbent approaches saturation. At saturation, q_e is a constant independent of further increase in C_e and the Freundlich equation no longer applies. Also no assurance exists that adsorption data will conform to the equation over all concentrations less than saturation; so care must be exercised in extending it to concentration ranges that have not been tested. (AWWA, 1990).

- The Langmuir adsorption model which has a firm theoretical basis is valid for single layer adsorption and does not take into account multi-layer adsorption, which the empirical Freundlich equation automatically does. The Langmuir equation is as follows:

$$q_e = \frac{q_{max}\, bC_e}{1 + bC_e} \quad \text{.................................. } 16.3$$

where b and q_{max} are constants and q_e and C_e are defined as for the Freundlich equation, above. The constant q_{max} corresponds to the surface concentration at monolayer coverage and represents the maximum value of q_e that can be achieved as C_e is increased. The constant b is related to the energy of adsorption and increases as the strength of the bond increases.

The values of q_{max} and b can be determined from a plot of $1/q_e$ versus $1/C_e$ in accordance with a linearised form of Equation 16.3:

$$\frac{1}{q_e} = \frac{1}{q_{max}\, bC_e} + \frac{1}{q_{max}}$$

The Freundlich equation usually describes adsorption data more accurately than the Langmuir equation, because of the heterogeneous nature of adsorbent surfaces; interaction between adsorbed molecules and other factors. While the Langmuir equation has limitations, which should be recognised, it comes into its own for describing equilibrium conditions for adsorption and for providing parameters for the determination of mass of solute absorbed per unit mass of adsorbent and b, the energy of adsorption, with which comparisons can be made between the adsorption behaviour in different adsorbate-adsorbent systems, or for varied conditions within any given system (Weber, 1972).

$$\frac{C_e}{q_e} = \frac{1}{q_{max}\, b} + \frac{C_e}{q_{max}} \quad \text{..................... } 16.4$$

Figure 16.1 shows a graphical representation of the Langmuir and Freundlich isotherms.

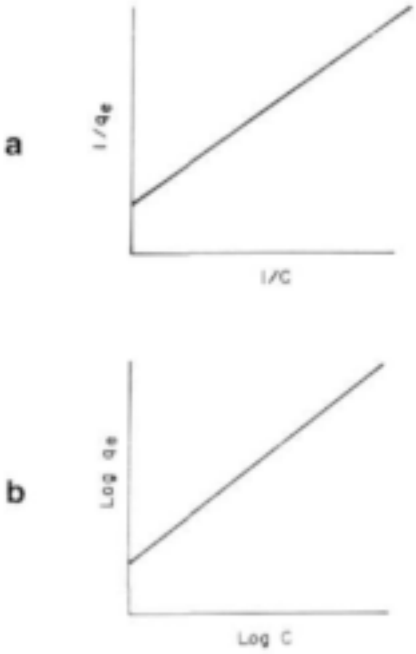


Figure 16.1

Linear forms for graphical representation of (a) Langmuir and (b) Freundlich adsorption isotherms as adapted from Weber (1972)

16.1.1.2 Adsorption capacity

The adsorption capacity of adsorbents depends on a number of physical and process variables. For activated carbon, which is of particular interest, these include:

- **Surface area or specific surface area per unit mass**, which is usually expressed as m^2/g. This is influenced by porosity and the size range of the pores; and also in specific cases by the size of the adsorbate ion, molecule or particle, which, if large, could limit accessibility to the further reaches of fine or particularly deep pores. Other factors influencing accessible surface area are the size of the carbon granules, which could vary from very fine as with powdered activated carbon (PAC) to relatively large for granular activated carbon (GAC). Industrial activated carbons have extensive surface areas in the range of 500 to 1 500 m^2/g, but as indicated, not all of this area is always accessible to the particular adsorbates.

- **The nature of the bond between the adsorbent and the adsorbate; the surface chemistry**. It is of interest to note that the activated carbon surface generally has a negative charge. Adsorbates are held to the surface of adsorbents physically by means of Van der Waal's forces and chemically by various forces such as hydrogen bonds and dipole-dipole interactions.

  With physical adsorption the adsorbate is not held as strongly to the adsorbent as in chemical adsorption. Physical adsorption is characterised by relatively low adsorption energies and is more predominant at low temperatures, while chemical adsorption is characterised by higher adsorption energies and favoured by higher temperatures (Weber, 1972). This is because chemical reactions proceed more rapidly at higher temperatures than at lower temperatures.

When considering adsorption of a solute from solution, it becomes evident that solubility is to a large extent a controlling factor for adsorption equilibria. Weber (1972) states that an inverse relationship can be anticipated between the extent of adsorption of a solute and its solubility in the solvent from which adsorption is occurring.

Additionally, it is interesting to note that adsorption from water of structurally simple organic components, that either exist as or have the potential to exist as ionic species, is least for charged species and highest for neutral species. In other words, solutes which are non-polar in nature are more strongly adsorbed than those which are polar.

The pH of the solvent solutions could be an important factor in the adsorption of solutes and in many instances neutral or near neutral pH favours higher adsorption capacity. Experimentation may be necessary to determine the optimum.

As has been mentioned, many of these adsorption-desorption reactions are reversible and substances continue to accumulate on the surface until the rate of adsorption equals the rate of desorption and a state of equilibrium has been reached when no further accumulation will occur.

16.1.2 ADSORPTION KINETICS

Several transport mechanisms are responsible for the removal of organic compounds by physical adsorption on porous adsorbents such as activated carbon. The transport occurs in a series of steps, each of which can affect the rate of removal and be the rate-limiting step in the process (AWWA, 1990), namely:

- **Bulk solution transport**: The adsorbate is moved from the bulk solution to the boundary layer of the water surrounding the adsorbent particles. This is influenced by turbulence in continuous flow and rapid mix systems.
- **Film diffusion transport**: The adsorbate is moved by molecular diffusion through the boundary layer of the water surrounding the adsorbent particle. This is influenced by the rate of flow of water as it passes the adsorbent particle.
- **Pore transport**: The adsorbate is moved through the adsorbate pores to available adsorption sites.
- **Adsorption**: The adsorption bond is formed at available sites with physical adsorption occurring relatively fast. Chemical adsorption, if it occurs, will be relatively slow, particularly at low temperatures, and may become the rate-limiting step.

16.1.3 ACTIVATED CARBON PREPARATION - A BRIEF OUTLINE

The physico-chemical nature of the adsorbent is of the utmost importance for the rate and capacity of adsorption and many substances have been examined in an attempt to achieve the optimum. However, for water and wastewater applications, activated carbon has not yet been bettered.

A variety of raw materials can be used for the commercial preparation of active carbon. These include coconut and other nut shells, wood, coal, lignite, bone and petroleum residues.

The first stage of the production process of activated carbon involves carbonisation, or pyrolysis, in the absence of air at temperatures below 700°C, to form a char. The next step is activation, which may follow quenching with water. Activation is carried out with oxidising gases such as steam and CO_2, CO and oxygen at temperatures of up to 1000°C. During this stage the porous structure of the carbon is developed by oxidation of the less stable carbon groupings within the char. Care is taken to restrict the formation of macropores and to favour a microporous structure (< 1000 Å diameter pores) (Weber, 1972).

16.1.4 CONTACTING SYSTEMS AND MODES OF OPERATION

16.1.4.1 Powdered activated carbon (PAC)

PAC in the form of fine grains of between 10 µg and 85 µg diameter is generally used to remove taste and odour problems often experienced in surface water supplies. It can be very effective and has the advantage that dosages can readily be adjusted to meet seasonal or other raw water quality fluctuations.

PAC can be used in both batch and continuous modes, although in South Africa the periodic addition of PAC is most common. Rates of adsorption on granular adsorbents are believed to be strongly dependent on the adsorbent particle size (Weber, 1972, AWWA, 1990) i.e. the smaller the particle, the higher the adsorption rate. However, recent data (Najm, 1994) suggest that it can take up to 24 hours or more for PAC to become fully loaded. It is difficult to recover the PAC for regeneration and reuse and this is not often tried in practice, and therefore unlike granular activated carbon (GAC: discussed below) PAC is usually used only once.

An important prerequisite for the successful use of PAC is that it should be mixed rapidly with the full flow of the feed water, rigorously avoiding any short-circuiting, and should remain in contact with the raw water for as long as possible in order to ensure complete utilisation of the adsorptive capacity of the carbon. Thus clarifier systems which retain a sludge blanket through which the feed water flows are an advantage downstream of the PAC dosing point.

The capital cost of establishing a PAC dosing facility is significantly less than for a GAC system. PAC is usually dosed as a slurry and the required dosing equipment is fairly simple and can vary from a manual system to make up the PAC slurry to a fully automated slurry make-up and dosing arrangement. However, PAC dosages should, depending on local costs, probably not exceed about 30 to 40 mg/ℓ for several months of the year before the cost advantage over GAC systems is lost.

The various points of PAC addition commonly used in water treatment practice include the raw water intake, the rapid mix area for coagulants and other chemical additives and the filter inlet area. The advantages and disadvantages of different addition points are summarised in Table 5.4.

On-site testing with PAC in various application sites is strongly recommended for determining the most effective and economical point of addition for particular situations.

Another alternative would be to provide a separate reactor between the sedimentation basin and the filters in order to avoid some of the disadvantages listed in Table 5.4. However, the additional capital costs could prove to be prohibitive.

A full discussion of PAC performance is given in the AWWA (1990) reference and this and Weber (1972) should be studied to obtain a broader picture of its potential. Weber (1972) makes the point that in comparing and selecting different types of PAC for the removal of taste and odour from water, the Freundlich equation is most useful in order to determine the required dosages experimentally, for each PAC type.

16.1.4.2 Granular activated carbon (GAC)

If the organic content of feedwater to a water works is fairly high and reasonably constant throughout the year and if satisfactory reduction in concentration cannot be achieved by conventional coagulation techniques, such as are often used for colour removal, it becomes more economical to use GAC in pressure or gravity contactors. [Note: GAC filters should not be used to remove turbidity, this should ideally be carried out upstream.

In pressure filters the GAC is enclosed and the filters can be operated over a wide range of flow rates. A disadvantage is that the GAC cannot easily be examined and because these filters are often cylindrical in shape, designing for a specific filter velocity and empty bed contact time (EBCT) becomes expensive. Gravity filters are preferred in larger systems, and using GAC in conventional fixed-bed adsorbers is advantageous in that very high quality water can be obtained, even with feed water of variable quality, provided that the original design is adequate. For many systems the choice between pressure filters and gravity filters is cost related.

Water may be applied to GAC in either upflow or downflow mode and upflow volumes may be either packed bed or expanded bed. The merits and demerits of the various configurations for GAC contactors are extensively covered in AWWA (1990) and by Weber (1972). Included are discussions and design information on series and parallel flow systems. A wealth of further references is provided in these publications. These deal with a wide variety of GAC applications and practical considerations.

The modern trend is to use GAC filters in combination with ozonation. It is widely accepted that ozonation of water results in the formation of biodegradable dissolved organic carbon (BDOC) or assimilable organic carbon (AOC) from the natural organic matter (NOM) present in the water. It should be recognised that other oxidants less powerful than ozone, such as chlorine, also result in the formation of BDOC/AOC - although not normally in such large concentrations as with ozone. In addition, given that one of the important characteristics of GAC is the catalytic destruction of chlorine species, having a free chlorine residual prior to GAC could be wasteful. Furthermore, pre-chlorination will also inhibit biological activity on the carbon media.

The release of this BDOC/AOC into the water distribution system (which would occur if ozonation were used as the final disinfection step) could result in regrowth problems within the distribution network. Therefore, in order to remove unwanted DOC and provide growth sites for aerobic bacteria which can assimilate the BDOC/AOC, GAC filters are used downstream of ozonation.

Often ozone is used at more than one point in the treatment process, and the place for the GAC would typically be after the final ozonation step. GAC operated in a biological mode is known as biologically enhanced activated carbon (BAC). There is considerable research being conducted in this field. It is a commonly held view that the presence of aerobic organisms in the carbon enhances the adsorptive life of the carbon, due to the fact that the organisms are feeding off the carbon, thereby keeping it clean. This has a significant impact on the operating costs of a GAC installation as it affects the time between carbon regeneration. The actual advantages of the BAC system would need to be thoroughly investigated for each different raw water to be treated. Rice & Robson (1982) provide a comprehensive review of the BAC technology.

Cognizance should be taken of the fact that although GAC may be installed to remove a specific organic compound (such as a pesticide Atrazine), the carbon, once installed, will adsorb the NOM present in the feed water, whose presence would normally not be problematic at the end of the process treatment train. The problem with this is that the GAC then becomes loaded with the 'benign' organics and has no capacity to handle the 'problem' organics when they arrive. To overcome this problem the carbon either has to be regenerated frequently (which is expensive) or operated in the BAC mode.

The design of full-scale GAC systems is usually based upon a filtration rate (m/h) and the required EBCT to achieve the desired adsorption. The EBCT adopted can vary from as low as 5 minutes up to as much as 40 minutes, depending on the objectives.

In South Africa the full-scale use of GAC in water treatment for potable use is not currently undertaken, although various pilot plant studies are being carried out. Some of the main reasons for turning to full-scale GAC systems would include legislation against the presence of pesticides and other carcinogenic organic compounds in the potable water, as has occurred in Europe and the USA, for example.

Regeneration of spent carbon is an important consideration in the decision to use GAC. Often, using GAC may be technically feasible to meet a particular water treatment objective but not economically viable due to the high costs of reactivation. Spent carbon regeneration is a highly specialised subject and the reader is referred to the AWWA (1990) as a starting point.

16.2 DESALINATION/ DEMINERALISATION

Sources of saline water include undiluted sea water, containing about 35 000 mg/ℓ of total dissolved solids (TDS), and brackish water, containing from 1 000 to 15 000 mg/ℓ of TDS (Corbitt, 1990). Brackish waters comprise highly mineralised groundwaters and diluted seawater. Desalination for high purity water applications is discussed briefly under the following technologies: ion exchange, electrodialysis, and reverse osmosis.

Growing demands for water coupled with an increase in the quality required by modern industry, mean that there is a widely felt need for low-energy but efficient water purification methods. Potential sources of water supply such as sea water and brackish waters which were formally regarded as marginal are being brought into use (Schutte, 1983). Whereas, in the past, desalination was only encountered in specialised industrial applications, the advances in membrane technology and the ability to produce cheaper robust systems mean that these systems can now compete with the more conventional treatment processes.

Demineralisation of high-salinity waters can be achieved by means of a number of processes including ion exchange, electrodialysis, and reverse osmosis.

16.2.1 ION EXCHANGE

The ion exchange process involves an exchange of ions from solution for ions retained on a reactive material. Almost all of the dissolved inorganic material found in solution is ionised forming chemically equivalent amounts of cations and anions. The specific ions removed from solution by the ion exchange process are determined by the type of reactive material (exchange resin) utilised and by the regenerant employed to provide the exchangeable ions on the exchange resin (AWWA, 1990; Weber, 1972). For example, in zeolite softening, sodium (Na^+) ions on the cation exchange resin are exchanged for calcium (Ca^{2+}) and magnesium (Mg^{2+}) in water passing through the exchanger. The hardness is removed but the treated water will contain an equivalent amount of Na^+ ions.

In the process of desalination for high-purity water applications, where all dissolved, ionised impurities are objectionable, a combination of exchange processes operated in series can remove essentially all ions and replace them with hydrogen cations (H^+) and hydroxyl anions (OH^-) which combine to form water. Thus, the exchange process contributes no dissolved solids to the treated water (Weber, 1972; Schutte, 1983).

However, in order to regenerate the resins, the exchanged ions will be released into the regeneration solution, which can often lead to disposal problems. This aspect must be considered in the overall design of an ion-exchange system.

16.2.2 ELECTRODIALYSIS

Desalination by electrodialysis (ED) operates on the principle that dissolved salts in water are in the form of electrically charged ions. The ions are transferred through membranes that are anion- or cation-selective from a dilute solution to a more concentrated one by the action of a direct current (AWWA, 1990; Schutte, 1983).

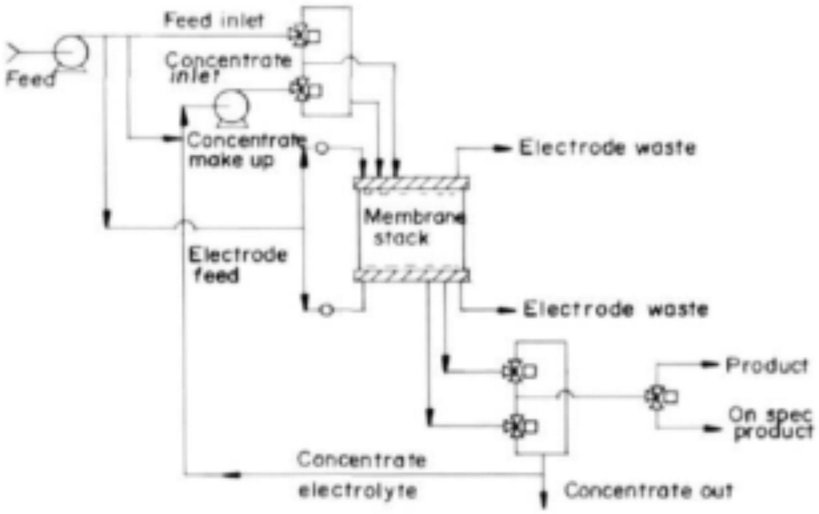


Figure 16.2

Typical electrodialysis reversal process flow diagram (AWWA, 1990)

The membranes are placed alternately between two electrodes and feed water is pumped into the spaces between the membranes. Negative ions pass through anion membranes while positive ions are rejected and *vice versa*. The ions that pass through end up in a concentrated brine solution. The result of these ionic movements is that alternate sets of compartments contain demineralised water while the other set contains concentrated brine solutions (AWWA, 1990). Due to the problem of membrane fouling by certain ionic species, specifically iron and manganese, the ED process was adapted to include reversal of the direct current (DC), which has a positive effect in reducing the fouling tendencies. This process is termed electrodialysis reversal (EDR) and is currently the most commonly used form of electrodialysis. Figure 16.2 shows a typical electrodialysis reversal process flow diagram.

Naturally, because of the sensitivity of the membranes to fouling, adequate pretreatment of the feed water is required, specifically to remove Fe(II) and Mn(II), and to reduce the scaling potential of low solubility salts, such as $CaSO_4$, $BaSO_4$, $SrSO_4$ etc. As in the case of ion exchange regeneration solutions, the EDR concentrate or brine requires disposal and this has to be addressed at the design stage. In the case of EDR, no additional ions are added to the overall system as in ion exchange. Electrical half reactions at the anode and cathode release gases which are discharged to the atmosphere in a controlled fashion, as follows:

At the cathode, water dissociates releasing hydrogen gas:

$$2H_2O + 2e^- \rightarrow 2OH^- + H_2 \;\ldots\ldots\ldots\ldots\; 16.5$$

Oxidation of the hydroxide ion occurs at the anode releasing oxygen gas:

$$2H_2O \rightarrow 4H^+ + O_2 + 4e^- \;\ldots\ldots\ldots\ldots\; 16.6$$

Formation of chlorine gas may also occur at the anode:

$$2Cl^- \rightarrow Cl_2 + 2e^- \;\ldots\ldots\ldots\ldots\; 16.7$$

CSIR (at Environmentek) may be approached for additional information on electrodialysis.

16.2.2.1 System components

A typical EDR unit consists of the following elements: pretreatment system, high-pressure pump, direct current(DC) supply, membrane stack, circulation pumps, brine disposal and membrane cleaning and electrode replacement.

- **Pretreatment**: Pretreatment requirements are somewhat more minimised in EDR systems than in say reverse osmosis systems because the reversal process aids in flushing out scale and other deposits in the cells. This automatic cleaning action eliminates the need for continuous addition of scale control chemicals (Schutte, 1983). However, removal of other constituents such as suspended solids and iron and manganese is still necessary.

- **High-pressure pump**: Feedwater goes into the feed pump at low pressure (10 to 300 kPa). The pump boosts the water pressure to operating pressures of from 500 to 600 kPa.

- **DC power supply**: Electric current is supplied to the electrodes by means of a rectifier which converts alternating current (AC) to DC power supply.

- **Membrane stack**: The membrane stacks consist of membrane arrays arranged in either hydraulic or electrical stages (AWWA, 1990 - the staging provides sufficient membrane area and retention time required for salt removal) and electrodes.

- **Circulation pumps**: These are low-pressure pumps that recycle a portion of the brine solution through the membrane stacks in order to increase recovery.

- **Brine disposal**: All membrane processes generate a concentrated stream that must be disposed of. Depending on local environmental regulations acceptable disposal methods include deep-well injection, ocean discharge or other sources of brackish waters, dilution followed by irrigation, and retention in evaporation ponds (AWWA, 1990).

- **Membrane cleaning and electrode replacement**: In all membrane systems regardless of the type of pretreatment system used, some membrane fouling will occur. The rate of fouling will vary from system to system depending on such factors as water quality, pretreatment, etc. The effects of fouling can be minimised by frequent washing of the membranes to remove the foulants, thereby restoring or improving the system production capacity. Electrodes may need replacing occasionally.

16.2.3 REVERSE OSMOSIS

When two solutions of different concentrations are separated by a membrane which is permeable by the solvent, solvent will flow from the dilute solution into the concentrated solution in order to equalise the concentrations. This phenomenon is called osmosis, and the driving force is the chemical potential on the two sides of the membrane (AWWA, 1990; Weber, 1972).

If pressure is applied to the concentrated solution, its chemical potential increases so that there is now an energy advantage in solvent travelling from the concentrated side into the dilute side, the osmosis process is reversed hence the name Reverse Osmosis. The permeate (liquid travelling through the membrane from the concentrated solution to the dilute solution) is virtually salt-free (depending on membrane rejection properties and the salt concentration in the brine solution).

A flow diagram of a typical reverse osmosis system is shown in Figure 16.3.

Detailed information on design, and other implementation aspects of membrane technologies are provided in AWWA (1990) and Schutte (1983).

16.2.3.1 System components

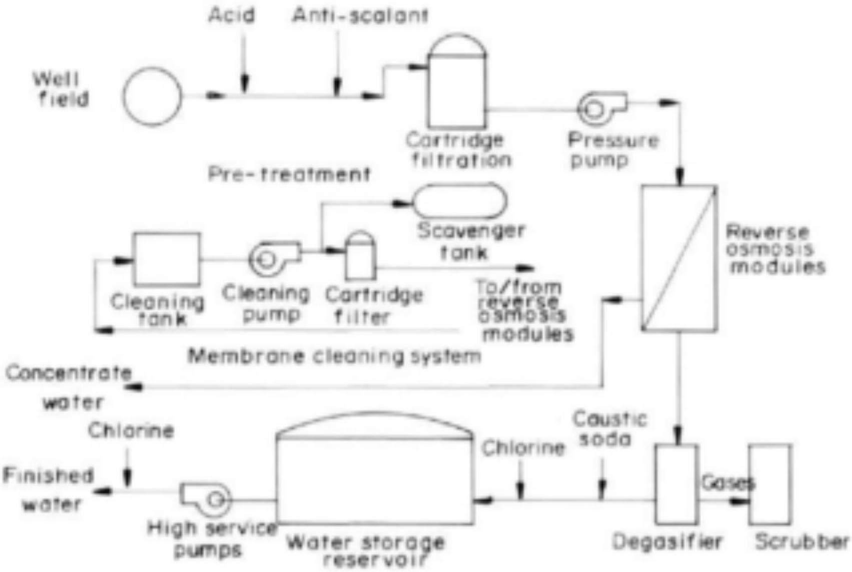


Figure 16.3
Typical reverse osmosis system flow diagram (AWWA, 1990)

A reverse osmosis system usually consists of the following elements: pretreatment system, high-pressure pump, membrane assembly, post-treatment for water stabilisation, brine disposal and membrane cleaning:

- **Pretreatment**: As with all membrane systems and ion-exchange resins, the objective of pretreatment is to reduce the potential of the incoming feed water to cause membrane fouling and chemical degradation of the membranes. This treatment usually consists of the removal of suspended solids, pH adjustment (for the removal of iron and manganese usually), dechlorination, if necessary, and the addition of a threshold inhibitor for scale control.

- **High-pressure pump**: The high pressure pump supplies pretreated feed water to the membrane system, at appropriate pressures to overcome the feed water's osmotic pressure. The appropriate pressure for brackish water desalination usually varies from 1 700 to 2 700 kPa ; but this is dependent upon the membrane type.

- **Membrane assembly**: The semi-permeable membranes limit the passage of dissolved salts, while they permit almost salt-free water to pass through. Feed water applied to the membrane assembly results in a fresh water product stream and a concentrated brine reject stream.

  The membranes, however, are not perfect in their rejection of dissolved salts and a small percentage of salts moves through the membranes and appears in the product water.

- **Stabilisation**: The product water from the membrane assembly is usually corrosive as a result of salt removal and CO_2 passage through the membrane and requires pH adjustment before being transferred to the distribution system for domestic use.

- **Brine disposal**: Once again brine disposal can be a serious problem (as with all the processes discussed in this section). Consideration should be given, at an early stage, to brine/concentrate disposal and the associated costs.

- **Membrane cleaning**: As mentioned earlier, cleaning of the membranes becomes necessary at some point in all membrane processes.

16.2.3.2 RO membrane characteristics

RO membrane and systems are usually characterised by one or more of the following:

- **Product (permeate)**: Output in litres or kilolitres (cubic metres) per unit of time (m^3/d).

- **Permeate flux**: A measure of the permeate output per unit area of membrane per unit of time ($m^3/m^2.d$ or $\ell/m^2.d$).

  Permeate flux is directly proportional to the effective driving force, i.e. the net pressure differential across the membrane or the difference between the applied pressure and the osmotic pressure difference between the feed water and the product water. For example, for brackish water in the TDS range of 1 000 to 5 000 mg/ℓ, the osmotic pressure varies from about 75 kPa to 225 kPa.

  Temperature affects the permeate flux quite considerably and the general rule is that the flux increases by about 2,5 per cent for every °C increase. In order to assess plant performance, it is, therefore, necessary to correct output or flux for temperature. The convention is to report performance at 25°C. The following formula is used:

  $$\text{Flux}_{25°C} = \text{Flux}_{T°C} + 0{,}01068\,(25°C - T°C) \quad \ldots\ldots\ldots\ldots \; 16.8$$

  where T°C is the feed water temperature.

- **Water recovery (or conversion)**: Defined as a percentage as follows:

  $$R_{rc}\,(\%) = (Q_P/Q_F) \times 100 \quad \ldots\ldots\ldots\ldots\ldots\ldots \; 16.9$$

  where:
  Q_P = product rate (feed rate - brine rate)
  Q_F = feed rate

- **Salt rejection**: The salt rejection (R_{rj}) performance of an RO membrane system is given by:

  $$R_{rj}\,(\%) = (1 - (C_P/C_F)) \times 100 \quad \ldots\ldots\ldots\ldots \; 16.10$$

  C_P is the salt concentration of the product water (product TDS)
  C_F is the salt concentration in the feed water (feed TDS)

  The salt rejection characteristic of an RO membrane is given by:

  $$R_{rj}\,(\%) = (1 - (C_P/(0{,}5(C_B + C_F))) \times 100 \ldots \; 16.11$$

  where C_B is the salt concentration in the brine (concentrate) and $0{,}5\,(C_B + C_F)$ is the average salt concentration to which the membrane is exposed.

For convenience the salt rejection is usually based on the conductivity at 25 °C of the feed, product and brine streams or on the concentrations of one or more constituents of the dissolved salts, such as sodium, chloride, sulphate, fluoride, nitrate, etc. It should be noted that salt rejection or conversely salt passage through the RO membrane is not pressure-related but directly proportional to the salt concentration in the feed water. Salt rejection for brackish water membranes is usually about 90 to 95%, while for seawater membranes the salt rejections are more than 98,5%.

TABLE 16.1
RECOMMENDED FLOURIDE LIMITS FOR PUBLIC DRINKING WATER SUPPLIES (USEPA)

Annual average of maximum daily air temperature based on temperature data obtained for a minimum of 5 years (°C)		Fluoride ion concentration (mg/ℓ)		
		Recommended limits		
	Lower	Optimum	Upper	Max. allowable
10,0-12,0	0,9	1,2	1,7	2,4
12,1-14,6	0,8	1,1	1,5	2,2
14,7-17,6	0,8	1,0	1,3	2,0
17,7-21,4	0,7	0,9	1,2	1,8
21,5-26,2	0,7	0,8	1,0	1,6
26,3-32,5	0,6	0,7	1,8	1,4

16.3 FLUORIDATION

Optimal fluoride concentrations in drinking water are believed to have beneficial health effects, in that they reduce incidences of dental caries. Communities that do not have naturally occurring fluorides in their water supplies, will sometimes need to fluoridate their water. A general discussion of water fluoridation and dosing requirements is provided.

Communities commonly add fluoride to their potable water supplies as a public health measure. Fluoridation of water supplies has been subject to many clinical trials and epidemiological studies that have demonstrated its dental-caries-inhibitory properties (AWWA, 1990). Inadequate levels of fluoride in drinking water result in increased incidences of caries (cavitation), while excessive fluoride concentrations result in a condition known as dental fluorosis - discoloured or mottled teeth (Viessman & Hammer, 1993).

16.3.1 OPTIMUM FLUORIDE CONCENTRATIONS

Fluoride concentrations that provide the greatest protection against dental caries with the least risk of dental fluorosis range between 0,7 to 1,2 mg/ℓ (Corbitt, 1990).

Optimal concentrations are local-air-temperature dependent because air temperature influences the amount of water people consume (Corbitt, 1990). Recommended limits are provided in Table 16.1.

16.3.2 FLUORIDE CHEMICALS AND CHEMISTRY

The three chemicals commonly used to adjust fluoride ion concentration in water supplies are sodium fluoride, sodium silicofluoride, and fluosilicic acid (Table 16.2) (Viessman & Hammer, 1993). Sodium fluoride and sodium silicofluoride are available in powder or crystal form and are best ap-

TABLE 16.2
COMMONLY USED CHEMICALS IN FLUORIDATION OF DRINKING WATER

Chemical name and formula	Suitable handling material	Available forms	Solubility g/100 mℓ	Specific gravity
Sodium fluoride NaF	Iron, lead, steel	odourless white powder	4 at most temperatures	2,79
Sodium silicofluoride Na_2SiF_6	Iron, lead, steel	odourless yellowish-white powder	0,44 (0°C) 2,45 (100°C)	2,68
Fluosilicic acid H_2SiF_6	Rubber lined	Liquid with a pungent odour	14,4 (20°C)	-

plied by dry feeders. Fluosilicic acid is best applied by liquid dosing pumps without prior dilution (Viessman & Hammer, 1993) because of its highly corrosive nature. It must be handled with care.

16.3.2.1 Sodium fluoride

NaF dissociates into sodium and fluoride ions when dissolved in water:

$$NaF \rightarrow Na^+ + F^- \quad \text{.......} \quad 16.12$$

Common grades of sodium fluoride typically make solutions that have a pH of about 7,6. Roughly, 2,3 kg of sodium fluoride will add 1 mg/ℓ of fluoride to 1,0 M of water.

16.3.2.2 Sodium silicofluoride

Na_2SiF_6 dissociates rapidly and completely when dissolved in water:

$$Na_2SiF_6 \rightarrow 2Na^+ + SiF_6^{2-} \quad \text{.......} \quad 16.13$$

The reaction further proceeds in two ways: hydrolysis of SiF_6^{2-} releasing fluoride ions and silica, or dissociation of SiF_6^{2-} releasing fluoride ions and silicon tetrafluoride (SiF_4) (AWWA 1990):

$$SiF_6^{2-} + 2H_2O \rightarrow 4H^+ + 6F^- + SiO_2 \quad \text{....} \quad 16.14$$

or

$$SiF_6^{2-} \rightarrow 2F^- + SiF_4 \quad \text{.......} \quad 16.15$$

Silicon tetrafluoride will volatilise if enough of it is produced, otherwise it rapidly reacts with water to form silicic acid (H_2SiO_3) and silica (SiO_2) (AWWA, 1990):

$$SiF_4 + 3H_2O \rightarrow 4HF + H_2SiO_3 \quad \text{.......} \quad 16.16$$

$$SiF_4 + 2H_2O \rightarrow 4HF + SiO_2 \quad \text{.......} \quad 16.17$$

Solutions of sodium silicofluoride are acidic, typically with a pH of about 3,6 and roughly 1,7 kg of sodium silicofluoride will add 1 mg/ℓ of fluoride to 1,0 Mℓ of water.

16.3.2.3 Fluosilicic acid

H_2SiF_6 dissociates completely in water:

$$H_2SiF_6 \rightarrow 2HF + SiF_4 \quad \text{.......} \quad 16.18$$

$$SiF_4 + 2H_2O \rightarrow 4HF + SiO_2 \quad \text{.............} \quad 16.19$$

$$SiF_4 + 3H_2O \rightarrow 4HF + H_2SiO_3 \quad \text{..........} \quad 16.20$$

Solutions of 20 to 35% fluosilicic acid will have a pH of about 1,2 and roughly 5,5 kg of 23% fluosilicic acid will add 1 mg/ℓ of fluoride to 1,0 Mℓ of water.

Other fluoride chemicals include ammonium silicofluoride, magnesium silicofluoride, potassium fluoride, hydrofluoric acid, and calcium fluoride.

16.3.3 DESIGN CONSIDERATIONS

Generally, the size and type of water facility, chemical availability and cost, and availability of operating personnel will determine the best design of fluoridation systems (AWWA, 1990). Small utilities tend to use liquid dosing pumps or saturators to apply fluoride solutions prepared in batches, while large plants use either gravimetric dry feeders to apply fluoride compounds or solution feeders to inject full strength acid directly from the shipping container (Viessman & Hammer, 1993; AWWA, 1990).

Additional factors important in the selection of fluoridation systems include fluoride injection point, safety, and waste disposal. In general, fluoride chemicals are best applied in the mains coming from the filters or at the clearwell to avoid fluoride losses that could occur at the filters due to reactions with other water treatment chemicals (Viessman & Hammer 1993; AWWA 1990), in particular, if heavy doses of alum and calcium are present from coagulation and lime-softening processes.

Safety is an important consideration to plant operational personnel when handling water treatment chemicals. The ubiquitous risk of overexposure cannot be ignored. Dust is a particular problem with the use of the dry fluoride compounds. Fluosilicic acid causes a "delayed burn" to skin tissue (AWWA, 1990).

Empty chemical containers must be disposed of in accordance with local environmental ordinances. Manuals describing considerations, alternatives, safety and other pertinent issues to fluoridation of water supplies are available to assist design engineers in selecting optimal fluoridation systems. These and many other references are presented (AWWA, 1990).

16.4 DEFLUORIDATION

Some water supplies contain high concentrations of naturally occurring fluorides. Where such concentrations exceed the recommended health limits (0,7 to 1,2 mg/ℓ), defluoridation may become necessary. A brief discussion of methods and recommendations is provided.

Fluoride is a common constituent of many minerals including apatite, fluorspar and cryolite (DWAF, 1993). It occurs naturally in small but varying amounts in practically all soils and water supplies (AWWA, 1990). Concentrations in excess of 2 mg/ℓ in drinking water are generally considered harmful to human health in that prolonged consumption of such water can lead to dental fluorosis (discolouration of tooth enamel) (Potgieter, 1990).

Current defluoridation methods include activated alumina (a form of ion-exchange), reverse osmosis, precipitation with lime and adsorption with alum.

16.4.1 ACTIVATED ALUMINA

Activated alumina is widely accepted as the exchange medium of choice for fluoride removal because of its low cost, greater ion exchange capacity and greater selectivity for fluoride ions than any other synthetic resins (ASCE/AWWA, 1990).

The affinity for common ions on acid-treated activated alumina is in the following order:

$OH^- > PO_3^{3-} > F^- > SO_3^{2-}; [Fe(CN)_6]^{4-}; CrO_4^{2-} > SO_4^{2-} > [Fe(CN)_6]^{3-}; CrO_7^{2-} > NO_2^- > Br^- > Cl^- > NO_3^- > MnO_4^- > ClO_4^- > CH_3COO^-$
(Schoeman *et al.*, 1982)

In the treatment process, the more strongly adsorbed ions can be used to displace (or regenerate) those less strongly adsorbed. Hydroxyl anions can, therefore, be used to displace fluoride anions, and caustic soda is normally used as a regenerant. Sulphate ions, which are less strongly adsorbed than fluoride ions, can be used to displace fluoride ions,provided the sulphate ion concentration is high enough, and aluminium sulphate (alum or dilute sulphuric acid) can, therefore, also be used as regenerants.

Detailed design information including costs and pilot plant performance data on fluoride removal by activated alumina and reverse osmosis is provided in a report by Schoeman *et al.*, (1982). ASCE/AWWA (1990), also contains useful design information on fluoride removal by activated alumina.

A typical adsorption-regeneration cycle for the removal of fluoride can be conceptualised as follows:

16.4.1.1 Acidification

When neutral (water washed) alumina is treated with acid, for example hydrochloric acid (HCl), acidic alumina is formed.

$$\text{Alumina.HOH} + \text{HCl} \rightarrow \text{Alumina.HCl} + \text{HOH} \quad \text{.......... 16.21}$$

16.4.1.2 Ion exchange

If this acidic form is contacted with fluoride ions, they displace the chloride ions. (Note: This exchange is best carried out in slightly acidic conditions to avoid the competition for active sites by hydroxyl ions which are more strongly adsorbed).

$$\text{Alumina.NCl} + \text{NaF} \rightarrow \text{Alumina.HF} + \text{NaCl} \quad \text{.......... 16.22}$$

16.4.1.3 Regeneration

To regenerate the alumina, which is saturated with fluoride, a dilute solution of the most preferred ion hydroxide is used.

$$\text{Alumina.HF} + 2\text{NaOH} \rightarrow \text{Alumina.NaOH} + \text{NaF} + \text{H}_2\text{O} \quad \text{........ 16.23}$$

16.4.1.4 Neutralisation

To restore the fluoride removal capacity, the basic alumina is contacted with an excess of dilute acid (e.g. HCl).

$$\text{Alumina.NaOH} + 2\text{HCl} \rightarrow \text{Alumina.HCl} + \text{NaCl} + \text{HOH} \quad \text{......... 16.24}$$

The acidic alumina, Alumina.HCl, is now ready for another fluoride adsorption cycle. For economic reasons, sulphuric acid rather than hydrochloric acid is used for acidification. This can be represented as follows:

$$2\text{Alumina.NaOH} + 3\text{H}_2\text{SO}_4 \rightarrow 2\text{Alumina.H}_2\text{SO}_4 + \text{NaSO}_4 + 2\text{H}_2\text{O} \quad \text{..16.25}$$

A caustic soda solution of 1% is usually used to strip the fluoride from the alumina. Following this, the medium is rinsed to remove excess regenerant and neutralised with dilute sulphuric acid (0,05N). With alum regeneration, on the other hand, no neutralisation step is necessary and it therefore appears to be the simpler process and more suitable for unskilled operators.

According to the literature, there is an optimum pH of the raw water for fluoride removal. This optimum pH is claimed to be between 5 and 6 and is usually achieved by dosing sulphuric acid. In this pH range, the selectivity of the activated alumina for fluoride is at its highest and the effect of competing ions is minimised.

16.4.2 FLUORIDE REMOVAL BY REVERSE OSMOSIS

Reverse osmosis (RO) is not specific for the removal of fluorides or nitrates, and is usually used for the removal of the whole spectrum of dissolved salts present in the water. However, it is clear from the literature that fluorides and nitrates can substantially be removed by RO although the process may be expected to be too expensive and inappropriate for single contaminant removal. Further information on the reverse osmosis process is referenced in Section 16.2.3.

16.4.3 FLUORIDE REMOVAL BY PRECIPITATION WITH LIME

In this process fluoride can be removed by precipitation as calcium fluoride (CaF_2) by adding lime ($Ca(OH)_2$) according to the reaction:

$$Ca(OH)_2 + 2F^- \rightarrow CaF_2 + 2H_2O \text{ .... } 16.26$$

Potgieter (1990) conducted an experimental assessment of defluoridation methods and found that lime addition could only achieve a 35% removal efficiency. Furthermore, he reported that the CaF_2 precipitate formed into colloidal particles which can cause separation problems unless a coagulation step is added to ensure proper fluoride removal. The low removal efficiency was apparently due to the high solubility of CaF_2 (approx. 16 mg/ℓ) and a high calcium demand to satisfy the solubility product.

Earlier work on Sasol effluent also gave poor efficiency, indicating lime to be a poor option for fluoride removal.

16.4.4 FLUORIDE REMOVAL WITH ALUM

Removal of fluoride ions by alum from water is said to be very effective if used with lime addition as a first step, followed by alum addition to cause coagulation (Potgieter, 1990). The addition of alum results in the following reactions:

$$Al_2(SO_4)_3.14H_2O + 3Ca(HCO_3)_2 \rightarrow 2Al(OH)_3 + 3CaSO_4 + 14H_2O + 6CO_2 \text{ .... } 16.27$$

$$Al_2(SO_4)_3.14H_2O + 6F^- \rightarrow 2AlF_3 + 3SO_4^{2-} + 14H_2O \text{ .... } 16.28$$

16.5 NITRATE/NITRITE (NO_3^-/NO_2^-) REMOVAL

Nitrates in drinking water are believed to cause methemoglobinemia known as the "blue baby syndrome" in babies under 3 months of age who are fed formulas prepared with water containing high nitrates (>10 mg/ℓ as nitrogen) (Corbitt, 1990). Typical concentrations of naturally occurring nitrogen are below 5 mg/ℓ as nitrate-nitrogen (DWAF, 1993), and the presence of higher

NO_3^-/NO_2^- is usually an indication of contamination of the water source by treated sewage or industrial effluents (Greenblau, 1990).

Removal of nitrates from drinking water can be achieved via such methods as distillation, reverse osmosis, biological reduction or ion exchange. The former two methods are relatively expensive and it is unlikely that they would be used to remove a single contaminant. Biological denitrification is generally used in wastewater treatment but could be an option in a particular water treatment application. However, one factor to consider would be the cost of providing a readily biodegradable carbon source (such as methanol). Currently, the most widely used method to remove nitrates from water is ion exchange using weak base and strong base nitrate-selective resins (ASCE/AWWA, 1990). A brief discussion is provided below.

16.5.1 ION EXCHANGE

A process description of ion exchange was provided earlier in Section 16.2. In single contaminant removal applications (e.g. nitrate or flouride) both selectivity for the ions to be removed and reversibility play an important role in process economics, i.e. the ions to be removed have to be more readily and preferentially adsorbed on the exchange medium than all the other dissolved ions that may be present in the feedwater and the exhausted exchange medium be easily regenerable with a concentrated solution of the originally bound ions.

The most widely used exchange media are synthetically produced so that it is possible to customise the selectivity of the exchange medium for removal of a specific contaminant of concern.

In the 1970s, a 6,5 Mℓ/d continuous countercurrent ion exchange plant was operated in the United States using a strong base resin and sodium chloride regenerant, and is reported to have achieved >98% nitrate removals.

16.5.1.1 Design considerations

Important parameters that need examining with ion exchange include suspended solids, the overall ionic composition of the water to be treated, and the removal of eluates from the regeneration process (Degremont, 1991). Pretreatment should be considered when treating water, with say, high turbidity for example.

Further information including references that provide design data for nitrate removal by ion exchange, is provided in ASCE/AWWA (1990).

16.6 WATER SOFTENING

Hardness in domestic or industrial water supplies is undesirable. Hard water retards the action of soaps and detergents, making it necessary to expend greater quantities of these cleaning agents. Furthermore, hard water forms scale deposits in the heating coils of boilers, kettles and/or in cooking utensils, with a consequent waste of fuel. A discussion of water softening processes will be provided under the following: chemical softening, ion-exchange and membrane softening.

Hard water can be problematic in domestic water systems, causing scaling and excessive usage of soap. The scaling encountered in fresh water distribution systems results from the presence of excessive alkalinity and calcium in the water. The scale formed is largely calcium carbonate and tends to predominate in hot water systems

TABLE 16.3
CLASSIFICATION OF WATER ACCORDING TO HARDNESS CONTENT

Hardness range (mg/ℓ CaCO$_3$)	Hardness description
0-50	Soft
50-100	Moderately soft
100-150	Slightly hard
150-200	Moderately hard
200-300	Hard
>300	Very hard

where its solubility product is exceeded. A method of overcoming water hardness problems is to soften the water.

16.6.1 WATER HARDNESS

There are five ways of expressing water hardness (Benefield *et al.*, 1982):

- **Total hardness**: the sum of the calcium and magnesium ion concentrations expressed as mg/ℓ of CaCO$_3$.
- **Carbonate hardness**: hardness equal to or less than the total alkalinity.
- **Non-carbonate hardness**: hardness exceeding the carbonate hardness, i.e. the difference between total hardness and carbonate hardness.
- **Calcium hardness**: the portion of the total hardness due to the calcium ion.
- **Magnesium hardness**: the portion of the hardness due to the magnesium ion.

Carbonate hardness is often referred to as the "temporary" hardness of a water, whereas the non-carbonate hardness is often referred to as the "permanent" hardness. Table 16.3 presents an international classification of water qualities according to their total hardness (Benefield *et al.*, 1982). The SABS guidelines for drinking water quality (SABS 241, 1984) give a range of between 20 (minimum) and 300 (maximum) mg/ℓ as CaCO$_3$ as the recommended range for the continuous use of water.

There are three basic methods that can be used to soften water: chemical softening, ion exchange and membrane softening:

16.6.2 CHEMICAL SOFTENING

Chemical softening involves chemical reactions that remove Ca^{2+} and Mg^{2+} ions from water in the form of CaCO$_3$(s) and Mg(OH)$_2$(s) precipitates. The methods most widely used are lime-soda softening and soda-ash softening processes. The advantage of chemical softening is that it can be carried out using processes which are common to water treatment, such as rapid mix, flocculation and sedimentation arranged sequentially in separate reactors or in a solids-contact softener where they occur in a single unit.

Information regarding modelling programmes which can be used to predict chemical doses for softening is presented in Chapter 13 on Stabilisation.

16.6.2.1 Lime-soda softening

The lime-soda softening process can be used to remove calcium and magnesium from water. It is also a useful process for the removal of dissolved silica and fluoride which are removed by adsorption onto the magnesium hydroxide flocs which are formed.

The reactions that occur when a water

containing both carbonate and non-carbonate hardness is softened by the addition of lime and soda ash, fall into five different groups. Lime is normally added to the water first.

(i) The first reaction does not produce any softening but it should be considered since it is responsible for a lime demand - it is the reaction between lime and carbonic acid present in the water:

$$\begin{gathered} H_2CO_3 + Ca(OH)_2 \rightarrow \\ CaCO_3\downarrow + 2H_2O \end{gathered} \tag{16.29}$$

(ii) The second reaction involves the removal of calcium carbonate hardness which is the calcium that can be removed by converting the alkalinity (bicarbonate) to carbonate and precipitating calcium carbonate.

$$\begin{gathered} Ca^{2+} + 2HCO_3^- + Ca(OH)_2 \rightarrow \\ 2CaCO_3\downarrow + 2H_2O \end{gathered} \tag{16.30}$$

Therefore the addition of lime alone will only remove the temporary hardness. This process on its own is often referred to as "cold" lime softening and is normally carried out at pH values of around pH 8.

(iii) At this point the availability of bicarbonate has been exhausted and to achieve further precipitation the addition of carbonate ions from an external source is required. Soda ash is used for this purpose and the calcium non-carbonate hardness is removed:

$$\begin{gathered} Ca^{2+} + SO_4^{2-} + Na_2CO_3 \rightarrow \\ CaCO_3\downarrow + Na_2SO_4 \end{gathered} \tag{16.31}$$

(iv) For the removal of the magnesium carbonate hardness there are two reactions involved. Equation 16.32 represents the situation where any bicarbonate alkalinity in excess of the amount associated with calcium carbonate hardness has to be neutralised with lime:

$$\begin{gathered} Mg^{2+} + 2HCO_3^- + Ca(OH)_2 \rightarrow \\ CaCO_3\downarrow + MgCO_3 + 2H_2O \end{gathered} \tag{16.32}$$

The magnesium carbonate formed in this reaction requires additional lime at a pH of between 10 and 11,5 in order to be removed as a magnesium hydroxide precipitate:

$$\begin{gathered} MgCO_3 + Ca(OH)_2 \rightarrow \\ CaCO_3\downarrow + Mg(OH)_2\downarrow \end{gathered} \tag{16.33}$$

(v) The removal of magnesium non-carbonate hardness requires the addition of lime as well as soda ash.

$$\begin{gathered} Mg^{2+} + SO_4^{2-} + Ca(OH)_2 + NaCO_3 \rightarrow \\ CaCO_3\downarrow + Mg(OH)_2\downarrow + Na_2SO_4 \end{gathered} \tag{16.34}$$

Due to the high pH associated with the final softened water and its suspended solids content, two additional steps are required in the process to produce a water of suitable quality. The first is neutralisation and the second is filtration.

A variety of process equipment is used to soften lime-soda. For example, conventional clarifiers can be used as the contact and precipitation reactors, or more novel pebble bed reactors can be used in which the precipitation of calcium carbonate occurs onto "pebbles" of the compound in the reactor. The choice of process equipment can affect the capital cost of the process and its degree of complexity of operation. However, for a given feedwater quality the cost of chemicals will be essentially the same; and the stoichiometric dosages can be calculated from the above equations.

16.6.2.2 Sodium hydroxide - Soda ash softening

An alternative to lime-soda softening is the sodium hydroxide (caustic soda) - soda softening process. Here the caustic soda is used to remove both carbonate and non-carbonate hardness in the place of lime. There are five groups of reactions occurring within this process:

(i) The reaction of caustic soda with carbonic acid. As in the case of lime-soda softening this reaction does not produce any softening but does represent a chemical demand.

$$H_2CO_3 + 2NaOH \rightarrow Na_2CO_3 + 2H_2O \ldots 16.35$$

During this reaction sodium carbonate is formed and will be consumed later in the process as the demand for carbonates increases.

(ii) Equation 16.36 represents the removal of calcium carbonate hardness:

$$Ca^{2+} + 2HCO_3^- + 2NaOH \rightarrow CaCO_3\downarrow + Na_2CO_3 + 2H_2O \ldots\ldots 16.36$$

Once again, there is the formation of sodium carbonate (soda ash) is formed.

(iii) There are two reactions associated with the removal of magnesium carbonate hardness, as was the case with the lime-soda process.

$$Mg^{2+} + 2HCO_3^- + 2NaOH \rightarrow Na_2CO_3 + MgCO_3 + 2H_2O \ldots\ldots 16.37$$

$$MgCO_3 + NaOH \rightarrow Mg(OH)_2 + Na_2CO_3 \ldots\ldots 16.38$$

(iv) The final reaction involves the removal of magnesium non-carbonate hardness

$$Mg^{2+} + SO_4^{2-} + 2NaOH \rightarrow Mg(OH)_2 + Na_2SO_4 \ldots\ldots 16.39$$

(v) The soda ash formed in reactions 16.35, 16.36, 16.37, and 16.38 will react with the calcium non-carbonate hardness

$$Ca^{2+} + SO_4^{2-} + Na_2CO_3 \rightarrow CaCO_3\downarrow + Na_2SO_4 \ldots\ldots 16.40$$

The addition of soda ash to the system will only be required if the demand for carbonate within the system is not satisfied by the sodium carbonate formed.

The relative cost of the two chemical softening methods will depend upon the chemistry of the raw water, but generally speaking for partial softening the lime process will be the least expensive alternative because of the cost difference between lime and caustic soda. However, this would need to be verified in each case.

16.6.3 ION EXCHANGE

Ion exchange, as the name suggests, involves the exchange of an ion in the feed solution with a different ion from the ion exchange resin as discussed in Section 16.2. earlier. The exchanged ion remains bound to the resin and is consequently removed from solution.

The resins employed in softening systems are generally cation exchange resins in the sodium form (Benefield *et al.*, 1982). Typical hardness exchange can be represented by the following equation:

$$2RNa + CaSO_4 \leftrightarrow R_2Ca + Na_2SO_4 \;.\; 16.41$$

The removal of calcium ions from solution is accompanied by the build-up of sodium ions from the resin. When the resin is exhausted (i.e. 95% or more of the resin is in the R_2Ca form) the resin is regenerated with a strong sodium solution, usually sodium chloride. This causes the calcium ions to be replaced on the resin with sodium ions, and the calcium containing solution is disposed of as waste. The disposal of this waste brine and the resin rinse solutions must be considered as part of the economic feasibility of this process. In the form discussed here, ion exchange actually contributes to increasing the total dissolved solids (TDS) of the solution since the equivalent mass of sodium (23 mg/meq) which enters the water is greater than that of calcium (20 mg/meq) which is removed from the water.

16.6.4 MEMBRANE SOFTENING

Unlike the more conventional softening process of lime-soda, the use of membranes to soften water is a much more recent development.

The development of synthetic membranes really only came into its own after World War II. Continued research has aided the development of so-called low-pressure reverse osmosis (RO) membranes - whose operating pressures of around 1 700 kPa are about half of those of their predecessors. Softening membranes can operate at even lower pressures - around 470 kPa (Hornburg & Morin, 1986).

Apart from the operating pressure of a membrane softening plant, other factors that determine the cost of the process are the membrane flux and the salt rejection of the membrane. As discussed in Section 16.2.3, membrane flux is a measure of the flow rate of permeate per unit area of membrane. It is normally expressed as $\ell/m^2.d$ and is corrected for temperature and pressure. A low flux rate would mean a larger membrane area for a given duty and therefore a more capital intensive process. The salt rejection relates to the percentage of salt that the membrane retains. For softening systems the particular salts of concern are naturally calcium and magnesium.

The specific salt rejection of a membrane will determine the overall degree of softening that can be obtained.

16.6.4.1 Membrane types

There are many membrane types. Traditionally, reverse osmosis (RO) membranes are of a rather "tight" character, rejecting most monovalent (Na^+) and divalent (Ca^{2+}) ions at efficiencies of greater than 95%. Since the manufacture of early RO membranes, membrane designers have been able to produce a much wider range of membranes with varying degrees of "tightness"; so that nowadays it is possible to almost customise the membranes for a given overall salt rejection and water treatment objective.

Various types of membrane can be used for softening and these are generally classed in the nanofiltration group. Even the use of charged membranes has also been investigated (Wensley *et al.*, 1980). Electrodialysis reversal (EDR) can also be used as a softening process.

Generally, the membranes are still grouped together into the following categories (Table 16.4). For completeness the non-ionic species rejection microfiltration membranes have also been included.

16.6.4.2 Membrane configurations

The many membrane types can be installed in a number of configurations, the most common of which are: spiral wrap, hollow fine fibre and tubular (Figure 16.4). In the spiral wrap arrangement the membranes are in the form of flat sheets which are then rolled together with membrane spaces to give the spiral wrap configuration. Hollow fine fibre modules consist of thousands of fine tubules of membranes (less than 1 mm in diameter) bound together. Feed water usually enters on the inside of the fibres and the permeate

TABLE 16.4
MEMBRANE PROCESSES

Process	Operating pressure kPa	Particle size microns	Molecular weight cut-off MWC
Microfiltration	<100 - 500	0,0775 - 5,5	100 000 - 500 000
Ultrafiltration	70 - 700	0,00325 - 0,325	1 000 - 50 000
Nanofiltration	310 - 1 000	0,001 - 0,0325	200 - 500
Reverse osmosis	1 000 - 10 000	0,0001 - 0,0055	100

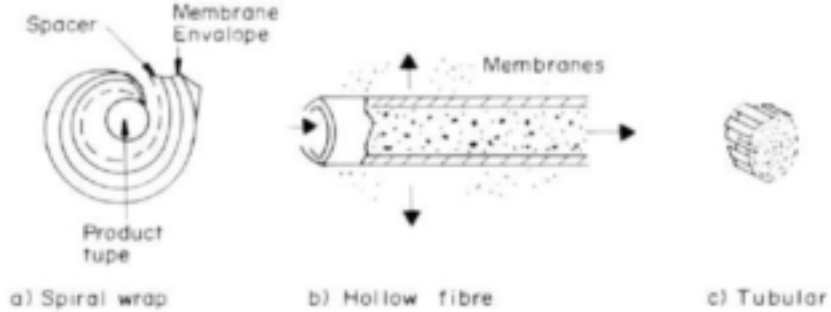


Figure 16.4
Membrane configurations: (a) spiral wrap; (b) hollow fibre; (c) tubular (AWWA, 1990)

passes through the membranes. The tubular system, as the name suggests, consists of membranes in a tubular form (typically 12,5 mm diameter) arranged in a module. Feed solution enters the inside of the tube and the permeate is collected on the outside within the module casing.

In South Africa, technologies have been developed, through the Institute of Polymer Science at the University of Stellenbosch, to manufacture membranes. Presently local ultrafiltration and RO membranes are commercially available in the tubular configuration.

16.7 AERATION

Aeration has a long history in water treatment. However, it will only receive attention insofar as it relates to the pre-oxidation step in the removal of iron and manganese from water for general domestic use.

The many uses of aeration systems in water treatment include applications such as stripping of free carbon dioxide, hydrogen sulphide and volatile organics. The need for aeration at the head of the process treatment train largely depends on the source of the raw water. If it is anaerobic in nature, it is more likely to contain odorous gases and greater concentrations of iron and manganese in which case an aeration step may be warranted.

In the past aeration was used for the oxidation of iron and manganese, but as discussed below, unless the conditions are just right, aeration is not particularly effective. Iron and manganese removal is discussed in more detail in Section 16.8.

These days, ozonation is becoming the most popular first step in the process treatment train because of its many uses (see section 16.9.3) and, because ozone is associated with either air or oxygen (depending upon the ozone generation process) aeration of the raw water is a natural consequence of ozonation.

16.7.1 TYPES OF AERATOR

The objective of aeration is to expose as much of the water as possible to the gas phase. There are several commonly used methods of achieving this in water treatment, including the waterfall types of aerator such as spray nozzles, cascades, and multiple trays; diffusion or bubble aerators; and mechanical aerators. The waterfall types of aerator breaks up the water into droplets or thin films, thereby increasing the water surface exposed per unit volume. The diffusion types of aerator bubbles compressed air through the water, and the mechanical aerators use motor-driven impellors to agitate the water (ASCE/AWWA, 1990).

Theoretical principles and design considerations of aeration systems are covered extensively by ASCE/AWWA (1990) and AWWA (1990).

16.7.2 IRON REMOVAL

The oxidation of iron by oxygen is very dependent upon pH, and Applegate & Sackinger (1987) report that an increase of one pH unit, when the pH is initially below 7, increases the oxidation rate by a factor of 100.

The rate of oxidation at a pH below 7 is represented by Equation 5.8 in Chapter 5, is

$$-d[Fe^{2+}]/dt = k\,[Fe^{2+}][OH^-]^2[O_2] \quad ..... \; 16.42$$

The hydroxyl ion concentration to the power of 2 indicates the strong dependency of the rate on pH. Wong (1984) states that the removal of iron by oxidation with air is generally recommended for water with high concentrations (>5,0 mg/ℓ).

16.7.3 MANGANESE REMOVAL

Knocke *et al.* (1987) report that aeration as the sole means of Mn(II) oxidation is not commonly practised because the process is kinetically slow and very pH-dependent. Below a pH of 8,6 manganese oxidation may be poor, and not until the pH reaches 9,5 is the oxidation rate considered acceptable (Wong, 1984).

16.8 IRON AND MANGANESE REMOVAL

Small amounts of iron and manganese are often present in water because of the large amounts present in the soil. Iron and manganese are undesirable in domestic waters because they cause aesthetic problems such as brownish coloured water, turbidity, staining and bad taste.

Anon (1987) reports that concentrations of iron above 0.3 mg/ℓ as Fe and manganese above 0,05 mg/ℓ as Mn can lead to these aesthetic problems. A general discussion of methods and recommendations is provided.

The presence of iron and manganese in water has long been a serious problem in public water supplies (Wong, 1984); and this is substantiated by a report (Anon, 1987) that in the United States iron and manganese exceed the recommended secondary maximum concentration levels in roughly 40% of public water supplies. The South African Bureau of Standards (SABS) recommended limits for iron and manganese in drinking water are 0,1 mg/ℓ Fe and 0,05 mg/ℓ Mn (SABS, 1984), respectively, and iron and manganese removal tends to be a consideration of most water treatment works at some time.

The presence of iron and manganese can also be problematic in non-potable water systems. For example during the desalination of water using membrane processes such as reverse osmosis and electrodialysis, iron and manganese can foul the membranes, thereby reducing membrane life and process efficiency (Applegate & Sackinger, 1987; Juby, 1989).

The most common method of iron and manganese removal is the oxidation of the relatively soluble Fe (II) and Mn (II) to insoluble Fe (III) and Mn (III, IV). This is followed by settling or filtration to remove the Fe (III) and Mn (III, IV) precipitates. As discussed in Section 5.6 earlier, many different oxidants have been suggested, including oxygen, chlorine, ozone, potassium permanganate and chlorine dioxide. However, there is a shift away from the conventional oxidants such as air and chlorine to the more powerful oxidants of chlorine dioxide, potassium permanganate and ozone. The reason for this is the rate at which oxidation takes place and the effects of the physico-chemical conditions of the system such as pH, temperature and organics content.

It is generally agreed that iron is more readily oxidised than manganese because it has a fast rate of oxidation. However, this is complicated when the ion is associated with natural organics present in the water. Mn (II) is oxidised to Mn (III, IV) by the same oxidising agents as Fe (II), however longer reaction times are required, and in certain cases the pH plays a far more significant role. Some details regarding the use of potassium permanganate, chlorine dioxide and ozone are given below.

16.8.1 OXIDATION WITH POTASSIUM PERMANGANATE ($KMnO_4$)

Although some large water works (such as Umgeni Water's Wiggins works) have used $KMnO_4$ to remove iron and manganese and there are examples of its use in non-potable water treatment, $KMnO_4$ is not widely used in South Africa. This may be because some of the advantages for using $KMnO_4$ have only appeared in the literature in recent times, as is discussed in this section.

16.8.1.1 Iron removal

In water systems free of organic material the oxidation of Fe (II) using $KMnO_4$ is extremely rapid, even under cold water temperature conditions (2°C) (Knocke *et al.*, 1991a). The reaction time in this case is only 0,1 seconds.

Theoretically, the stoichiometric dosage of $KMnO_4$ required to oxidise 1 mg/ℓ of Fe (II) is 0,94 mg $KMnO_4/\ell$. Tests carried out in organic-free waters have shown the observed stoichiometric requirement to be 0,924/mg Fe (II). Even under acid conditions Fe (II) is oxidised readily by $KMnO_4$ (Applegate & Sackinger, 1987).

In contrast to this Knocke *et al.* (1991a) report that the effectiveness of oxidation of Fe (II) using $KMnO_4$ in the presence of fulvic and humic organic acids was greatly reduced. When fulvic acids were present contact times in excess of an hour were required, and in both cases excess oxidant was required in order to satisfy the competitive demand exerted by the organic matter. The resistance to oxidation has been put down to the formation of complexes with the organics.

16.8.1.2 Manganese removal

Theoretically, 1,92 mg/ℓ of $KMnO_4$ is required to oxidise 1 mg of Mn(II), and observations (Knocke *et al.*, 1991a) showed that 1,93 mg of $KMnO_4$/mg of Mn (II) are actually required. This is different to what Wong (1984) noticed in practice. He reports that the required dosage is generally lower than the theoretical dosage. Knocke *et al.*, (1991a) found that temperature, pH and organics content affect the manganese removal reaction time. At low pHs (pH 5) in organic free water the oxidation was reportedly complete in less than 20 seconds.

Juby & Pulles (1990) report poor oxidation of Mn (II) in mine service water at a pH of approximately 6 and with 1,5 minutes retention time. This may have been due to the presence of organics in the mine water, since Knocke *et al.* (1987) report that for low TOC (< 3 mg/ℓ) waters most Mn (II) oxidation occurs within 5 minutes. The type of organics present also seems to be important since in more recent work (Knocke *et al.*, 1991a) it is reported that for a fulvic acid concentration of 10 mg/ℓ the concentration of Mn (II) was reduced by half in 6-7 seconds. Either way the presence of organics increases the oxidant demand and this should be accounted for.

16.8.2 OXIDATION WITH CHLORINE DIOXIDE (ClO_2)

The use of chlorine dioxide for iron and manganese removal is rare in South Africa. Although various experimental trials have been conducted in recent years, only a few small-scale installations exist that use ClO_2 for its oxidising power (Juby, 1992). One major reason for this lack of wide-scale usage is that ClO_2 is expensive. Juby (1992) reports that it costs between 4 and 15 times more to produce a kilogram of ClO_2 than a kilogram of chlorine gas depending on the methods to generate the ClO_2 and the source of chlorine.

16.8.2.1 Iron removal

The oxidation of Fe (II) with ClO_2 is similar to that observed when using $KMnO_4$, except that the rates are slightly slower (Knocke *et al.*, 1991a). However, even at low temperatures (2°C), a pH of 5,5 and 1,25 times the stoichiometric dosage, more than 90% of any Fe (II) present can be expected to be oxidised within 5 seconds (Knocke *et al.*, 1991a). The theoretical stoichiometric requirement in organic free water is 0,24 mg ClO_2 / mg Fe (II). The diminishing effect of oxidation in the presence of organic acids (fulvic and humic) is very similar to that observed with $KMnO_4$.

It has been shown that during the oxidation of Fe (II) with ClO_2 a significant part of the ClO_2 is reduced to the chloride ion (Cl^-) (Knocke *et al.*, 1991a). This indicates that a transfer of five electrons occurs during the oxidation.

16.8.2.2 Manganese removal

The stoichiometric dosage of ClO_2 required for manganese oxidation is approximately 2.45 mg/mg Mn (II), assuming that the reduction byproduct is the chlorite ion (ClO_2) Knocke *et al.* (1987).

Later tests conducted by Knocke *et al.* (1991a) showed that chlorite was in fact the oxidant by-product (cf. the oxidation of Fe (II) with ClO_2), indicating a single electron transfer during the oxidation. This results in a dosage requirement for oxidation of Mn (II) which is nearly 5 times greater than that for Fe (II).

In low dissolved organic content waters the reaction time of Mn (II) oxidation decreases with an increase in pH (Knocke *et al.*, 1991a). However even at a pH of 5,5, the reaction should be over in less than 30 seconds. Nevertheless, Knocke *et al.* (1991a) observed that even though the oxidation rates were rapid they were slower than when $KMnO_4$ was used as the oxidant.

The presence of dissolved organics in the water reduces the rate of oxidation of Mn (II) and can result in incomplete oxidation (Knocke *et al.*, 1991a). However, the oxidation rate is still rapid (less than 2 minutes) for organic concentrations of less than 10 mg/ℓ. This is in contrast to the oxidation of Fe (II) in organic-laden waters, and is put down to a much lower degree of complexing by humic and fulvic acids with Mn (II).

In practice the dosage of ClO_2 required to oxidise Mn (II) is only slightly greater than the theoretical value (Knocke *et al.*, 1991a). Chapter 5 provides additional information on this subject.

16.8.3 OXIDATION WITH OZONE (O_3)

There are presently only two sizable ozone installations at water treatment plants in South Africa. One plant is at the Western Transvaal Regional Water Company where the ozone is dosed in a pre-ozonation step, one of the main reasons being for manganese oxidation. The other plant is at Umgeni Water's Wiggins water works which is presently being upgraded to a 90 kg/h installation. Ozone, because of its versatility, it is to be used at Wiggins for various purposes, and these include iron and manganese oxidation; although these are not the primary purposes.

Thus South Africa does have some experience in the use of ozone for iron and manganese removal. If the trends in the USA, Europe, and the East are followed there should be a greater use of ozone in South Africa in the future. Such plants will then have the facility to potentially remove iron and manganese via ozonation although in most cases this would probably not be the primary purpose. Some information, including some of the specific difficulties with iron and manganese removal using ozone, is presented below.

16.8.3.1 Iron removal

The use of ozone for oxidising Fe (II) is also a possibility (Knocke *et al.*, 1991a and Van Leeuwen, 1989). Ozone is classed as a strong oxidant along with $KMnO_4$ and Cl_2 (Applegate & Sackinger, 1987). Reckhow *et al.* (1991) conducted a laboratory study to investigate the use of ozone for oxidising Fe (II) and Mn (II) and found that the oxidation of Fe (II) into an insoluble form occurred rapidly in the absence of organic matter which, if present, protects the Fe (II) from the ozone by forming complexes. This confirms work carried out by others which demonstrated that ozone is ineffective in oxidising Fe (II) when it is complexed by certain samples of humic and fulvic acids (Knocke *et al.*, 1991a). Because all raw waters areassociated with some natural organic matter (NOM), the difficulties mentioned above should be noted and test work conducted in each case.

16.8.3.2 Manganese removal

Reckhow *et al.* (1991) also investigated the oxidation of Mn(II) with ozone at various pH values. They found that the oxidation of Mn (II) occurred in competition with the oxidation of the organic matter, hence, higher doses of ozone were required to achieve the same Mn (II) removal as was achieved in the absence of organic matter. They observed furthermore, that an increase in bicarbonate alkalinity did not accelerate the rate of Mn(II) oxidation in the absence of organic matter.

However, in the presence of organic matter, the increase in bicarbonate alkalinity resulted in a more complete oxidation of the manganese following the depletion of the dosed ozone.

Subsequent investigations by others (Wilczack *et al.*, 1993; McKnight *et al.*,1992) provide more practical insights into the use of ozone for Mn(II) removal. As in the case of iron removal, tests should be conducted to establish the effectiveness of ozone for the oxidation of manganese because it is very water-specific.

16.9 COLOUR REMOVAL

Naturally coloured water is aesthetically objectionable. The causes of colour in water are generally believed to be naturally occurring dissolved organic compounds, such as humic and fulvic substances, as well as iron and manganese. A brief discussion under the following headings is provided: coagulation, nanofiltration and ozonation, followed by biologically enhanced filtration.

Many natural waters contain humic and fulvic substances as part of natural organic matter which is present in the water, and these impart a brownish colour.

The same organic substances responsible for colour in water can also cause taste and odour problems, particularly after chlorination. They also have the potential to form disinfection by-products such as trihalomethanes (THMs), which are believed to be carcinogenic (Skukairy & Summers, 1992). Colour is reported in mg/ℓ Pt-Co according to the platinum-cobalt method, and 1 mg/ℓ Pt-Co is equivalent to 1 Hazen, the unit often used to report colour in South Africa. The SABS (1984) recommends a colour limit of 20 mg/ℓ Pt-Co in potable water.

Natural waters found in the southern Cape can have colour values from as low as 20 or 30 Hazen, up to 600 to 800 or more. In most instances the intensity of the colour varies with the time of year.

Colour removal is commonly carried out by coagulation with aluminium sulphate (alum) or an iron salt. Juby and Botha (1994) investigated alternative treatment methods for the removal of colour, using nanofiltration and ozonation followed by biologically enhanced filtration, and achieved colour values of less than 5 Hazen with nanofiltration, and removal efficiencies up to 95% with ozonation and biological filtration.

A brief account of the three different process trains is given below.

16.9.1 COAGULATION WITH ALUM

The conventional unit processes/operations used to remove colour are coagulation with alum ($Al_2(SO_4)_3 .6H_2O$) or an iron salt, followed by flocculation, sedimentation and filtration. These processes/operations often rely on relatively large dosages of coagulant to precipitate the humates and fulvates and/or to adsorb the colour compounds onto the metal hydroxide flocs.

The area requirements of these systems are extensive because of the generally poor settling characteristics of the flocs formed. Often polymers are added as well, in order to strengthen the flocs to improve the settling characteristics. Large volumes of sludge are produced from these plants, and generally because the sludge does not dewater easily, sludge disposal can be a problem.

Important final steps in the process would be disinfection and stabilisation, these being important when treating coloured waters as they have a low alkalinity and a low hardness.

A variation of this process is the use of dissolved air flotation, to separate the alum floc from the water. This has been used successfully both locally and overseas.

Despite what may seem like negative points about the conventional process, it generally produces excellent quality water, with Hazen values of less than 5 units - the detection limit of commonly used instruments. However, although the colour values are low, the literature reports that coagulation processes may not reduce the potential for THM formation to within acceptable limits. And, although South Africa does not presently have a recommended limit for THMS, such limits are being considered (Van Steenderen *et al.*, 1991).

Further information on coagulation processes is provided in Benefield *et al.* (1982), Weber (1972) and AWWA (1990).

16.9.2 NANOFILTRATION (NF)

As mentioned in Section 16.6, nanofiltration membranes are of a "looser" character and operate at lower system pressures than RO membranes (Table 16.4). These NF membranes not only have a high rejection of divalent ions (Ca^{2+}, Mg^{2+} - hardness), they can also remove colour associated with NOM without generating undesirable sludges which are prevalent in conventional colour removal processes (AWWA, 1990).

The humic and fulvic substances which impart the brownish colour to the water are fairly large complex organic molecules, and are therefore easily removed by membranes of the NF class.

A colour removal system involving NF membranes would include pretreatment (to remove the bulk of the suspended solids from the water - consisting possibly of screening, filtration and possibly pH adjustment), followed by the membrane filtration process itself.

An advantage of the NF process is that the membrane is able to reject THM precursors and the bacterial organisms in the water as well. This assists in producing a good quality water downstream of the disinfection and stabilisation steps. A disadvantage of the process is that the membranes are susceptible to fouling.

Juby & Botha (1994) report on pilot-scale NF work that was conducted with various NF membranes, and describe some of the advantages and disadvantages of this process. They were able to achieve excellent colour removal efficiencies, at product water recoveries of between 80 and 90%. Another consideration with this process would be the disposal of the high-coloured concentrate stream from the NF system - which theoretically could be discharged back into the natural water course.

Preliminary cost calculations by Juby & Botha (1994), show that the capital cost of a NF system (to treat about 4,5 Mℓ/d of water) would exceed that of a conventional alum coagulation system by about 40%. Furthermore, the operating costs of the conventional system would be expected to be slightly lower than those of the NF system. This would be greatly influenced by the degree of membrane fouling encountered and the expected membrane life.

16.9.3 OZONATION

Ozone is a powerful oxidant and disinfectant, with a thermodynamic oxidation potential that is the highest of all common oxidants (Glaze *et al.*, 1987). The ability of ozone to remove colour in water and effluents is well-known; linked to its potential to attach carbon-carbon double bonds and change the refractive properties of large organic molecules.

Even with the oxidation power of ozone, it is generally not possible to remove all of the brownish colour from natural waters. Therefore, unless the colour intensity is relatively low to begin with, it is not generally possible to reduce the colour value of the water to less than the recommended limit of 20 Hazen using ozone alone at economical dosages. This has been recognised both overseas (Tan & Amy, 1991) and locally (Juby & Botha, 1994).

The ability of ozone to fragment large organic molecules into smaller more biodegradable molecules was mentioned earlier in this chapter. Using this concept, Juby and Botha (1994), conducted pilot-plant work on a system using ozonation followed by biologically enhanced filtration for the removal of natural colour from water.

Conceptually, at full scale, this process would consist of coarse filtration to remove the bulk of the suspended material, ozonation (dosage dependent on the colour intensity of the water), filtration in biologically enhanced (BAC) filters (granular activated carbon for example), followed by stabilisation and the addition of a small amount of chlorine to provide a residual for the distribution system.

Colour removal efficiencies of up to 95% were achieved with the pilot plant (Juby & Botha, 1994; Juby, 1996), at ozone dosages significantly lower than when using ozone alone.

This process has the advantage of lowering the overall organic content of the water, thereby reducing the THM formation potential and the biological re-growth potential of the product water produced - the latter resulting in a reduced chlorine demand.

Capital cost estimates show that this process would cost about 12% less than a conventional colour removal process, producing 4,5 Mℓ/d (Juby & Botha, 1994), and would also have a slightly lower operating cost.

While this process looks promising as a competitive alternative to the conventional process, considerable work is still required in order to develop the process design parameters for full-scale systems. Such developmental work is being planned (Juby, 1996).

17

Mechanical and Electrical Plant and Equipment

A Bell

The extent to which mechanical and electrical plant and equipment are incorporated in a water works depends on a large number of factors, each of which must be carefully considered by the designer.

Such factors can include the size of city, town or community to be served, the source(s) and quality of water to be treated, the processes required, the degree of sophistication and automation most suitable for the size of works in conjunction with operation and maintenance skills available, stages of future extension, the topography of the works site in relation to the water supply source(s) and consumer location.

When establishing, refurbishing or rehabilitating a water scheme, the designer has various options from which to select mechanical and electrical equipment, such as different design options, product ranges, product quality and equivalent patents. This equipment and plant should, however, be selected to comply with the philosophies expressed in the design report on the complete works and its environment. Whether water to be purified should, for example, gravitate through the various process steps or whether it should be pumped, should be spelt out clearly.

The mechanical and electrical aspects are described here in sufficient detail for an understanding of the general philosophies and methods that are of interest to the designer, but lack detailed design information. In this regard certain aspects covered, such as pumps, blowers and compressors, power supply and reticulation, will require input from other chapters of this text, and from other sources.

17.1 PUMPS

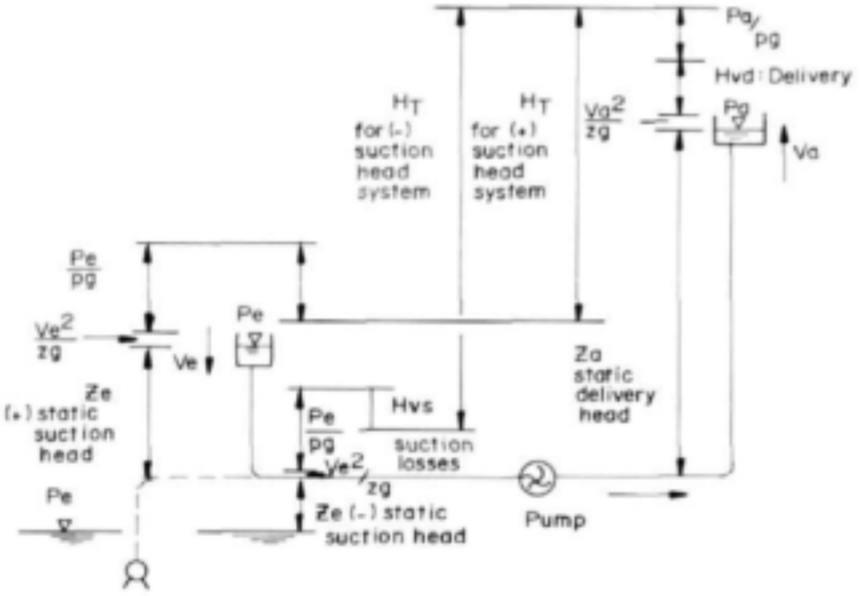
Pumps are widely used in the water industry to move liquids. The Witwatersrand is an outstanding example of a heavily populated area which is remote from its main source of supply where purified water is brought to the region from the Vaal River over a distance of more than 100 km and a height of up to some 450 m.

In purification works, pumps are primarily used to transfer water to and from works, chemical solutions in and around works, supply wash water to filters and recover filter wash water and supernatant from sludge drying beds.

For a better understanding of the transfer of liquids, its handling in terms of the definition heads and the power required will also be dealt with briefly.

17.1.1 LIQUIDS HANDLING

The liquids of importance in a water works plant are treated and untreated water, sludges, chemicals and solutions. Except for thicker sludges and relatively viscous solutions these liquids can, for purposes of selecting pumps for use in a plant, be regarded



Z and H is in m of fluid, P in Pa, V in m/s ρ in kg/m³ and g in m/s²

Figure 17.1 *Bernoulli equation*

as fluids with a viscosity equivalent to water.

Various types of pump are available on the market.

The energy transferred to the pumped medium in terms of unit weight under gravitational acceleration expressed in terms of units in length, is known as the ***head*** H. This total head (Ht), as derived from the Bernoulli equation for the pumping plant at design flow, can be expressed as follows:

$$H_T = \frac{Pa - Pe}{\rho g} + Z_a - Z_e + \frac{Va^2 - Ve^2}{2g} + H_{vs} + H_{vd} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \ 17.1$$

The headlosses in the intake and delivery lines can be calculated as described for closed conduits or pipes in Section 17.6.

The ***power*** input by a pump is defined as follows:

$$P = \frac{\rho g Q H}{1\,000 \times \eta}$$

$$P = \frac{\rho Q H}{102\,\eta} \text{ kW (for water)} \quad \ldots\ldots\ldots\ldots\ldots\ldots \ 17.2$$

Where Q is in m³/s, H in m, ρ in kg/m³ and η is the efficiency (for cold water ± 0,82 for a centrifugal pump).

Pump efficiency (η) is defined as the ratio of pump power (P_Q) (useful hydraulic power) to the power input (P) at the pump drive where the efficiency incorporates all the losses such as hydraulic, friction, leakage, mechanical power and others, i.e.

$$\eta = \frac{P_Q}{P} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \ 17.3$$

For pumping units, cavitation can occur in a pump should the pressure at any point fall below the pressure corresponding to the vapour pressure of the liquid at that temperature. This cavitation condition in a pump is determined by the NPSH value at the pump intake and is also dependent on various pump characteristics.

$$NPSH = \frac{P_e}{\rho g} + Z_e - H_{vs} - H_{va}$$

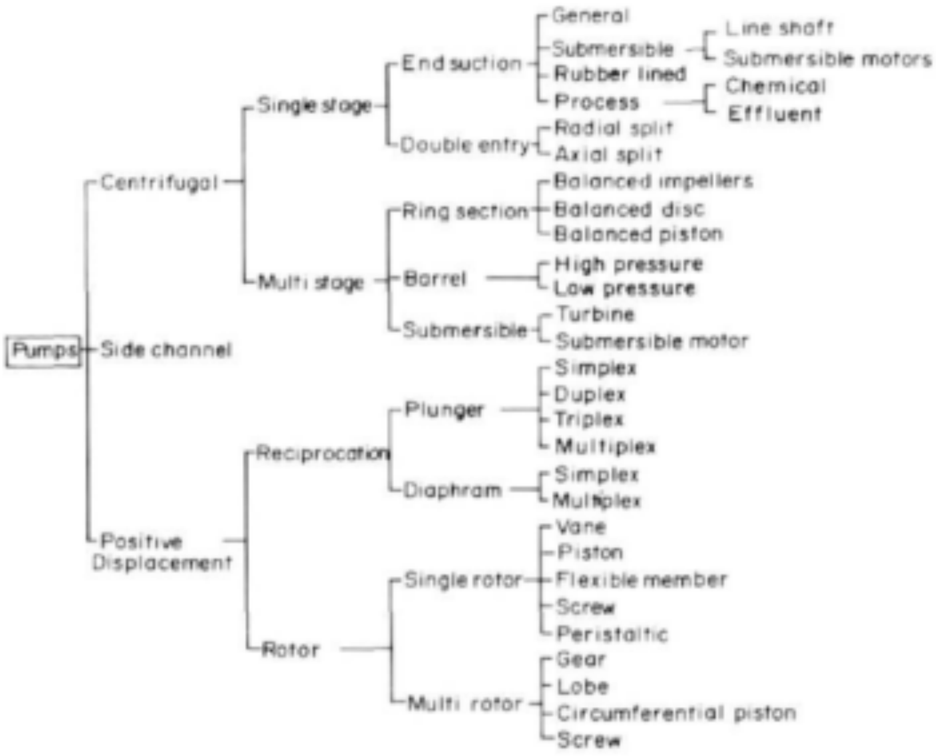


Figure 17.2
Types of pump

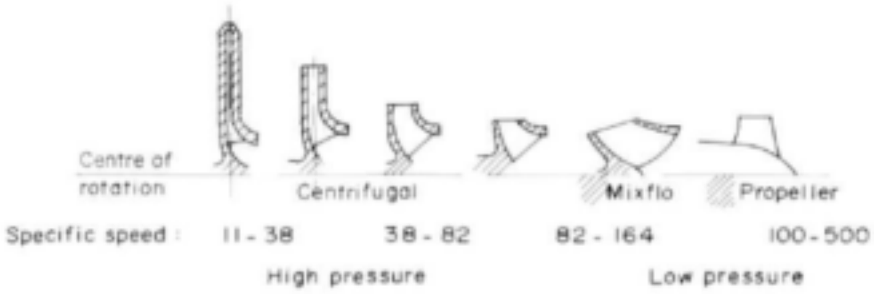


Figure 17.3
Relative impeller shape

P_e, Z_e and H_{vs} are as in Figure 17.1 and H_{va} is the vapour pressure at the liquid temperature.

17.1.2 CLASSIFICATION

Pumps can roughly be divided into three main categories, i.e. centrifugal, special types and positive displacement pumps, as are also depicted in Figure 17.2.

As positive displacement pumps in turn comprise reciprocating and rotary pumps, they will be dealt with as such.

Many of these pumps are successfully being used in the water treatment and supply fields.

Centrifugal pumps are generally divided into three categories relating to the impeller and the major direction of flow with reference to the axis of rotation. In a centrifugal pump the liquid is forced by atmospheric or other pressure into a set of rotating vanes which constitute the impeller discharging the liquid at a higher pressure and velocity at its periphery.

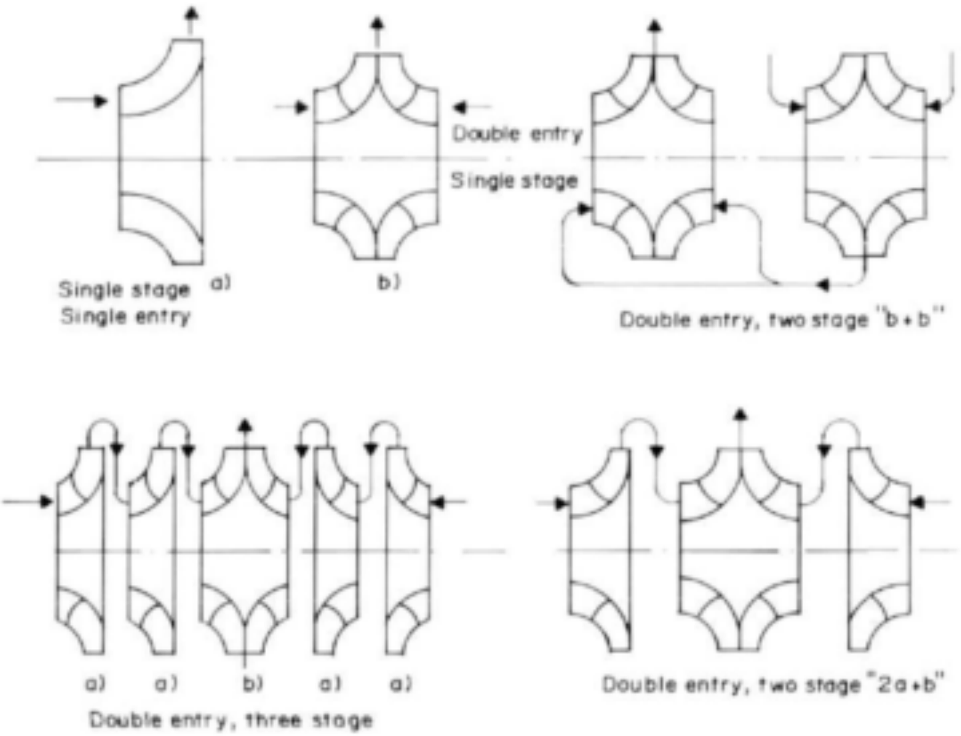


Figure 17.4
Pump impeller arrangements

Impellers can also be classified in accordance with the flow arrangement at the suction end, namely single and double suction, and in respect of their mechanical construction such as open, closed and semi-open. Should the required delivery head be developed by a single impeller it is a single-stage pump and two or more impellers are required in series for a multistage pump.

Special pumps such as air and jet pumps are generally not used in water works.

Reciprocating pumps comprise two types, namely the piston or plunger and the diaphragm or controlled volume pumps. The plunger pump at a constant speed essentially delivers the same capacity at any pressure within the capability of the driver and the strength of the pump. It has a high efficiency almost independent of pressure and capacity. The plunger can come into direct contact with the fluid.

The diaphragm type of the double disc diaphragm liquid end or eccentric cam drive design is used to accurately (within ± 1%) displace a predetermined volume of liquid in a specific period of time (Figure 17.5).

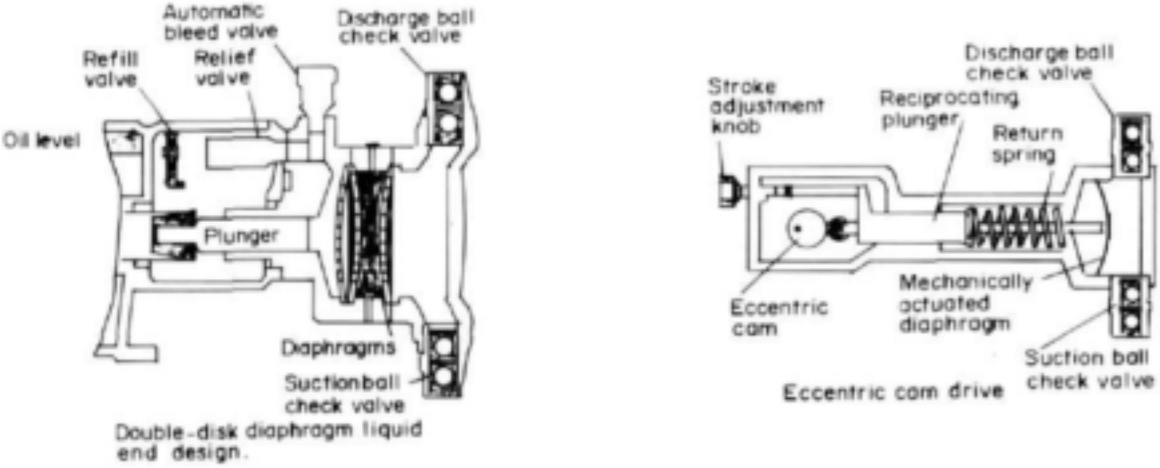
There are a variety of **rotary pumps** i.e. gear, progressive cavity and peristaltic pumps. Due to high accuracy and capacities ranging up to approximately 36 m^3/h the latter is ideally suited to the transfer of chemical solutions. It is also suitable for pumping sludges.

In progressive cavity or screw pumps (mono pumps) fluid is pumped axially between the meshing internal stator screw threads (Figure 17.6).

17.1.3 CHARACTERISTICS

When selecting a pump for a specific application and duty it is necessary to understand the characteristics of the different alternatives. For this purpose the characteristics of positive displacement and centrifugal pumps are dealt with here.

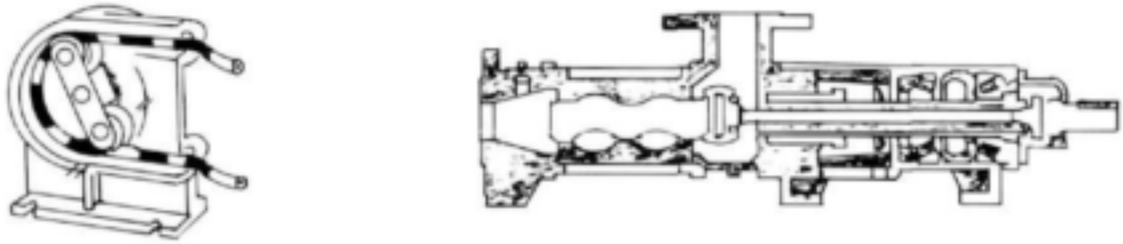
The main characteristics of **positive displacement pumps** are the provision of the high discharge pressures and of pumping viscous liquids. Most types are self-prim-



Double-disk diaphragm liquid end design.

Eccentric cam drive

Figure 17.5
Reciprocating and diaphragm pumps



Peristaltic pump

Screw (mono) pump

Figure 17.6
Peristaltic and screw pumps

ing. They are usually more expensive than centrifugal pumps, are more expensive to maintain and maximum flow rates tend to be lower than for equivalent centrifugal pumps.

Most positive displacement pumps have little slip at moderate pressure so that the volume pumped is rarely a function of discharge pressure. However, there is a tendency among some types to pulse, particularly at slower speeds. The volume pumped is controlled by adjusting the speed, or by bypassing back to the suction. The delivery of the pumps must never be restricted in any way. Because of the high pressure produced and the possible danger to both pump and pipelines, a pressure relief valve should be fitted.

Piston pumps are used to achieve the high pressures needed for mixing (up to 3 MPa), and high-pressure filtration. These pumps operate by alternating suction and discharge strokes. The discharge pulsates with time between a maximum and zero in the form of a sine curve. The pulsations are not usually desirable and can be smoothed by adding a pulsation damper such as an air cylinder, or by using a multi-cylinder pump.

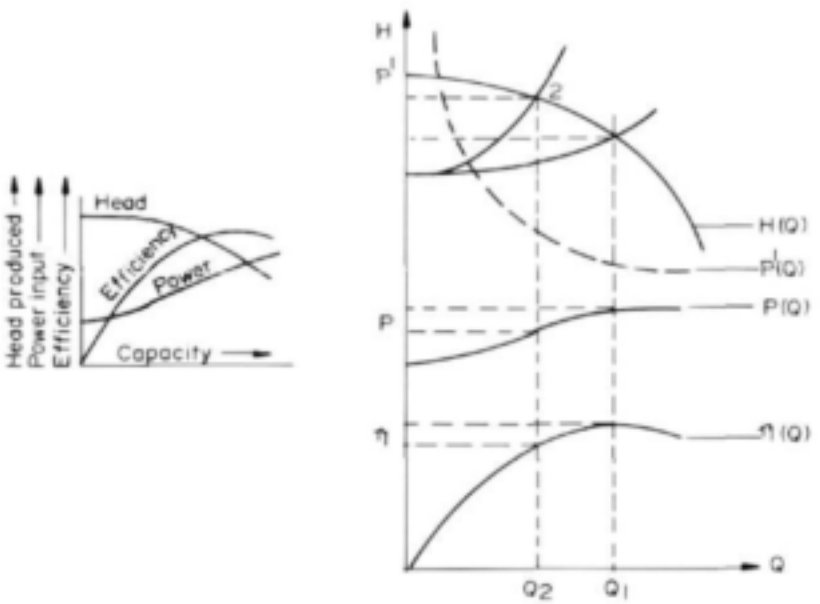


Figure 17.7
Typical characteristics curves at constant speed

The flow can sometimes be adjusted by adjusting the piston stroke or alternatively the speed may be controlled.

Diaphragm pumps operate on the same principle as piston pumps. A reciprocating diaphragm performs the same function as the piston and is self-priming. Delivery can be varied by varying either the stroke length or the frequency of the pulsations.

Compared with the piston pump, it has the advantage of being able to pump non-homogeneous products which have suspended particles.

Diaphragm pumps can be run dry, and the diaphragm is cheaply replaced. Volumetric efficiency does, however, fall off at higher pressures.

Overall efficiency is expressed as the ratio of water power output to mechanical power input P (normally multiplied by 100 to give a percentage efficiency). Mechanical power required at the pump is determined by the overall efficiency figure which may vary widely according to the type and size of pump and operating conditions. These overall efficiencies can be in the order of 90 to 95%.

Pump power P

$$= \frac{9{,}81 \times Q \times H_T}{\eta} \text{kW} \quad .. \; 17.4$$

where:

Q is the flow rate in m^3/s,
H_T the difference between delivery and inlet pressures in metres of water and
η the overall efficiency of the pump.

In both cases **pulsation dampeners** (cushion chambers) can effectively take up irregularities and induce a uniform flow in suction and discharge lines. The volume of an air dampener can vary depending on the type of pump, from 3 to 8 times the plunger displacement volume. For high-delivery pressures the air or gas should be prevented from dissolving into the water or pumped fluid by means of a diaphragm or bladder.

The performance of a **centrifugal pump** is generally described in terms of its characteristics, namely flow rate or capacity Q (m^3/s), the input power P (kW), the energy input into the fluid pumped H (m), the efficiency (η) and the speed of rotation N (r/min). As these are mutually interdependent, **characteristic curves** representing the performance, namely head-quantity, power-quantity and efficiency-quantity are generally used (Figure 17.7).

The relationship between these performance characteristics can be expressed by Equation 17.5 for a given capacity on the characteristic curve.

$$\eta = \frac{\gamma QH}{1000\,P} \quad .. 17.5$$

Optimum pump efficiencies can be related to **specific speed** (N_s) per pump stage and the pump capacity at the duty point where

$$N_s = \frac{N\sqrt{Q}}{H^{0.75}} \quad .. 17.6$$

where:

Q = flow rate per impeller inlet eye in m^3/s

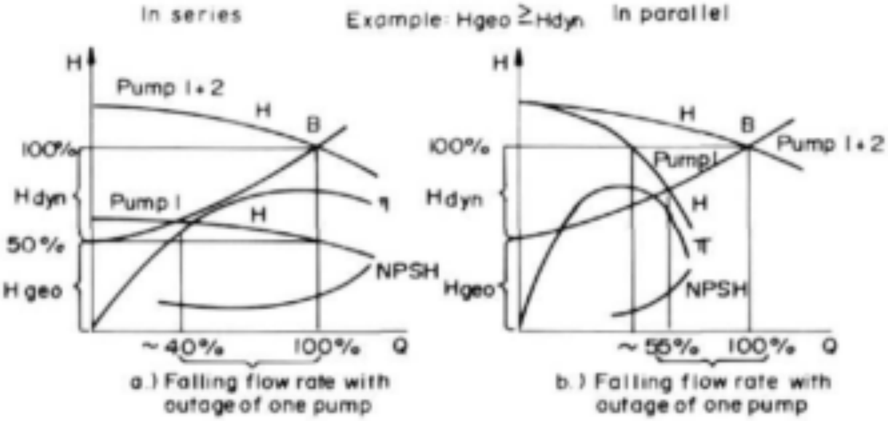
H = head per pump stage in m

N = pump speed in r/min

The specific speed is a characteristic number applicable to any type of pump irrespective of its physical size, throughput or speed. In Figure 17.5 the range of specific speed applicable to each type of impeller is indicated. Efficiencies increase for larger pumps of equivalent specific speed and from data it appears to be at an optimum for specific speeds in the range of 42 to 64.

The influence of N (r/min) on η, H and Q of a pump can be seen in Figure 17.8 (a) whilst the influence of two or more pumps being used in series and parallel operation are shown in Figures 17.8 (b) and 17.8 (c) respectively.

Due to a normally conservative design approach the friction head in a pipeline system is often less than the pump delivery at best efficiency point. Should it be necessary the impeller blade diameter can be reduced slightly where speed control is not applicable, to match actual conditions.



a.) Falling flow rate with outage of one pump

b.) Falling flow rate with outage of one pump

Figure 17.8
Influence of two or more pumps

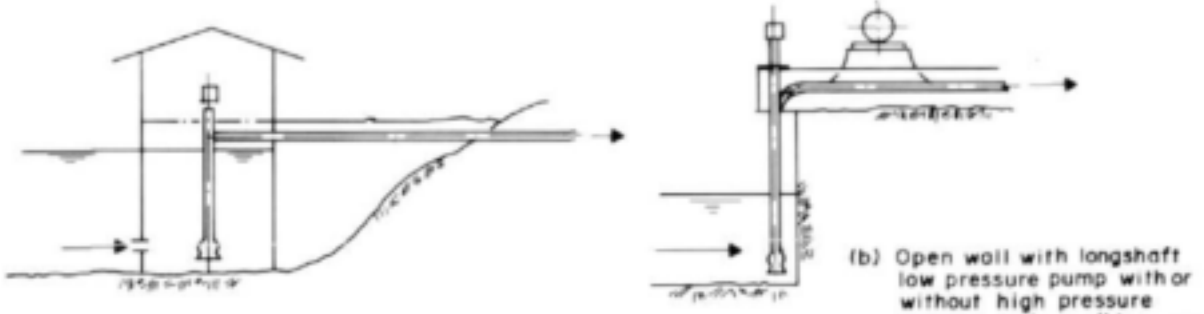
17.1.4 SELECTION AND APPLICATION OF PUMPS

Of the multitude of pumps available on the market certain types are more suitable than others for specific applications. In the purification works field the major applications are:

- transfer of water to and from works
- boreholes
- filter wash water and recovery
- transfer of chemical solutions and slurries
- reverse osmosis membranes

The cost of civil structures and the maintenance of pumps invariably have a deciding influence on the selection of pumps especially when transferring low grit content water from the intake point from a dam or river. Centrifugal pumps are invariably the most suitable for these applications. Installations are shown in Figures 17.9 and 17.10.

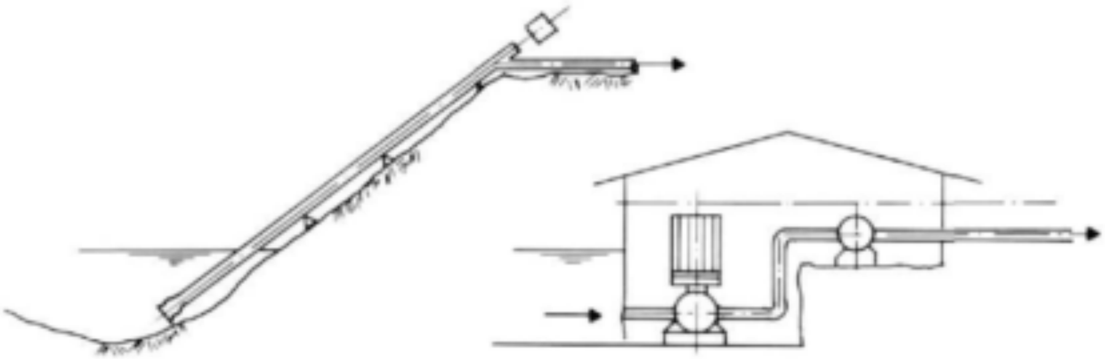
Large vertical or inclined long-drive shaft pumps normally require high maintenance. The drive shaft support bearings should preferably be oil lubricated when pumping untreated water. Horizontal drive spindle



(a) Long shaft low lift: delivery to approx. 60m

(b) Open wall with longshaft low pressure pump with or without high pressure pump or submersible pump for smaller works.

Figure 17.9
Variety of pump stations



(a) Inclined shaft irrigation intake or adapted for submersible pump for smaller works.

(b) Dry well horizontal spindle booster or main transfer pump

Figure 17.10
Pump station examples

pumps are reliable and generally require low maintenance.

Submersible, as well as vertical long-shaft single- or multi-stage centrifugal pumps are well suited to the abstraction of water from **boreholes**. The same applies to screw (mono) pumps for maximum delivery heads of approximately 160 m.

A horizontal drive spindle centrifugal pump in a dry well is most suitable for the supply of **wash water to filters** and is therefore primarily used for this application.

Relatively low head float operated pumps, i.e. small submersible and vertical spindle centrifugal pumps, are used for the recovery of filter wash water from holding tanks or supernatant from thickening tanks or sludge lagoons.

Whether liquid chemicals and or **chemical solutions** should be gravitated or pumped in and around the works, should be dealt with in accordance with the procedures set out in the design report. Suitably adapted pumps are available for different applications.

Coagulant or reagent solutions are often transferred by centrifugal pumps from a solution tank to a constant head feeder tank from which accurate dosing takes place. Should it be required to pump variable flow rates (*turn down ratio* 10:1) accurately (*repeatedly* ±1%), controlled volume diaphragm and peristaltic pumps can be used.

Lime **slurry solutions** should preferably be transferred in open ducts to facilitate easy cleaning. Should it however be imperative that they be pumped, screw (mono) and peristaltic pumps are suitable for this application. The delivery lines should be evenly graded, easy removable and equipped with cleaning eyes at all bends for cleaning. To prevent the settling out of solid particles, flow velocities must be kept high.

Screw (mono), peristaltic and diaphragm and plunger pumps (with modified stuffing boxes) are known to be suitable for the transfer of sludges and slurries up to the following respective solids content by weight, namely 2%, 2%, 10% and 65%.

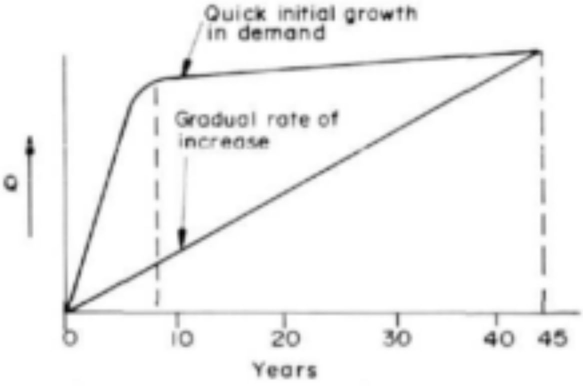
Small centrifugal pumps are ideally suited to transfer sampling water in a works and as chlorine boosters.

The plunger type pump with its high efficiency is most suitable to supply high-pressure feed water to **reverse osmosis membranes** in a desalination process. Although vibrations created by the plunger pulsations can partially be dampened, they remain a problem.

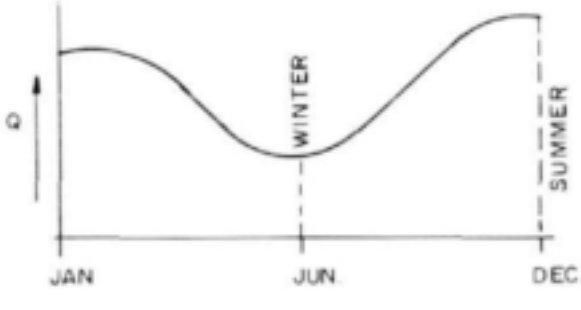
High-pressure centrifugal pumps, less efficient with flow control needle valves on the discharge end, as well as screw (mono) pumps for pressures up to approximately 160 m heads, are very reliable.

17.1.5 NUMBER OF PUMP-MOTOR UNITS

The number and capacity of pump-motor units required, and to a certain degree the type of unit depending on whether fixed or variable speed units may be more suitable (i.e. flexibility in delivery), will largely depend on factors such as the flow requirements for the specific application, the capacity of reliable pump units available on the market, the availability of replacement units, how soon after completion of a works the average maximum output will be reached (Figure 17.11(a)), future extension phases envisaged for the works, seasonal change in water demand (Figure 17.11(b)) and the level of risk of supply or non-supply of water or



a) Average growth in demand



b) Seasonal changes in demand

Figure 17.11
Temporal and seasonal demand

TABLE 17.1
TYPICAL FRICTION COEFFICIENTS

Gas	Pipe	Friction coefficient fanning factor *f*
Town gas	Mains (in poor condition)	0,005
Flue gas	Boiler flues	0,010
Compressed air	Pipes (clean)	0,0035
Compressed air	Pipes (average)	0,0040
Compressed air	Pipes (old)	0,006

fluid involved.

Water supplies to cities, power stations and other high risk consumers can be regarded as being of a high risk nature requiring stand-by pumps, motors or units.

The same applies to filter wash water pumps and other critical fluid supplies in a water works. For such high risk applications one for every three to four duty rating units, with not less than two units, for each duty rating is recommended as stand-by.

17.2 FANS, BLOWERS AND COMPRESSORS

These devices are used for ventilation, washing of sand filters and the provision of compressed air for the operation of valves etc. in purification works. They can be categorised as follows:

Low pressure devices (0 - 7 kPa)
- axial flow fans
- centrifugal fans

Medium pressure devices (5 - 200 kPa)
- sliding vane blowers
- rotary piston blowers (Rootes type)
- liquid ring type blowers

High pressure devices (+ 200 kPa)
- reciprocating compressors
- screw compressors

The handling of air and gas which covers transfer and the thermodynamic principles of gases that comprise the primary and secondary friction losses in pipeline systems will also be touched on.

17.2.1 HANDLING OF AIR AND GAS

The following thermodynamic laws apply to the so-called perfect gases:

Boyle's law which states that: $PV = \text{constant}$
Charles's law where: $V/_T = \text{constant}$
Characteristic gas equation : $PV = MRT$

where P is the absolute pressure of the gas (N/m^2), V the volume of the gas (m^3), T the temperature of the gas (degrees K), M the mass of the gas in kg and R the gas constant = 287 J/kg K for air.

In practical circumstances these laws give values that are sufficiently accurate.

The **friction loss** due to the flow of any gas in a straight pipe is given by the following equation:

$$h_f = \frac{f\,l\,v^2}{2\,g\,d} \qquad \text{.......... } 17.7$$

where:

h_f is the headloss (m),
L the length in (m), D the pipe diameter (m),

TABLE 17.2
ALLOWANCES FOR FITTINGS

Type of fitting	Number of pipe diameters (to be added to pipe length)
Bend (90 degrees) (long radius)	15-20
Elbow (90 degrees) (standard)	25-30
Tee (through)	15-20
Tee (side arm)	40-60
Valve (angle type) (fully open)	30-200
Valve (gate type) (fully open)	13-15
Valve (globe type) (fully open)	60-450
Valve (butterfly) (fully open)	20
Valve (swing check) (fully open)	135
Valve (ball check) (fully open)	150
Sharp edged inlet	0,50
Inward projecting pipe	1,00
Rounded inlet	0,05
Gradual reduction	0,05
Sudden reduction	0,28-0,45
Gradual enlargement	(0,14-1,10).K
Sudden enlargement	$K = (1-(d/D)^2))^2$
Exit (sharp-edged, rounded or projecting)	1,00

f a dimensionless friction loss coefficient known as the Fanning factor and
V the velocity (m/s).

Secondary losses for the flow of air around bends, reductions, enlargements, through valves, etc. are expressed in terms of equivalent pipe diameter lengths. These equivalent pipe-diameter-lengths are to be added to the length of straight piping for the calculation of the total loss of head (m) or pressure.

Flow of air in a pipeline values of f are given in Table 17.1. Equivalent pipe diameter lengths, for various fittings, are given in Table 17.2.

17.2.2 LOW-PRESSURE DEVICES

Fans, i.e. low-pressure devices, are generally used for the ventilation of chemical or other buildings in a purification works, or air-conditioning in offices or the forced ventilation of chlorine rooms.

Axial flow, also known as propeller fans, in use today are of two principal types:

- those with blades which are stamped out of sheet metal; and
- those with blades which have thickness and are shaped like the propellers of an aeroplane, being cast of light weight metal.

Fans of the first type are like the common air fan seen in offices and stores. Industrial exhaust fans of this type have the same type of balding, differing principally in the ruggedness of construction and in the mounting of blades and motor. The blades are usually fashioned of sheet metal curved helically in the manner of a screw. They are designed to exhaust against resistances close to zero. Their capacity is materially reduced when required to operate against static pressures of as little as 2,5 to 5,0 mm of water.

Fans with propeller shaped blades can usually operate at resistances of up to 25 to 50 mm of water. Operation at these higher resistances is accompanied, however, at cost of a materially higher noise level.

Where the system resistance is low, propeller fans are by far the lowest in cost for a given exhaust capacity of any in the fan family. The limitation of propeller fans in their capacity to move air through duct systems is evident from the pressure limits cited. Both types are commonly seen in use for exhausting air through short lengths of duct serving stores and the like. Low air velocities and short duct length make this type of application feasible.

While there is a considerable choice in speed of the direct connected motor employed for driving fan blades available to the manufacturer, they are generally assembled for capacities most commonly in the range of 6,1 to 7,6 $m^3/s.m^2$ of circular area within the blade ring - roughly, the circular area corresponding to blade radius. They may sometimes range up to 10 $m^3/s.m$.

These figures provide a convenient basis for rough estimation of the free delivery capacity of a propeller fan in the absence of manufacturers rating data. Thus a 900 mm diameter propeller fan, the circular area of which is about 0,64 m^2, could reasonably be assumed to have a capacity within a range of 3,2 to 6,4 m^3/s, and most likely around 3,9 to 4,9 m^3/s.

The propeller type can serve duct systems of greater lengths where larger diameter ducts are used to maintain low frictional resistance. It finds application in the mining industry.

Centrifugal fans can be characterised into three types in relation to the impeller blade tips which are either forwardly curved, radial or backwardly curved in respect of rotation.

Forwardly curved blade tips provide the maximum head for a given impeller size and speed but this type of fan is less efficient and the pressure at duty point is usually lower than at the no-flow point, which can cause unstable conditions if fans are arranged to run in parallel. Because of its lower speeds this type of fan finds application in the ventilation of buildings where noise levels are important. Efficiencies range between 50 and 60% and static pressures are usually of the order of 150 mm.

Radially tipped blades have greater strength against centrifugal force and are often used in higher pressure applications and in dust extraction systems.

Backwardly curved blades are the most efficient but impellers have to be larger in diameter and may have to run at higher speeds to obtain the same duty point as forwardly curved blade impellers. It is often used in high pressure multistage fan blowers running at high speed where large quantities of air are required and energy costs are important. Gases handled by this type of fan should be clean to avoid damage to the impellers.

17.2.3 MEDIUM-PRESSURE DEVICES

Rotary compressors, blowers, and vacuum pumps are machines of the positive-displacement type. Such units are essentially constant-volume machines with variable discharge pressure. Volume can be varied only by changing the speed or by bypassing or wasting some of the capacity of the machine. The discharge pressure will vary with the resistance on the discharge side of the system. Rotary compressors are generally classified as sliding-vane type, straight or rotary piston or rootes type, and liquid-piston type.

The **sliding or rotary-vane blower** is usually used for capacities of up to 0,10 m^3/s. Generally pressure ratios per stage are limited to 4 or 5 to 1.

Rotary-vane compressors may be dry, lubricated, or oil-flooded. Prior to 1960 two-stage oil-flooded vane compressors of up to 0,45 m^3/s capacities compressing to 1000 kPa, were used in portable compressors.

They are now usually limited to less then 0,1 m^3/s for air compressors and to small refrigeration compressors and boosters.

The normal **straight lobe or rotary piston or rootes type** of unit is available for pressure differentials of up to about 9 kPa and capacities up to 7 m^3/s.

Modern lobe compressors with special profile lobes are used to supply oil-free compressed air. This design can supply 860 kPa from 0,1 to 0,7 m^3/s.

The **liquid-ring types** are offered as single-stage units for pressure differentials of up to about 520 kPa in the smaller sizes, and capacities of up to 1,88 m^3/s, when used with a few kPa pressure differential.

Staging is used for higher pressure differentials.

17.2.4 HIGH-PRESSURE DEVICES

The primary use of compressed air provided by high-pressure devices or compressors in a purification works, is for the remote control of valves and the provision of air in the dissolved air flotation process.

Reciprocating compressors are the type most widely used in the chemical industry. They are furnished for steam-engine, electric-motor, and gas- or diesel-engine drive; and, in a few cases, for turbine drive through reduction gears.

Reciprocating compressors are furnished either single-stage or multi-stage. The number of stages is determined by the compression ratio P_2/P_1.

The compression ratio per stage is generally limited to 4, although small-sized units are furnished with a compression ratio as high as 8 and even higher.

Single-stage air-cooled compressors are available in sizes of up to about 75 kW. Such units are available in one, two, three or four stages for pressures as high as 24 000 kPa.

The **screw-type oil flooded rotary compressor** is capable of handling capacities of up to about 1,2 m^3/s at pressure ratios of 7 or 8 to 1. The single-stage twin-screw compressor is used in the refrigeration industry where the low bearing load on the main rotor is an advantage. It is also being used as an air compressor in the smaller sizes.

For dry rotary screw type compressors the maximum compression ratio per stage is 4,5:1. Their capacities range from 0,2 to 9,4 m^3/s at pressures from 100 to 1 240 kPa.

17.2.5 SELECTION

Typical devices used for different applications in water treatment plants are tabulated in Table 17.3.

TABLE 17.3
TYPICAL APPLICATION OF DEVICES

Component requirement	Typical device
1. Ventilation in chemical building	- Roof ventilators with or without fan
2. Ventilation in chlorine store	- Axial flow fans
3. Office, control room, laboratory if required	- Air conditioner if regarded as necessary
4. Air scouring of filters	- Rootes type blowers
5. Air for operation of control valves	- Reciprocating compressors

17.3 MOTIVE POWER AND ELECTRICITY

The Eskom electricity supply network presently provides well in excess of 90% of the electricity consumed in the RSA. There are still however remote communities that are being served with electricity generated by diesel engine driven generators. Should smaller water works have to be served where electricity from either of these sources is not economically viable, it may be optional whether electricity needs to be generated for the works or whether petrol or diesel engine driven pumps will suffice.

When new plants are designed engineers should take care that, where feasible, the best energy conservation practices are followed, such as avoiding pumping where gravity can be used.

With electricity being the most commonly used motive power source, it is warranted that electric drive units (i.e. motors) as well as electricity supply and reticulation and lighting, and lighting, be dealt with in this section.

17.3.1 MOTIVE POWER

The bulk of the mechanical devices used in the purification works in the RSA today is electrically driven although diesel, petrol and gas engines may still be found as pump drives in remote areas. It should, however, be mentioned that minor items of equipment can very well be pneumatically or hydraulically driven.

Diesel engines, although more expensive than petrol units, consume less fuel, are more reliable and require less maintenance. They operate at 250 to 1000 r/min with power ranging from five to several thousand kW and can be built to be converted to gas or gas and oil. They are often used to generate power in remote areas.

Petrol or gas engines (4 to 8 cylinders), normally equipped with an electric starter, are used for pumping at 900 to 1 800 r/min. They develop 450 to 600 kW. The maximum efficiency of fuel use lies within a narrow range although the power output range is wide.

Petrol, gas and diesel engines are also used to generate standby emergency power or as standby prime mover. The switch-over can be effected within minutes. It must, however, be stressed that petrol or gas engines are very rarely used in the RSA.

Electricity generated by diesel engine is generally supplied at the accepted three-phase 380/400 volts norm.

17.3.2 DRIVE UNITS

Although pumps are still being driven by gas and steam turbines and diesel engines, three phase constant and variable speed electric motors are primarily used in the water industry as pump drives at present. Variable speed units are not cost-effective and it is recommended that they be used only whenever the head-duty (i.e. balance between lead and flow) cannot be met by fixed speed units.

Electric motors generally comprise three types, namely the most robust and commonly used asynchronous squirrel cage induction motor, and the less frequently used asynchronous slip-ring and synchronous motors. The latter, although more costly, have a power factor of 1,0, thus giving maximum electrical efficiency for the least electrical cost. Both asynchronous types of motor have a power factor of approximately 0,9. This can however be increased by means of capacitor banks to approximately 0,95.

The slip-ring motor, for which purchase and running costs are higher, is mainly used

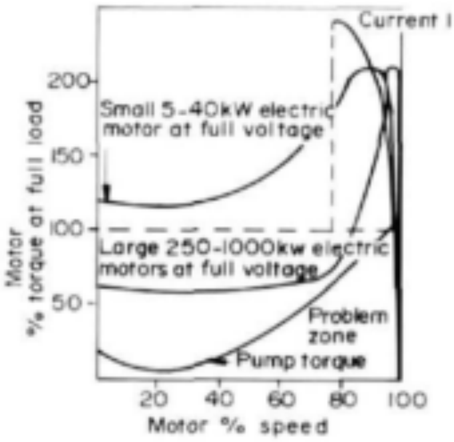


Figure 17.12
Starting torque curve for electric motor and pump

in applications where a low starting current with high starting torque is required. Installations still exist where it is used for variable speed applications.

Induction motor with electronic variable speed drives are presently being used for variable speed applications.

With respect to pump drives the starting method is an important consideration in the electrical-mechanical design process. The proven starting methods for three phase motors are direct connection (which includes star-delta starting), starting via choke and starting via transformer only. Valveing at pumps is dealt with in Section 17.8.

As depicted in Figure 17.12, the centrifugal pump starting torque is so low that it does not require special consideration. With regard to larger pumpsets, however, care should be taken that the motor starting torque exceeds the pump starting torque by a reasonable margin over the full range of the run-up speed, in order to ensure adequate acceleration of the pumpset. If this requirement is not met there is a danger that the motor windings will burn out.

It is imperative that the degree of automation designed into a water scheme and works be spelt out clearly for each component because of the influence it may have on the selection of mechanical and electrical items of equipment and plant. Due consideration must also be given to the location of controls envisaged. Instrumentation and process control are dealt with in Chapter 18.

17.3.3 ELECTRICITY SUPPLY AND RETICULATION

In certain regions of the RSA electricity is either generated on a small scale or distributed to consumers by local authorities who then collect the revenue and maintain the distribution lines in those regions. Negotiations with a power supply authority for requirements in excess of 1 MVA should start well in advance of the design process. A period of approximately 24 months is recommended, which also applies for remote areas where there are no transmission lines or where the local grid supply is inadequate. Network supplies have been found to be distributed at the following voltages: 132, 88, 66, 33, 22, 11, 6,6 and 3,3 kV. Due consideration must also be given to preferential off-peak power tariffs wherever available.

Supply authorities generally step down the supply voltage down (with their own transformers) to 3,3 kV but Eskom has, in exceptional cases, however provided electricity for large pumping stations and installations at as much as 3,3 or 6,6 or 11 kV to match motors. Higher voltages may be more economical for large installations.

Low voltage power at 380/400 volts is normally adequate for electric motors up to 300 kW which would be applicable to most water treatment plants.

The positioning of distribution panels, control consoles and the routing and sizing of cables and cable ducts must be properly designed and pre-planned. Considering the fact that a water treatment plant (also a pumping station) is primarily a works building, all low voltage distribution and lighting cables can be surface mounted except probably in the office, laboratory and control rooms. Surface mounted cable trays are very appealing if neatly mounted and routed.

The Machinery and Occupational Safety Act (MOSACT 1993) implies that lockable high tension switch rooms be provided as only qualified electricians are allowed in them. Due to the danger of a fire occurring in a high tension switch room, personnel escape routes must be studied carefully and suitably placed emergency escape doors must be provided if necessary.

17.3.4 LIGHTING AND LIGHTNING

Industrial **lighting** design standards for offices, ablutions, laboratories and switch rooms should be acceptable. SABS 0114 PART I is a Code of Practice for interior lighting standards. A light intensity of approximately 160 Lux at floor level is an acceptable norm for area lighting, stair- and passageways and loading bays. High-bay high pressure sodium lights with a high efficiency and long life (approximately 16 000 h), that do not attract insects, can be recommended for applications such as pumping stations and chemical buildings. Mercury and fluorescent tubes are also efficient but unfortunately attract insects. As the above-mentioned general area lighting is too low for work surfaces, additional wall mounted spot lights should be provided to increase this to a norm of approximately 450 Lux. Tungsten halogen instantaneously bright lights are suitable for this purpose.

External site lighting must be designed to suit the specific security and other requirements. Instantaneously bright lights are suitable for such purposes except that they have a relatively short life (approximately 1000 h).

It is essential that works buildings such as pumping stations, chemical buildings and others be protected against lightning strikes. SABS specification 03/1985 "The Protection of Structures Against Lightning" covers the design of such systems with "PONTINS Lightning Protection Catalogue" covering the installation thereof.

The lightning protection system must be connected to the earthmat required for the electrical system for the works. A resistance of approximately 4 ohms can be regarded as a fair norm for an earthmat which functions best in wet or damp ground. Should this be difficult to achieve the surrounding area can be kept wet and or paved as the latter tends to keep the subsurface damp. Mechanically strong 75 mm^2 copper conductors laid in straight lines with very gentle curves have been found to be suitable for this application.

17.4 FLOW IN PIPES AND CANALS

The fluids in a purification works, primarily water and chemical solutions, basically pass through different pipe and canal components. The hydraulic design of the different components is important and should ensure that water either does not overspill at any undetermined locality or that the components are not over-designed and unnecessarily costly.

It is necessary that a flow diagram be drawn up for the main stream flow through the works. For this purpose the theory of the flow in pipes and canals is only dealt with briefly in this text.

17.4.1 PIPES

The headloss encountered when water flows through a pipe system comprises the primary as well as the secondary losses such as losses through specials and valves, such as reducers, around bends, flow meters and others.

The **primary headloss** is in essence the friction loss in the straight pipe lengths. The flow in a tube can either be steady or unsteady, uniform or non-uniform and laminar or turbulent. Should the flow velocity at any point remain constant the flow is steady or there is a fixed rate of discharge past any section. For uniform flow conditions the velocity characteristics remain the same for successive sections. The motion in a circular tube is laminar when there is no mixing between neighbouring cylindrical laminae, and turbulent when a sufficiently large disturbance is present in the flow. Thus when the viscous action damps out turbulent flow, the flow is laminar and, above a critical Reynolds number R_e, the flow is turbulent for incompressible fluids

$$R_e = \frac{vD\rho}{\mu} = \frac{vD}{v}\;\frac{\text{Inertia forces}}{\text{Viscous forces}} \quad \text{.... } 17.8$$

where V is the average velocity (m/s) across the section, D the pipe diameter in (mm), ρ the specific weight, μ the viscosity of the fluid and v the relative or kinematic viscosity.

Laminar flow occurs when $R_e < 2\,000$ and turbulent flow when $R_e > 4\,000$ with a transitional zone in between.

For turbulent flow, Balsius, Nikuradse and others concluded that

$$\frac{1}{\sqrt{\lambda}} = 2 \log \frac{3{,}7D}{K} \quad \text{(smooth pipes) ..... } 17.9$$

and

$$\frac{1}{\sqrt{\lambda}} = 2 \log \frac{D}{K} + 1{,}14 \quad \text{(rough pipes). } 17.10$$

where K is the surface roughness of the pipe (mm).

The Colebrook-White formulae (for smooth and rough pipes respectively) are:

$$\frac{1}{\sqrt{\lambda}} = -2 \log \left(\frac{K}{3{,}7D} + \frac{2{,}51}{R_e\sqrt{\lambda}} \right) \text{ .... } 17.11$$

and

$$\frac{1}{\sqrt{\lambda}} = 1{,}14 - 2 \log \left(\frac{K}{D} + \frac{9{,}35}{R_e\sqrt{\lambda}} \right) \text{. } 17.12$$

$$V = -2\sqrt{2g}\,Di \,\log \left(\frac{K}{3{,}7D} + \frac{2{,}51\,v}{D\sqrt{2g}\,Di} \right) \text{. } 7.13$$

(*i* being h/L)

These formulae were derived, after intensive study at the Hydraulics Research Station in Wallingford, Oxford. Tables were produced for different surface roughnesses K where the headloss (m/km) can be read off against a flow Q (ℓ/s) and pipe diameter (mm).

For use in Equations (17.11) and (17.12) above, Moodie expressed

$$\lambda = 0{,}0055 \left[1 + \left\{ 2000 \frac{K}{D} + \frac{10^6}{R_e} \right\}^{1/2} \right] \text{. } 17.14$$

Table 17.4 gives K-values as taken from Ackers (1924).

Other simplified empirical formulae were derived for rough pipes such as "Hazen-Williams"

$$i = \frac{K_1 \dfrac{V}{(C_h)^{1{,}85}}}{D^{1{,}167}} \quad \text{............................... } 17.15$$

Table 17.4
Recommended K-Values (mm)

Classification (assumed clean and new unless otherwise stated)	**Suitable design chart values of k mm**		
	Good	**Normal**	**Poor**
Smooth Drawn non-ferrous pipes of aluminium, brass, copper lead, etc. and non metallic pipes of Alkathene, glass, Perspex, plastic, fibreglass etc.	-	0,003	-
Asbestos cement	-	0,015	-
Spun bitumen lined	-	0,03	-
Spun concrete lined	-	0,03	-
Wrought iron 0,03	0,06	0,15	
Rusty wrought iron	0,15	0,60	3,0
Uncoated steel	0,015	0,03	0,06
Coated steel 0,03	0,06	0,15	
Galvanised iron	0,06	0,15	0,30
Coated cast-iron	0,06	0,15	0,20
Uncoated cast-iron	0,15	0,30	0,60
Tate relined pipes	0,16	0,30	0,60
Old tuberculated water mains, with the following degrees of attack:			
Slight	0,60	1,5	3,0
Moderate	1,5	3,0	6,0
Appreciable 6,0	15	30	
Severe	15	30	60
(Good: up to 20 years' use normal: 40-50 years' use poor: 80-100 years' use			
Concrete			
Class 4. Monolithic construction against oiled steel forms, with no surface irregularities, smooth-surfaced precast pipelines with no shoulders or depressions at the joint	0,06	0,15	-
Class 4(a) Smooth-surfaced precast pipe lined in units of 1,8 m or over, with spigot and socket joints, orogee joints pointed internally	-	0,15	0,30
Class 3. Monolithic connection against steel forms, wet-mix or spun precast pipes, or with cement or asphalt coating	0,30	0,60	1,5
Class 2. Monolithic construction against rough forms, rough texture precast pipes or cement gun surface (for very coarse texture, take k = size of aggregate in evidence)	0,60	1,5	-
Class 1. Precast pipes with mortar squeeze at joints	-	3,0	6,0
Smooth trowelled surfaces	0,30	0,60	1,5

TABLE 17.5
VALUES OF HAZEN-WILLIAMS COEFFICIENTS C_H

Type of pipe	**Values of C_H for pipes of diameter**					
	25 mm	**80 mm**	**152 mm**	**305 mm**	**610 mm**	**1220 mm**
Uncoated cast iron: smooth and new		121	125	130	132	134
Coated cast iron: smooth and new:		129	133	138	140	141
30 years old:						
Trend 1: slight attack		100	106	112	117	120
Trend 2: moderate attack		83	90	97	102	107
Trend 3: appreciable attack		59	70	78	83	89
Trend 4: severe attack		41	50	58	66	73
60 years old:						
Trend 1: slight attack		90	97	102	107	112
Trend 2: moderate attack		69	79	85	92	96
Trend 3: appreciable attack		49	58	66	72	78
Trend 4: severe attack		30	39	48	56	62
100 years old:						
Trend 1: slight attack		81	89	95	100	104
Trend 2: moderate attack		61	70	78	83	89
Trend 3: appreciable attack		40	49	57	64	71
Trend 4: severe attack		21	30	39	46	54
Miscellaneous:						
Newly scraped mains		109	116	121	125	127
Newly brushed mains		97	104	108	112	115
Coated spunt iron: smooth and new		137	142	145	146	148
Old:- Take as coated cast iron of same age						
Galvanised iron: smooth and new	120	129	133			
Wrought iron: smooth and new	129	137	142			
Coated steel: smooth and new	129	137	142	145	148	148
Uncoated steel: smooth and new	134	142	145	147	150	150
Coated asbestos cement: clean		147	149	150	152	
Uncoated asbestos cement: clean		142	145	147	150	
Spun cement lined and spun						
Bitumen lined: clean		147	149	150	152	153
Smooth pipe (inc. lead, brass, copper, polythene, smooth PVC, etc): clean	140	147	149	150	152	153
PVC (Wavy): clean	134	142	145	147	150	150
Concrete:- (Scoby)						
Class 1 ($C_s = 0,27$) clean		69	79	84	90	95
Class 2 ($C_s = 0,31$) clean		95	102	106	110	113
Concrete						
Class 3 ($C_s = 0,345$) clean		109	116	121	125	127
Class 4 ($C_s = 0,37$) clean		121	125	130	132	134
Best ($C_s = 0,40$) clean		129	133	138	140	141
Tate relined pipes: clean		109	116	121	125	127
Prestresses concrete pipes: clean			147	150	150	
* **Note**: The above values for C_H are approximately correct at a velocity of 0,9 m/s. For other velocities, approximate corrections to C_H can be made using the following table.						

Values of C_H at 0,9 m/s	Velocities below 0,9 m/s values or for each halving, re-halving etc. of velocity relative to 0,9 m/s	Velocities above 0,9 m/s for each doubling, redoubling etc. of velocity relative to 0,9 m/sec
Below 100	add 5% to C_H	Subtract 5% from C_H
100 - 130	add 3% to C_H	Subtract 3% from C_H
130 - 140	add 1% to C_H	Subtract 1% from C_H
Above 140	Subtract 1% from C_H	add 1% to C_H

or $v = 0{,}354\, C_h\, D^{0{,}63}\, i^{0{,}54}$ 17.16

with:

$K_1 = 6{,}84$ in S.I. units and
C_h being the Hazen-Williams constant.

The C_h values, however, vary quite considerably with flow velocity and diameter, for which correction factors thus have to be applied. See Table 17.5 for Hazen-Williams C_h values and C_h correction factors.

Secondary losses can either be expressed in terms of an additional equivalent length of straight piping for each item to the headloss (h_s) or in terms of a $h_s = \frac{KV^2}{2g}$ where

K is a loss coefficient for the item involved. Typical K and equivalent length (L/D) values are given in Table 17.6.

TABLE 17.6
SECONDARY LOSSES

Type of fitting		K	l/d approx
Entry losses			
Sharp-edged entrance		0,50	22
Re-entrant entrance		0,80	36
Slightly-rounded		0,25	11
Bellmouthed entrance		0,05	2
Foorvalve & Strainer		2,5	113
Intermediate losses			
Elbows (R/D = ½ approx)	22,5°	0,20	9
	45°	0,40	18
	90°	1,00	45
Close radius bends (R/D = 1 approx)	22,5°	0,15	7
	45°	0,30	14
	90°	0,75	34
Long radius bends (R/D = 2 to 7)	22,5°	0,10	5
	45°	0,20	9
	90°	0,40	18
Sweeps (R/D = 8 to 50)	22,5°	0,05	2
	45°	0,10	5
	90°	0,20	9
Mitre elbows			
22,5° - 2 piece		0,15	7
30° - 2 piece		0,20	9
45° - 2 or 3 piece		0,30	14
60° - 2 piece		0,65	28
3 piece		0,25	11
90° - 2 piece		1,25	56
- 3 piece		0,50	22
- 4 piece		0,30	14
Tees			
Flow in line		0,35	16
Line to branch or branch to line :			
sharp-edged		1,20	54
radiused		0,80	36
Angle Branches			
Flow in line		0,35	16
Intermediate losses (continued)*			
Line to branch of branch to line:			
30° angle		0,40	18
34° angle		0,60	27
90° angle		0,80	36
Sudden enlargement*			
Inlet dia. outlet dia.			
4 : 5		0,15	7
3 : 4		0,20	9
2 : 3		0,35	16
1 : 2		0,60	27
1 : 3		0,80	36
1 L 5 and over		1,00	45

17.4.2 TYPES OF PIPE

Different pipe materials are used to transfer fluids at water treatment plants. The most commonly used materials have been listed together with manufacturing specifications and applications in Table 17.9.

17.4.3 Canals

Flow in an open canal or channel can either be steady and uniform or non-uniform, in which case the flow can be rapidly varied or gradually varied.

For uniform laminar and turbulent flows in a wide canal the average velocity can be expressed as:

TABLE 17.6 (CONTINUED)

Type of fitting	K	ℓ/d approx
Sudden contractions*		
Inlet dia. Outlet dia.		
5 : 4	0,15	7
4 : 3	0,20	9
3 : 2	0,30	14
2 : 1	0,35	16
3 : 1	0,45	20
5 : 1 and over	0,50	22
Tapers*		
Flow to small end	neglible	
Flow to large end		
Inlet dia. 4 : 5	0,30	1,5
to 3 : 4	0,04	2,0
Outlet dia. 1 : 2	0,12	6,0
Valves		
Gate valve - fully open	0,12	5
Gate valve - 1/4 closed	1,00	45
Gate valve - ½ closed	6,00	270
Gate valve - 3/4 closed	24,00	1080
Globe valve	10,00	450
Right angle	5,00	225
Reflux valve	1,00	45
Butterfly valve	0,30	13
Exit losses		
Sudden enlargement	1,00	45
Bellmouthed outlet	0,20	9

$$K = \frac{\text{Head Loss}}{V^2/2g} = \frac{\text{Head Loss}}{\text{Velocity Head}}$$

$$nD = \ell/_D = \frac{\text{Length of straight pipe to give equivalent loss of head}}{\text{Pipe diameter}}$$

*Figure for enlargement, contractions and tapers apply to smaller diameter

+I.W.E. *Manual of British Water Engineering Practice*, Vol II, W. Hefer & Sons Ltd., Cambridge

$$V = \frac{q}{D} = \frac{\rho g D^2 \sin\vartheta}{3\mu} \quad \text{.......... } 17.17$$

where q is the flow rate, D the average depth of flow and ϑ the slope of the canal.

By introducing the Reynolds number into the equation and $\sin\vartheta = \frac{3V^2}{Rg\,D}$

$$i = \frac{h}{L} = \frac{3V^2}{RgD}$$

and

$$h = \frac{6\,L\,V^2}{R\,D\,2g} = f\,\frac{L\,V^2}{D\,2g} \quad \text{............... } 17.18$$

f being 6/R for smooth surfaces.

From published data laminar flow occurs for R ranging up to between 400 and 900. For rough surfaces $f = C/R$ where tests have shown C varying in the range of 7,9 and 9,8.

With the Manning formula being commonly used for turbulent flows in open canals of irregular shape the velocity (V) and flow rate (Q) can be expressed as:

$$V = \frac{1}{n} r^{0,667} s^{0,5} \quad \text{.............................. } 17.19$$

$$\text{and } Q = \frac{1}{n} \frac{a^{1,667}}{p^{0,667}} s^{0,05} \quad \text{.................... } 17.20$$

where the hydraulic radius $r = a/p$, a being the cross-sectional area, p the wetted perimeter, $S = \iota = h/L$ the energy loss and n the Manning coefficient, per unit length or gradient of canal.

For steady flows in canals the above formula can be adapted to suit different canal shapes as given in various design handbooks in which values of n are also given.

Non-uniform flow conditions relating to headloss and velocities are also widely dealt with in various handbooks such as that referred to above.

17.5 VALVES AND GATES

Valves and gates fulfil numerous important control functions in water works, pumping stations and pipeline systems. The different varieties of gates and valves have not been

TABLE 17.7
TYPICAL VALVES ON WATER WORKS

Valve function	Types of valve for the required function	
1 Isolating valves	(a) Gate valves manufactured to SABS 664 and 191: - metal seating faces - rubber coated gate (resilient seal) especially suited to sludges and slurries (b) Butterfly valves (c) Needle valves (d) Screw-down stop valves - small diameter (e) Diaphragm valves (f) Membrane or rubber tube type (g) Multijet valve (h) Rotary valve - rotary plug - spherical	
2. Air valves	(a) Single small orifice AV	For the release of small volumes of air during the functioning of the pipe system.
	(b) Large orifice AV	For the release and inlet of large volumes of air (Anti-vacuum valve) during the filling and draining cycles respectively of a pipeline or system.
	(c) Double orifice AV	For both above functions.
	(d) Sonic air release V	For surge protection of pipelines.
3. Reflux or non-return valves	(a) Single door	Manufactured to SABS 192
	(b) Multiple door	Used for large diameter applications (suitable for surge protection).
	(c) Swing check butterfly	Not suitable for surge protection purposes.
4. Multiple-purpose valves: (functions) - Pressure relief - Pressure control - Reducing - Sustaining - Flow control - Float or level control	(a) Diaphragm valves	Manufacture: "Clayton", "Canon", etc.
	(b) Membrane type valve	Manufacture: "Grove", "Inbal" etc.
	(c) Needle valve	Manufacture: "Larner Johnston", etc.
	(d) Ringneedle valve	
	(e) Multi-jet Valve	Manufacture: "Neyrtec", "Bailey".
5. Special	(a) Sleeve valve: valves	Discharge to atmosphere, low headloss, cavitation free
	(b) Self centring disc valve:	Level control
	(c) Flow-stop valve:	High flow velocities will cause this valve to close

TABLE 17.8
SELECTION CRITERIA FOR VALVES

Aspect	Criteria and selection	
1. Supply and installation of valves	(a) Valve must be removable from pipework: use flange adaptor, flexible coupling or rigid dismantling joint on one side (b) To prevent drainage into pumping station or from reservoir (tank): bolt valve to flange on discharge pipe of reservoir (tank) (c) Flange drilling and thicknesses to: BS 4504, SABS1123; Bolting to SABS 135 and 136; Gaskets to BS 132	
2. Selection criteria	(a) Gate and butterfly valves: handwheel opening effort: Maximum 400 Newtons on 250 mm radius; Gearing and spindle bearings - see Right Valve selection chart (b) Maximum flow velocity through valve: 10 m/s (c) Minimum valve size: 70% of pipe diameter	
3. Isolating	Most suitable types:	gate, butterfly and spherical valves used valves for OPEN/CLOSED application only
4. Level control valves	Most suitable types:	diaphragm, needle, multi-jet and self-centring disc valve. Arrange for OPEN/CLOSED functioning only - valve must not hunt
5. Pressure relief valves	Most suitable types:	membrane type gas filled, spring loaded, diaphragm valves
6. Flow control	Most suitable types:	membrane, diaphragm or needle valve coupled to flow meter

dealt with extensively in this text as it is a continuously developing technology.

17.5.1 VALVES

A valve is generally required to perform a certain function and valves can therefore conveniently be classified as:

- Isolating
- Air
- Reflux
- Multipurpose
- Special valves

Typical examples of valves suitable to perform these duties are listed in Table 17.7.

17.5.2 USE OF VALVES IN PIPE SYSTEMS

To assist the designer in selecting a valve, important general and selection criteria such as installation and the suitability of certain valves for certain duties have been tabulated in Table 17.8.

17.5.3 GATES

Different types of gate such as circular, square, rectangular, radial and adjustable weir, are available on the market for use in water treatment works. They can be operated with either a headstock (bevel geared, worm geared and electric actuator types), twist loc or tee-key type system.

Gates are mostly used at inlets to water treatment plants, or to shut off flow between reservoir compartments feeding a works.

Handstops and stoplogs are frequently used at inlets to filters and in sludge lagoons respectively, the latter to draw off the supernatant water.

17.5.4 SELECTION AND CONTROLS

Valves and gates are normally selected to perform a specific task and the detail design thereof is usually done by the manufacturer to suit the performance criteria, such as type of fluid or gas conveyed, flow conditions, corrosivity of the fluid and environment and design pressure. With respect to the latter, a valve or gate is normally manufactured to suit a specific design pressure. If it is to be drop tight at, say, 20 m head, it will be 20 m and no more.

By virtue of the nature of water treatment plants the valves and gates are designed for low-pressure duties, and cavitation is not critical.

Due to the reliability required from valves and gates operating under corrosive conditions in a water treatment plant, special care must be given to the corrosion resistance of valve and gate components.

Valves and gates can either be locally or remotely controlled, operated by hand, or hydraulically (fluid pressure in the pipeline or control chamber, oil pump), electrically (censor, solenoid, motor drive), or pneumatically (air or gas activated or driven). The choice must be carefully considered for general and specific applications.

Pumping stations invariably form an integral part of a water treatment works and the control of pumps and the selection of valves need mentioning. For low heads (up to 60 to 75 m) and low kW (say up to 150 kW), centrifugal pump sets can generally be started against an open valve on the delivery side, in which case a hand-operated or motorised gate valve is suitable. In the case of larger pump-motor units, however, it is recommended that a pump should be started against a closed control valve. As a general guide, power-operated control valves should be used in 250 or 300 NB and larger discharge pipes and for 400 kW and larger motor drive units. In the latter case ring needle or spherical valves are suitable.

A reflux valve with a power-operated gate valve is in use on a larger pump set with a delivery head of 50 m (flow 3 m^3/s) and 1 200 kW drive unit.

17.6 FLOW METERS

Flow measurement is an invaluable process control and management tool in a water works. Three basic flow functions, namely flow rate, flow volume and flow control, will enable a works manager to balance the incoming, outgoing and loss volumes at convenient intervals (say monthly) and to effectively dose chemicals between these intervals of constant flow.

To achieve this objective it is necessary to measure the incoming and outgoing flow volumes and rates, rate and volume of filter wash water consumed, volume of chemical make-up water, rate of chemical solutions dosed and, if at all possible, the volume of sludge discharged.

Of the numerous flow metering techniques that have been proposed in the past, only a few have found widespread application. The designer will be faced with a number of acceptable alternatives for each application and will have to trade off the more important between many different parameters such as capital cost, operating cost, maintenance cost, maintainability, accuracy, reliability and energy loss. Accuracy is expressed as a percentage of the specific flow for all meters, except in the case of variable area and pressure differential meters where it relates to the percentage of the full-scale reading or maximum pressure differential.

17.6.1 TYPES OF FLOW METER

The most common flow meters in use today can be categorised in terms of function and drive mechanisms as follows:

Volume meters hydraulically driven:
(a) positive displacement meters
(b) turbine meters.

Rate of flow meters hydraulically driven:
(c) differential pressure meters
(d) variable area or rotameters.

Volume and rate of flow meters electrically driven:
(e) electromagnetic flow meters
(f) vortex shedding flow meters
(g) ultrasonic flow meters.
(h) laser-Doppler meters

17.6.2 FLOW METER SELECTION

By using the characteristics of the fluid being measured (cleanliness, conductivity and viscosity) in conjunction with the most suitable drive mechanism for the locality of use, the range of choice of meters can be narrowed down. After the meters capable of performing the measurement have been identified, the designer should base his choice on a comparison of their respective specifications with the requirements and operating conditions of the application. Fisher and Porter offer a selection guide that can be used.

17.7 CORROSION

Corrosion can be defined as being a destructive attack on metals and materials which may be chemical or electro-chemical in nature.

As various metals, thermoplastics and cementitious based materials are the engineering materials most commonly used in a water works, the attention will be limited to these materials and to the corrosion mechanisms, while some design parameters and preventative measures will be dealt with briefly.

17.7.1 CORROSION MECHANISMS

The corrosion mechanisms of the most important engineering materials used in water purification works, i.e. concrete and metals, are dealt with extensively in Chapter 13.

17.7.2 CORROSION POTENTIAL AND PROTECTION

The potential of surface and ground water to corrode concrete can be expressed in terms of the Langelier and Corrosivity Ratio indices. These indices are, however, only indicative and not absolute parameters.

Recommended norms and standards of materials and coatings for various structural components, pipes and containers for the

Table 17.9
Materials and Coatings for Storage of Fluids

Component	Fluid conveyed or stored	Specification or norm
1. Pipes at water works	Treated and untreated waters	Epoxy lined and coated to SABS 1217 (solvent free epoxy is preferred)
2. Long supply pipelines - steel	Treated and untreated waters	Epoxy lined and coated to SABS 1217. Cement mortar lined to AWWA 602. Cathodic protection of buried and submerged structures
3. Supply pipelines	Untreated low pH and low alkalinity water with high Fe & Mn	u PVC pipes to SABS 966
4. Weirs in water works	Treated and untreated waters	Stainless steel or 3Cr.12
5. Chlorine room, highly chlorinated water	Chlorine gas around plant	u PVC pipes to SABS 966 or polyethylene pipes
6. Chemical solution and make-up tanks	Corrosive chemical solutions	- Fibreglass lined - Butyl rubber lined - Epoxy lined - Polyethylene tanks
7. Floors around chemical solution and make-up tanks	-	Epoxy coated
8. Structural steel	Protective coating of iron and steel structures against corrosion	BS 5493: Code of practice (Galvanising not recommended for steel piping). SABS 0120 Part 3 Section H.C.

conveyance and storage of fluids for a water works are listed in Table 17.9.

17.8 Design Considerations and Criteria

Although most of the criteria and design considerations have been dealt with in the foregoing paragraphs it is considered important to elaborate slightly on quality control and acceptance tests of pump and motor units.

The purpose of performing quality control and acceptance tests on equipment is to demonstrate the fulfilment of the technical, hydraulic and mechanical specifications and the guarantees agreed to in contracts for the mechanical and electrical equipment. It should in essence be applied to all items of equipment supplied and installed in a purification works. The South African Standard "Code of Practice for Quality Management Systems" SABS 0157 (Parts I to III) has been drawn up for this purpose.

Various inspections and tests should be

performed at different stages. During the manufacturing stage, processes and procedures should be adopted for the testing of materials and cabling, and pressure tests performed. Likewise there are various other standards that should preferably apply for the acceptance of pump units (DIN. 1944; ISO 5195, 3555 and 2548; API 610), for vibration tests and limits for bearings (VDI 2056) and drive drafts (API 610) and noise levels that emanate from mechanical (bearings and balance) as well as hydraulic causes (turbulent flow, and non-ideal flow profiles through impellers). When considering the hydraulic efficiency of a pump, penalties can accrue for efficiencies below that specified. The Department of Water Affairs and Forestry, for example, apply penalties for efficiencies below specification (R4000,00 per kW - 1993 rate).

A designer should give due consideration to housing all major pumping and air transfer plant in a separate room to reduce noise levels in the remaining areas of a water treatment works.

18

Instrumentation and Process Control

F J Retief

In order to operate any complex process system properly, one needs to know at all times what is happening with all aspects of the process. For many modern installations, adequate operator information cannot be obtained by direct observation only. In many cases, simple manual control of equipment is not adequate either. Because of these needs, a distinct technology, industrial instrumentation, has developed.

It is not possible to treat this matter in a single chapter so comprehensively that it can be used as a design guide to the application of industrial instrumentation methods in this branch of process engineering. This chapter should rather be seen as a tutorial in the concepts of industrial instrumentation and should enable the designer to specify his instrumentation needs.

Compared to civil engineering, instrumentation is a very new technology and there is a large difference in maturity between the two disciplines. This is especially true of the quality of work which is accepted as "standard practice". In civil engineering practices have been standardised and formalised in numerous codes of practice which have been formulated by many different bodies such as the SABS. By comparison, very little such material exists in the realm of instrumentation engineering. The result is that, unless special care is taken, installation standards will vary enormously. An indication of this is the fact that, when instrumentation tenders are invited, it is quite common to obtain tendered prices which vary over a range of 1,5 to 1.

Designing and building a structure require considerable knowledge and input. Following this, however, not much involvement is required from the designer and constructor. This is not the case with instrumentation systems. After installation on site, it is generally necessary for an instrumentation installation to undergo a fairly lengthy commissioning period. This needs to be allowed for in the construction programme of the project as well as in the conditions of payment of the instrumentation contract.

Signing an instrumentation contract is therefore very much an "until death us do part" undertaking. In this situation, the practice of simply allocating a tender to the lowest tenderer, is obviously fraught with pitfalls.

Maintenance on civil structures is mostly carried out for aesthetic, rather that functional, reasons. There are, of course, exceptions. In the case of instrumentation systems, however, routine maintenance is essential if reliable functioning and a long service life is to be expected.

The applications and functions of instrumentation - i.e. the destinations which are to be reached, are discussed. The technologies of process control - will be treated in Sections 18.2 to 18.6.

Section 18.7 deals with the subjects of reliability and maintenance.

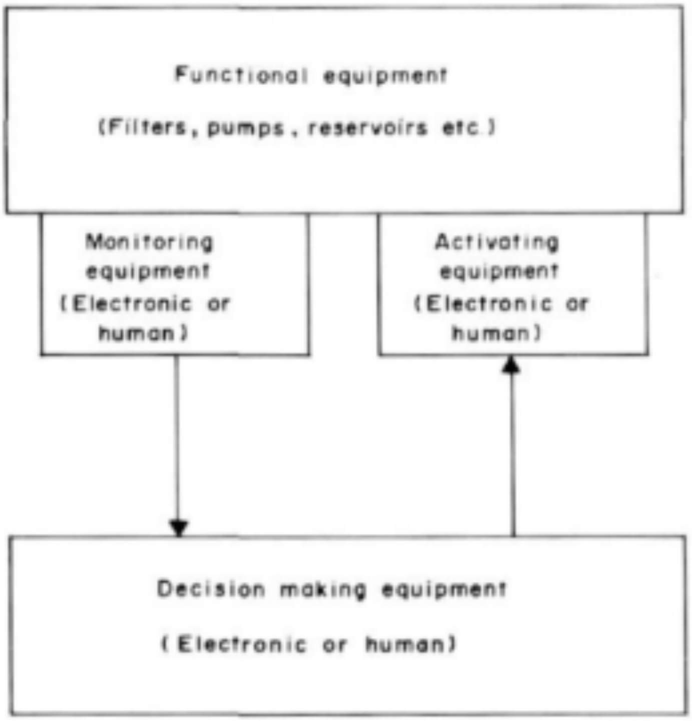


Figure 18.1
The elements of the control process

18.1 FUNCTIONS OF INSTRUMENTATION IN WATER WORKS

It is important to note that **all** process control takes place in three stages. These are shown in Figure 18.1.

- **Monitor** what is happening on the works.
- On the basis of what has been observed, **decide** whether any action should be taken and, if so, what that action should be.
- **Activate** some device in response to the decision.

This cycle of events is referred to as a **control loop**.

If all three steps are executed with physical equipment, i.e. without any human assistance, we have an instance of **automatic** control. If human intervention is required for any of the steps, we have **manual** control.

The functions of instrumentation on a water works can largely be described in these terms and variations on them.

18.1.1 LOCAL MONITORING AND/OR CONTROL

Probably the most elementary application of electronic instrumentation in water engineering is the provision of a measuring device to monitor the value of a process variable, convert it into an electric signal and make the result visible to an operator. In such cases it is very often not necessary to use electronic equipment, and a direct-reading mechanical instrument may be used. A Bourdon pressure gauge on the scavenge pipe of a reservoir to measure the depth of water in the reservoir, and a bimetallic strip thermometer to monitor the temperature of a pump bearing, are typical examples.

In such situations the decision and execution stages of the control cycle must necessarily be carried out manually by the operator.

It is also possible to connect an electronic measuring instrument to some decision-making device which can then activate an executive device. A simple example would be a flow transducer which is installed in the raw water pipe from a reservoir to a purification works. The flow transducer measures the flow, it is electronically compared with a desired value (the decision function) and, if required, a valve is adjusted to maintain a constant flow (the executive action). If all the equipment is installed in the immediate vicinity of the reservoir and not in some remote control room, the situation is referred to as "local control". If the signals have to be transmitted for any reasonable distance, it is referred to "remote control".

The concepts of "local" and "remote" are of course ambiguous. If two persons are some distance apart from each other, what is "local" to the one is "remote" to the other and *vice versa*. This can be confusing and one needs to be aware of the implied location if misunderstandings are to be avoided.

Note also that the distinction between "manual" and "automatic" is completely independent of the distinction between "local" and "remote". Figure 18.2 illustrates these concepts and typical applications of instrumentation in water treatment and transport.

18.1.2 ALARMING

Most water works processes can run for hours on end without operator intervention. Under these circumstances it is unrealistic to expect the operators to keep a watchful eye on all aspects of the operation at all times. The situation therefore requires automatic alarming.

In a simple alarm system an instrument would monitor a variable and generate a signal when an alarm condition is detected. If there are only a few conditions on the plant which need to be alarmed, it may be enough to have a siren or other noise-making device to indicate when a problem arises. However, if there are several plant conditions which can cause alarms, then it becomes necessary to give the operator some indication of which particular condition is causing problems by means of an alarm indication device. This may be a hardware alarm panel or, if a process control computer is being used, this function will normally be carried out by the computer.

A printed alarm log can also be useful.

18.1.3 REMOTE MONITORING AND/OR CONTROL

There are a number of situations in which local control is not adequate and when remote control may be required, such as:

- The measuring point and the controlled point are physically far from each other. The classical example of this is the case of a pump which has to pump water to a remote reservoir. The measuring point is at the reservoir, but the control point is at the pump station.
- A remotely located facility, usually a pump, has to be operated unmanned. A common configuration here is to have the pump controlled locally (referred to the pump station!) and automatically by level switches, but to provide remote monitoring of the status of the pump station from a manned station. Remote control is, of course, also a possibility.
- A widely distributed facility, comprising perhaps an inlet works, a chemical dosing plant, a filtration unit and a clear water reservoir, should preferably all be controlled from a central point.

While "remote" control is impressive, it is also much more expensive. It should therefore only be used in those situations which justify the greater expense.

18.1.4 SAFETY INTERLOCKING

If things go wrong, either through equipment failure or operator error, many water handling processes can become quite dangerous. The situation then requires some form of "safety interlocking". It can be de-

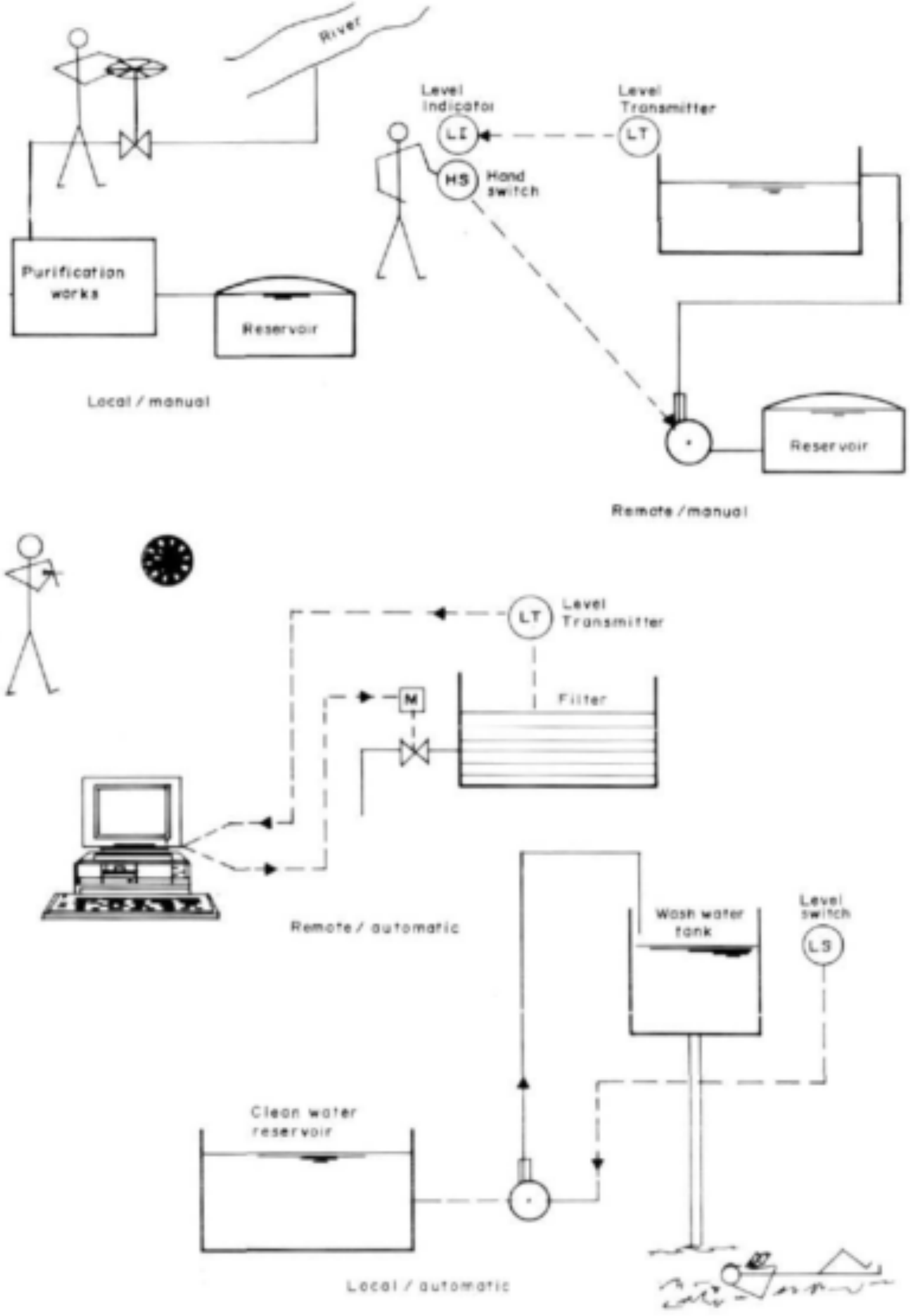


Figure 18.2
Various control modes

fined as the use of instrumentation to prevent operators or machines from taking actions which can be damaging to themselves, to others, to equipment or to the environment, or which may result in loss of water or disruption of service.

A simple example is the fitting of switches to the doors of electrical cubicles which deactivate the internal circuitry when the door is opened. Another example is the low level switch which is wired into the circuitry of a pump and which will prevent the pump from starting if the level of water in the supply reservoir to the pump is too low. Much more complicated examples exist. Situations arise where a certain action is permissible under certain conditions but forbidden under others.

Simple interlocking functions are implemented with hardwired logic circuitry, but more complex cases are best handled by means of some sort of a computing device.

18.1.5 AUTOMATIC SEQUENCING

The best example of automatic sequencing in a water works is the process of backwashing a sand filter. This involves about twenty different steps which have to be executed in a specific sequence. A step is initiated only if the previous step has been completed. This is usually signalled by a switch which is activated by the completion of the step or by means of a timer.

This type of function is repetitive and that is one field where computers generally outperform humans. Since the following of an incorrect sequence will often lead to damage, the expense of installing some form of automatic sequencing system to backwash filters, or starting large pump sets, etc., may well be justified.

18.1.6 AUTOMATIC CONTROL

It should be quite clear by now that classic automatic control is not by any means the only application of electronic equipment in water engineering. It is nevertheless a major application.

The decision step of automatic control can be extremely complex. Many process control loops contain time delays, either in the physical process itself, or in the measuring or actuating steps. The general result is that, by the time that the decision-making device discovers that things are going wrong, it is already rather late to take action. If the device is designed to take drastic action under these conditions, it is quite possible that it will overreact and that the whole system will tend to oscillate. If the device is designed to take modest action when it discovers that the process is not doing what it should, the modest action may not be enough to put matters right. The required control strategy is therefore critically dependent on the response times of the process, the control valves, etc.

Standard process controllers exist which can be adjusted to allow for the dynamic characteristics of a particular process. If it is a hardware process controller, this tuning is done by the adjustment on site of some settings on the device. If the control function is being implemented by computer, the tuning is done by loading the appropriate tuning constants into a preprogrammed control program.

The two most common control strategies, both implementable in hardware as well as in software, are known as on/off control and PID control. On/off control (usually with a "dead band" incorporated) is used for the control of a binary controlled device such as a pump set or a valve which is used only in its fully open or fully closed states. PID (Proportional-Integral-Derivative) control is used with devices which can accept an analog control signal. Examples are variable speed pumps, valves which are used to modulate flow, etc.

18.1.7 DATA LOGGING

Water engineers are familiar with the sight of an operator doing the rounds of his water works with a clipboard in his hand and writing down observed values of flow, pressure, etc.

This logging process can be automated. This requires that all the observed variables be monitored by electronic instrumentation, that their signals be wired to a central point, usually a control room, and that chart recorders and/or a logging printer be installed there. If a printer is used, it requires a computing device of some kind to sample the various signals at regular intervals, calculate averages, and arrange everything in columns, etc., before the results can be presented to the printer.

Some water works owners prefer not to have an automatic logging system. They believe that operators should tour the plant regularly, and consider the forced discipline of manual data logging from local meters to be a plus point.

18.2 PROCESS MEASUREMENT TECHNIQUES

In this section, paragraphs 18.2.1, 18.2.2 and 18.2.3 deal with general matters pertaining to process measurement techniques. The remaining paragraphs deal with the measurement of those process variables which are commonly encountered in water purification engineering.

18.2.1 SWITCHES AND TRANSMITTERS

The signals which are produced by electronic measuring devices, are of two kinds - analog and binary. The latter are often referred to as "digital" signals, but "binary" is more descriptive and accurate.

An analog measuring device, referred to as a transducer or **transmitter**, produces an electric signal which can vary over a range of values with the strength of the signal representing the measured value.

For control purposes it is often not necessary to know the precise value of a particular process variable. It may be enough to know only that the variable is above or below a certain specific level. The information that a pump bearing is at a temperature of "less than 95°C" is enough to tell an operator or computer that the bearing is not overheating and in good order. **Switches** produce a two-valued (i.e. binary) signal only. Either the signal is present, or it is absent. Its precise value when it is present, is not important. Some things which happen in water treatment are inherently binary in nature and can only be monitored by means of a switch. A pump motor is either tripped or not tripped. It cannot be 34% tripped! Such "variables" are universally monitored by means of switches.

All process variables such as flow, temperature, etc. can be monitored either by switches or transmitters. Transmitters for a particular process variable cost several times as much as a switch for the same variable. They also require more careful installation, calibration and maintenance and the processing of the generated analog signal by a computer, or other electronic device, is also much more expensive than the processing of a binary signal. A transmitter does produce more information than a switch, but the extra information comes only at a significant cost in terms of money and complexity.

Unless that information is really useful, a switch should rather be used.

18.2.2 NOTATION

Engineers tend to describe their designs in terms of drawings rather than lines of text. Such drawings, especially if they are symbolic in nature, use sets of symbols which need to be defined and agreed upon. The Instrument Society of America has produced such a set of symbols which are universally used by instrumentation technologists. Because of the extremely wide variety of instrumentation devices which exist, the ISA symbol system employs a circle for all instruments and defines the function of the instrument by means of a group of letters written within the circle.

The first letter of this code refers to the process variable which is being measured, for instance, an "F" for flow, "P" for pressure, etc. Subsequent letters qualify the device further. "S" signifies a switch, "T" a transmitter, "I" an indicator, etc. A simplified version of the symbol system is given in the following table.

First character	Meaning	Other characters	Meaning
F	Flow	A	Alarm
L	Level	C	Controller
P	Pressure	H	High
T	Temperature	I	Indicator
W	Mass	L	Low
X	Status	O	Open
Z	Position	Q	Quantity
		R	Running
		S	Switch or stopped
		T	Transmitter

Example 1: FIT Flow Transmitter with local Indicator
Example 2: LSLLevel Switch to operate when level is Low
Example 3: XSR Status Switch to show that pump is Running

When used in combination with a Piping and Instrumentation (P&I) Diagram as illustrated in Figure 18.3, this is the primary communications medium between the process engineers and the instrumentation engineers on a project. It is advisable that this simple system be understood and used by all involved.

18.2.3 MEASUREMENT RANGE

Measuring instruments generally have a limited range within which it may be assumed to provide accurate measurements. Although the gauge range is perhaps from 0 to 100 units, it will not be accurate at the lower end of this range. The ratio between the full-scale value and the lowest value which will produce an accurate reading, is known as the "turndown" ratio. Effective turndown ratios of instruments can be as low as 4 or as high as 20. Since it is difficult to define, it is usually not explicitly stated in instrument specifications.

18.2.4 FLOW METERS

Flow is one of the most important variables measured on a water works. A variety of methods of measuring flow exists and each type of meter has its own particular set of characteristics. It is therefore essential that the requirements of a particular application be taken into consideration when selecting a meter for that application.

For accurate results it is a requirement of all flow meters that there must be a straight length of pipe both upstream and downstream of the flow meter. Typical values are ten pipe diameters upstream and five diameters downstream. The suppliers' recommendations should, however, be checked.

There are two types of flow meter. Flow rate meters produce an analog signal which

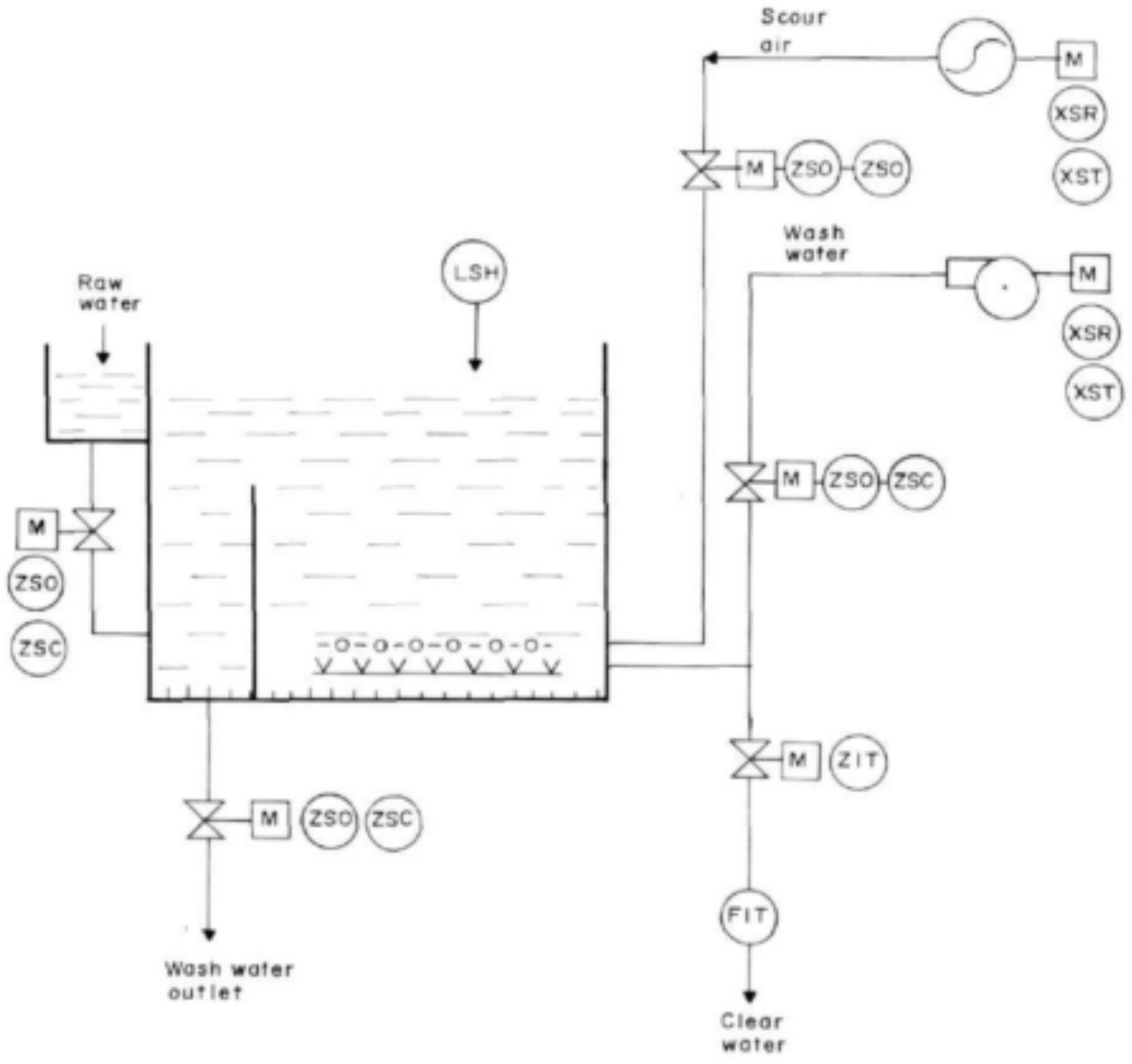


Figure 18.3
P & I diagram of filter

is proportional to the instantaneous flow rate in litres per second (say). Flow quantity meters produce a short binary pulse for every cubic meter (say) of water which has flowed through the meter. A pulse counting device is then necessary to determine the total quantity of water which has been metered. All respectable flow meters can provide both types of signal.

The most accurate, and most expensive, of flow meter types is the magnetic flow meter. The water, which is electrically slightly conducting, is made to move through a magnetic field. The interaction of the moving conductor with the magnetic field produces a small electric voltage which is proportional to the flow velocity. This voltage is amplified and processed to provide a suitable output signal. The turndown ratio is about 15 to 1.

Next down the line of the price/performance curve is the full-flow turbine or propeller type. The speed of rotation of a turbine in the bore of the pipe is converted into an electrical signal. The measurement range is also about 15 to 1.

For large flow rates, the magnetic and full-flow turbine types are the only ones which are sufficiently accurate to produce readings which are generally considered acceptable for fiscal purposes. For small flow rates, the positive displacement meter is also acceptable in this role, but they are limited to flow rates which are smaller than those likely to be encountered in a water treatment works.

The above-mentioned magnetic and turbine flow meters are of the full-flow type. Versions of these meters which are mounted on a probe which is inserted through the wall of the pipe to a point somewhere in the flow stream, are also available. The device then measures, either electromagnetically or by means of a small paddle, the flow velocity at that point. It is then assumed that the observed velocity is representative of the flow velocity profile throughout the pipe. These devices are less expensive, but also less accurate, than their full-flow cousins. Their measurement ranges are about 10 to 1.

There are two type of flow meter which transmit pulses of ultrasonic sound energy into the pipe and which work on the principle that the movement of the water in the pipe affects the apparent velocity of the sound in the water. The transmission type transmits a signal from a transmitter on the one side of the pipe to a receiver on the other side, but somewhat upstream or downstream. Such instruments are moderately priced and can give good results. The other type transmits the pulses at an upstream slanting angle into the pipe. The pulses are reflected by minute particles and bubbles in the water and the reflected sound is picked up by the receiver - which may be the same physical device as the transmitter. Because of the uncertainty of the effective distance from the wall of the pipe to the point from which the effective reflection occurs, and other problems, these instruments should only be used if high accuracy is not required or if the instrument can be volumetrically calibrated after installation. Some models are relatively inexpensive. Both versions have the considerable advantage that some models can transmit the ultrasonic signals through the walls of steel pipes. This permits the retrofitting of such meters to existing pipes without cutting into the pipe at all.

Differential pressure flow meters have a flow restriction or other device in the pipe which cause a pressure drop which is proportional to the square of the flow velocity. This pressure drop is measured and the square root electronically derived to provide a signal which is proportional to flow. The measurement range of these instruments is limited to about 5 to 1.

Various types of flow switches exist, but they are not very often used.

For direct observation and small flows an old stalwart is the variable area or rotameter type of flow meter. In this type a circular plug fits loosely into a vertical tube which is slightly conical - the thinner end of the cone being at the bottom. The fluid moves from the bottom to the top of the tube past the plug and raises it to a level which is proportional to the flow rate.

The flow measuring devices discussed thus far are only suitable for measuring full bore flow in pipes. Flow in open channels is invariably measured by measuring a level change which is caused by a venturi flume, a parshall flume or one of a variety of weirs. Any of the level measuring devices which is discussed in the next paragraph, can be used in this application. A common feature of all these devices is that the observed level is not a linear function of the flow to be measured.

Linearisation of the measured level signal is thus required, but this can usually be done fairly easily by electronic hardware or software.

This list of flow meter types is not comprehensive, but it does cover the more common types.

18.2.5 LEVEL METERS

Three methods of level measurement are commonly used.

The first uses a pressure sensor to measure the pressure at the bottom of a reservoir, etc. If proper precautions are not taken, a small error will be caused by changes in atmospheric pressure. Some pressure gauges which are used in this application have an outlet to atmosphere which corrects for this error. Some designs require that the pressure gauge be mounted on a scour pipe or similar outlet to which access can be obtained. In other versions the pressure sensor is mounted on the end of a long waterproof cable and it is installed by simply suspending it from the roof of the reservoir to a point near or on the floor. This is the least expensive way of measuring level.

The ultrasonic level sensor is installed above the highest level which the water will ever go and is thus not in contact with the water at all. It transmits pulses of ultrasonic sound down to the water and measures the time which it takes the sound reflected from the surface to arrive back at the transmitter.

In the case of the capacitive level transmitter, a metal pipe which is covered with a thin layer of electrically isolating material, is suspended vertically in the vessel of which the water level is to be measured. The water (which is electrically slightly conductive) on the outside of the isolating layer forms a capacitor with the metal pipe on the inside. The higher the water level, the greater the capacity. This variation in capacity is measured electronically and converted into a usable signal.

There is not much to choose between these three methods of measuring level. All three can be successfully used in most cases.

All three of the above-mentioned ways of obtaining analog measurements of level can be adapted to give only a level switching action. In addition, if a level switch only is required, the conductive switch can be used. This relatively simple device has two electrodes which are installed at the level at which the switch must operate. When the water reaches the electrodes, it creates an electrical connection between them and this is used to close a contact. Tilting float switches can also be considered for this application.

18.2.6 PRESSURE METERS

The most popular way to measure pressure uses a strain gauge which is mounted on a diaphragm which is exposed to the pressure or differential pressure which is to be measured. Earlier models suffered from calibration drift. Modern versions, which use molecularly bonded strain gauges, have overcome this problem.

Pressure switches generally use the actual movement of a diaphragm against an adjustable spring to activate an ordinary electromechanical switch.

For direct observation of pressure, bourdon tube-based types are still commonly.

18.2.7 TEMPERATURE METERS

Apart from the classic mercury or alcohol thermometer, which can be used only for direct observation of temperature, there are four devices which can convert temperature into an electrical signal. These are the thermocouple, resistance thermometer, bimetallic strip and the thermistor.

Thermocouples consist of wires made of two dissimilar metals which are joined together. When the joint is heated, an electrical voltage is generated.

The electrical resistance of all pure metals increases significantly and very linearly with increasing temperature. The resistance thermometer uses this property.

The well-known bimetallic strip is used extensively as the functional element of temperature switches.

A thermistor is a special type of resistor of which the resistance drops considerably when its temperature increases over a fairly narrow temperature range. Thermistors are used in small range temperature transmitters and in temperature switches.

In a water treatment works these devices are used mainly for the monitoring of bearing and motor winding temperatures.

18.2.8 Turbidity and colour meters

Turbidity is an indication of the amount of particulate matter in water. It is measured by directing a beam of light at a sample of the water to be assessed. In some instruments the attenuation of the light as a result of the presence of solids, is measured. In other instruments the light which is scattered by the suspended particles, is measured. Variations on the theme exist. Thus the falling stream absorptiometer is more suitable for the measurement of high turbidities such as may exist in raw water while the nephelometer type of instrument is more suitable for the low turbidities associated with purified water.

The tendency for turbidity meters to foul their optics, and other reasons, causes them to lose their calibration rather quickly. Regular cleaning and calibration with a liquid of standard turbidity is therefore essential.

Colour has an effect on turbidity readings since the particular matter may have a dark or light colour. Organic or inorganic solvents may also affect the colour of water. A colorimeter which uses platinum cobalt as reference, can be used to determine water colour.

18.2.9 Chlorine residual meters

On-line residual chlorine measurement can be done either by an amperometric titrator or by a colorimeter. Because colour and turbidity interfere with colorimetric measurements, the amperometric method is preferred.

18.2.10 pH meters

pH is measured by means of a special electrode which generates a voltage which is a function of the hydrogen ion concentration of the water. These electrodes contain an extremely delicate glass membrane which has a tendency to foul up, especially in turbid water. Like turbidity meters, these instruments therefore require much more regular attention than, say, pressure or flow transducers.

18.3 Decision-Making Equipment

Having measured how closely the plant is performing to its specification, someone or something must decide whether this is acceptable or whether some variable must be adjusted to obtain better operation. The following options are available:

18.3.1 HUMANS

The human being is an extremely valuable decision-making "machine". Compared with electronic equipment, man's strongest point is his flexibility in unusual and unpredictable situations. His weakest points are his inability to concentrate on a task for long periods and his relative unpredictability and unreliability. Automatic equipment excels in these situations.

18.3.2 COMPUTERS

The ability of computers to accept data, analyse them according to some programmed procedure and then to produce a result, is well-known to all engineers.

If this ability is to be exploited in an instrumentation system, the computer has to be connected electrically to the source of the data as well as to the equipment which is to be controlled. This interface between computer and environment is known as an I/O (Input/Output) system. Computers can of course process both binary and analog signals. An Input/Output system for a process control computer therefore consists basically of devices which can carry out the following functions:

- Receive binary signals from measuring equipment
- Receive analog signals from measuring equipment
- Output binary signals to control equipment
- Output analog signals to control equipment.

Since, compared to some other process industries, the control requirements of water engineering are relatively modest, a PC type of computer is normally sufficiently powerful to carry out all tasks which may be required.

The software which is required to implement the various instrumentation functions which were mentioned in the previous paragraph, is normally not written in a standard computer language like Pascal or Fortran. Software specialists have written suits of modular programs which can relatively easily be configured to the needs of each individual user. Such software suits are know as SCADA (Supervisory Control And Data Acquisition) packages. This subject will be returned to in paragraph 18.6.3.

18.3.3 PROGRAMMABLE LOGIC CONTROLLERS

Because general purpose computers are extremely flexible machines, programming them for process control applications used to be a task for specialist programmers only. Recognising this fact, computer-like machines, called Programmable Logic Controllers (PLCs), were developed which are more limited in their applications, but much easier to program and use in the process control environment.

A typical PLC has a powerful I/O system, a data-processing unit and a memory in which to store its control program. It would mostly not have a keyboard or a video screen and can therefore communicate directly with the process to be controlled, but not with the operator. How this is done, will be described later.

PLCs come in a wide range of sizes, capabilities and prices.

18.3.4 INTELLIGENT INSTRUMENTS

The very low cost of microprocessors has led to the situation where many measuring instruments have small computers built into them. These are imbedded into the circuitry of the instrument and the user of such an instrument may not even be aware that it is there. They fulfil all kinds of functions within the instrument such as square root extraction in orifice plate flow meters, driving digital displays, scale changes, etc.

This also creates the opportunity to turn a measuring instrument into a decision-making device as well. Thus one can buy flow meters which can be set up to directly control a valve in order to implement a flow control loop - or a level meter can be used to directly control a pump. Many variations on this theme exist.

18.3.5 NOTE ABOUT COMPUTERS

The many kinds of computers, micro- and otherwise, which one hears about, can be very confusing. From the user's point of view, one of the most important parameters by which one could classify any device with computing capabilities, is the ease with which the end user can change its control program. Thus, for the three types of computing device described above, the situation is as follows. SCADA programs are designed to be very user-friendly and any "computer literate" person can learn to reconfigure an application within about two days. No additional equipment is required. PLCs are normally programmed by the contractor, take perhaps two weeks of training to master and, since they have no keyboard or screen, have to be programmed via a PC. The embedded microcomputer can normally be programmed by the original equipment manufacturer only who uses special equipment for this task. If the equipment manufacturer is in a direct contractual relationship with the end user, the possibility exists that the supplier might program his device to the specific user's needs. If, however, the microcomputer is built into a standard piece of equipment which is perhaps imported, then the control program is "untouchable".

18.3.6 HARDWIRED LOGIC

Computers in their various forms probably account for the bulk of the decision-making equipment used in process control. When, however, the decision rules are simple and the number of installations where they have to be applied are limited to one or a few only, hardwired logic which is based on relays, timers, etc., is still a viable proposition. Situations which often arise are the need to control a small pump station or in the control circuitry of large electrical motors. In the latter case it often makes good sense to hardwire some of the most critical safety interlock functions directly into the control wiring of the motor. Secondary safety interlocking, start-up sequencing and other more complicated control functions which relate not only to the specific motor, but to the works as a whole, can then be given to a PLC or control computer.

18.4 FINAL ACTUATORS

When an operator or an automatic device decides that the process is not performing as it should and he/it takes executive action, that step eventually boils down to one of four actions:

- Switching a motor (usually electric) on or off

- Changing the speed of a variable speed motor
- Opening or closing a valve
- Changing the setting of a modulating valve.

The devices which implement these functions are known as final actuators.

18.4.1 MCCs

This common abbreviation has two meanings. The control unit for a single electric motor is known as a Motor Control Circuit. When a number of such units are built into a larger panel to control a group of electric motors from a single room, the whole panel is known as a Motor Control Centre.

From the point of view of instrumentation, the MCC (in both meanings) is the place where the instrumentation interfaces with the electric motors being controlled. This is the place where the computer or PLC monitors the status of the system via its input modules - i.e., whether the motor is running or not, whether it is perhaps tripped and therefore not available for duty, etc. This is also the destination for the signal from the computer output module which will make the motor run when the computer instructs it to do so.

Since the instrumentation for a works is often supplied by one contractor and the electrical system by another, great care must be taken to ensure that this interface - and all other signal interfaces - are precisely defined. Most civil engineers have at one time or another been very frustrated because the pump which arrived on site did not fit the pedestal which had been cast for it! The electrical/instrumentation interface problem is the instrumentation engineer's analog of this situation.

18.4.2 VARIABLE SPEED DRIVES

These devices accept an analog signal from an analog output module of a computing device and adjusts the speed of an electric motor proportionally to the strength of the signal.

18.4.3 VALVE ACTUATORS

Valve actuators can be electrically, hydraulically or pneumatically driven. Each type has its advantages and disadvantages and its proper areas of application.

From an instrumentation point of view, it usually makes little difference which type is chosen. This choice can be based almost exclusively on economic and hydrodynamic considerations. Linear motion plug type valves driven by a diaphragm can sometimes be a problem because the motion of the valve shaft near the fully closed position is critical and it is therefore difficult to obtain a feedback signal to confirm that the valve is closed.

18.4.4 MODULATING VALVE ACTUATORS

If the actuator is electrically driven, modulating control of valves can be achieved in one of two ways.

The first method does not require an analog output signal from the controlling device, but only two binary signals. Valves take a finite time to move and computers are very fast. It is therefore usually possible to position a valve at any desired point in its control range by sending it a signal to run towards that point and to remove the signal when it is reached. The electric motors which are used in such valve positioners are equipped with brakes which lock the rotor in position when it is not running. It is therefore safe to assume that if a valve has been set to a certain position, it will stay in that position.

The second method to control a valve electrically is to use an automatic valve positioner. This instrument accepts an analog signal from its controlling device and sees to it that the valve is positioned in proportion to the signal, thus somewhat simplifying the programming of the computing device. This method is significantly more expensive than the first method.

When the actuating device is pneumatically or hydraulically energised, a driving piston or diaphragm is used. It has to be assumed that such a piston, and/or the valves which control the flow of the driving fluid to the actuator, can leak and thus, unlike in the case of the electric actuator, it may not be assumed that a valve, if once positioned, will stay in exactly the same position. In the case of pneumatically driven actuators, the compressibility of the driving air can also cause variations in the setting. The only solution then is to use the pneumatic or hydraulic equivalent of a valve positioner. Such devices accept an electric analog signal, convert it into a "fluid" signal and position the valve continuously to the required position by means of fluid amplifiers, etc. These devices are less expensive than their electrical counterparts and to control a modulating valve by means of a fluid motor and positioner is often an attractive option.

18.5 OPERATOR INTERFACES

The information which is being gathered by the transmitters and switches of an instrumentation system needs to be made available to the operators in some or other form. The operator must also have mechanisms by means of which he can instruct the instrumentation system and the plant what to do. Various methods, which vary in sophistication and complexity, are available for this purpose.

18.5.1 DIRECT OBSERVATION AND MANUAL CONTROL

In simple instrumentation systems, locally mounted, non-electronic meters with manual control of final actuators may be the only control mode provided. In more complex systems where some form of remote and/or automatic control is used, one must allow for the possibility that the automatic control or remote signalling equipment may fail. In such a case one is forced to return temporarily to local manual control. It is therefore generally advisable to have local indicators on process transmitters.

As was pointed out before, there are only two types of final actuator: motor controllers and valve controllers. It is standard practice to equip these controllers with push buttons, etc. so that they can be manually controlled by the operator. This applies even if the final actuator will normally be remotely controlled. In such cases the actuator is also equipped with a local/remote switch. The local mode is useful for equipment tests, for commissioning, and as a back-up control mechanism.

18.5.2 CONTROL PANELS

Where a large number of points have to be monitored and/or controlled, or if they are spread over an uncomfortably large area, it makes sense to wire all the monitoring and control signals to a centrally located control panel. On such a panel one can find:

- Indication lamps to indicate the status of received binary signals
- Meters to indicate the values of received analog signals
- Switches and push buttons to generate binary control signals
- Adjustable knobs to generate analog control signals
- Recording equipment
- Alarm annunciating equipment
- Intelligent instruments which have some decision-making capability.

If it is required of the control panel that it should have a moderate or high level of decision-making ability, the required hardwired logic or a PLC may be built into it. As these computer-like devices have no key boards or screens; the control panel is one way in which they communicate with the operator.

Since water works require relatively little interaction between their operators and the works itself, operators do not work at the control panel for extended periods of time. Control desks with sloping surfaces are therefore not often necessary. Vertically faced panels are adequate.

18.5.3 SCADA COMPUTER SYSTEMS

Supervisory control and data acquisition (SCADA) is the most powerful mechanism available to an operator to interface or communicate with the works by means of a computer. When a computer is used as a process controller, it is used with software suits known as SCADA packages.

When a water works or water distribution system is large enough to justify a central control room and a control computer, a SCADA system is the standard solution. They are used in all process control industries - not only in water engineering. Very advanced SCADA software such as that sometimes required by the petrochemical industry, will only run on special process control computers, but SCADA packages which are suitable for the more modest requirements of water engineering, will run on any of the larger IBM-compatible PCs.

See Figure 18.4 for a block diagram of a typical SCADA system. The major blocks on the diagram, from the top down, are the following:

- At the top is the functional equipment of the works (the filters, valves, pumps, etc.) divided into major functional blocks. One block may represent a filtration plant, another a raw water pump station, etc. There can be as many blocks as the situation requires. Each piece of functional equipment is monitored and controlled by measuring equipment and final actuators as described before.

- The signals generated and required by this equipment are wired to PLCs - generally one PLC for each functional block of plant. The PLC is installed in a control panel in the immediate vicinity of the equipment being served by it. The control panel is usually designed and the PLC programmed to enable the functional block served by the PLC to work as an independent entity.

- The PLCs are wired together and to the central SCADA computer by means of a medium-speed data highway. This can be either a copper wire or fibre optic cable.

  The SCADA software which runs in the computer comprises a number of software modules.

- Basic to the whole system is a data base. This is a store of information which does

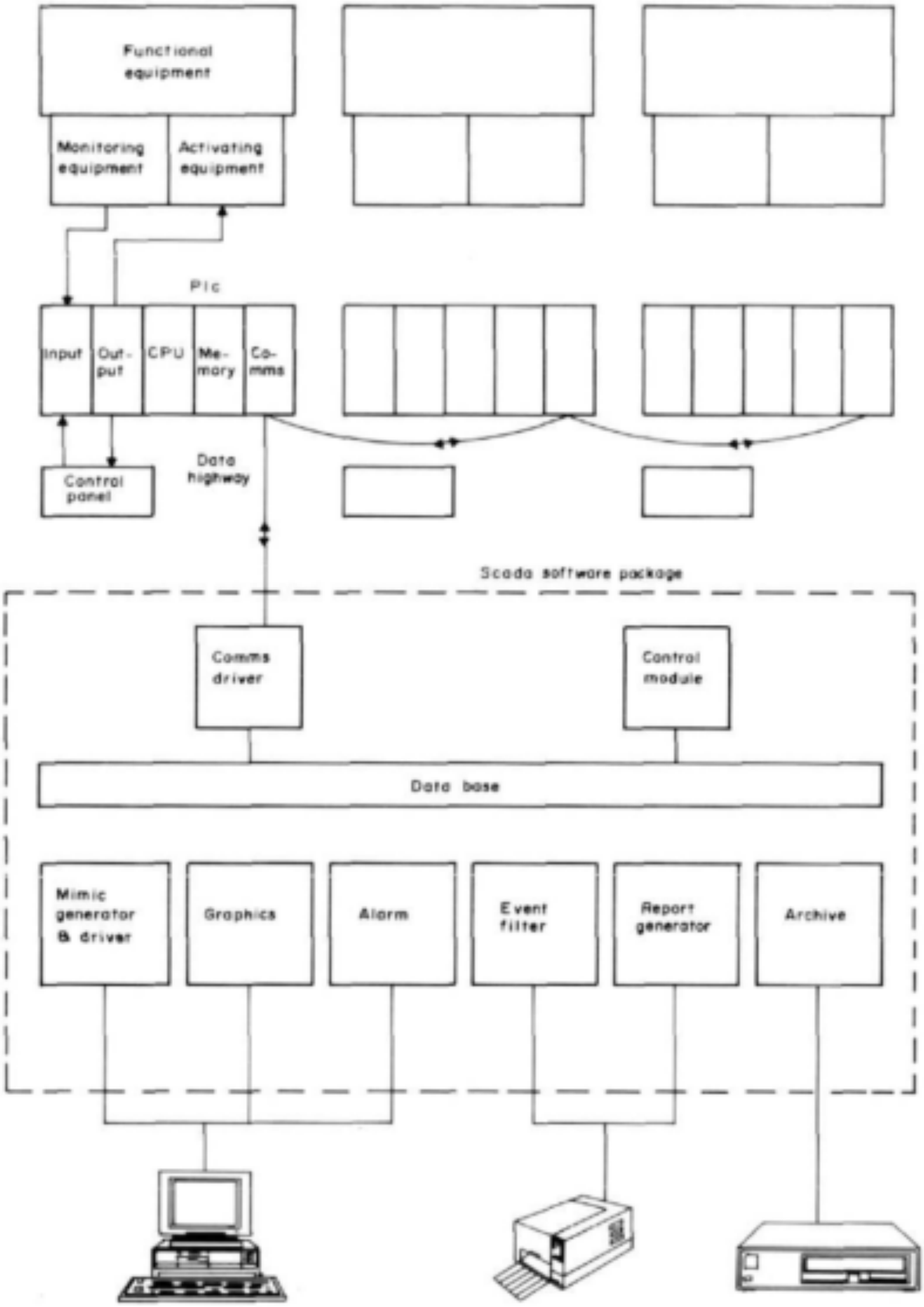


Figure 18.4
Diagram of SCADA system

not do anything by itself. The data just sit there and wait for other software modules to read what is already in the data base or to overwrite existing data with new values. For each and every input and output variable on the works there is a record in the data base. This record contains a reference to the variable, information on which I/O module or which PLC handles it, its most recent value, etc. If it is an analog variable this record would also contain scaling factors for it, alarm points beyond which the variable is not permitted to go, etc.

Surrounding the data base, is a set of functional software modules.

- The most important of these is the communications driver module. This module scans all the PLCs continuously for the values of all their Input variables and writes them into the appropriate records in the data base. The communications driver also scans those records in the data base which are associated with final actuators and outputs the values to the PLCs which are responsible for the particular devices. In summary it can be said that it is the task of the communications driver to see to it that whatever happens on the works, is reported in the data base as soon as possible and *vice versa*. In practice the response time of such systems is on the order of a few seconds.

- Another module of the SCADA system would be the mimic generator and driver. During original system definition the generator part of this module would have been used to create computer screen mimics of the plant and the icons of the parts which can change status would have been linked to the relevant record in the data base. When the system is running, the mimic driver part of this module monitors the data base and causes the icons to change colour or shape when the value in the data base changes. In this way the operator can see what is happening on the works.

- The graphics module can collect a string of values of an analog variable over a period of time and display it as a graph on the computer screen.

- The alarm module checks for any binary condition which has been defined as an alarm condition or any analog value which exceeds a given alarm limit. Whenever this happens, an audible alarm is activated and the computer screen displays information about the condition which caused the alarm.

- The event filter looks at all incoming data and, on a set of rules provided, decides which events are important enough to be logged on the logging printer.

- The report generator is triggered at regular intervals (perhaps every shift) to print a summary report of the past shift's data on the report printer (which may be the same as the logging printer).

- The archive module takes regular readings of all values and stores them on disk for future use.

- If decision-making tasks are assigned to the computer, the control module takes the necessary data from the data base, analyses them, makes its decision and feeds its response back into the data base.

All these modules time-share the computer's central processor, but the processing of the data by each module takes place so rapidly that it appears as if all the tasks are running simultaneously.

Many variations on the above theme are possible. The control panels at the PLCs are optional. If there is only one functional group of equipment, the computer can itself be fitted with a set of I/O modules and the PLC can then be eliminated. The tasks given to the SCADA system and their relative importance will vary widely from system to system.

18.6 COMMUNICATIONS

There have been many references to electronic binary and analog signals above. The subject of electronic signalling in more detail is dealt with here.

18.6.1 BINARY SIGNALS

Binary signals are usually transmitted as a DC voltage of nominally 12 or 24 volts with 24 volts being the more common. Sometimes 120 or 230 V AC is also used, but this involves an element of danger and is used only when special circumstances dictate it.

Binary signals can be transmitted successfully for distances of up to hundreds of meters.

18.6.2 THE 4 - 20 mA CURRENT LOOP

This method of communication is commonly used to transmit single analog signals.

This system does not use a voltage to represent an analog value. Instead, a small electric current, which is made to be proportional to the value which must be transmitted, is used. The main reason for this is that if a voltage signal was used, the resistance of the signal wires will cause a voltage drop and the reading will thus be affected. With a current signal, the current which enters at one end of the signal wire must exit at the other end, irrespective of the resistance of the wires. Such a system is therefore capable of much greater accuracy than a voltage signal system. The system also uses a "floating" zero. By this is meant that the bottom end of the scale of the variable which is being transmitted, is not represented by zero current. Instead, 0% is represented by a current of four milliamp and 100% by a current of 20 milliamp. This allows the easy detection of broken circuits and, since there is always a small current flowing, it can be used to supply the power necessary to drive instruments which consume only such small amounts of electricity. Such instruments are known as loop powered devices.

These signals can be used for distances of up to several hundred metres.

The 4 - 20 mA signal is used for all types of transmitters, be they flow transmitters, level transmitters or whatever. It is also used by all the major suppliers of instruments and all the major suppliers of PLCs and process control computers have modules available which can read or generate these signals. It is thus possible to freely "mix and match" the instruments from various suppliers with PLCs from other suppliers, etc.

18.6.3 DIGITAL DATA HIGHWAYS

A major disadvantage of the above-mentioned communication methods is that a separate wire or pair of wires is necessary for every signal to be transmitted. For larger numbers of signals, this quickly becomes prohibitively expensive. In fact, the effective limit on the distance over which binary and analog signalling is used, is set by cable cost considerations rather than by limits on the technology.

The solution is to wire the signals to a local PLC or similar device, which converts them into digital form and which has the ability of time-sharing these signals on a digital data highway. Digital data are quantified in terms of bits. A single binary variable requires one bit to represent it. A typical analog variable requires about ten bits. A very modest data highway would have the ability to transmit perhaps 20 000 bits per second. This corresponds to about 2 000 analog readings per second which is more than enough to handle all the data requirements of quite a large water works.

The actual transmission of the digital signals can take place either by means of copper wire cable or by means of fibre optic cable. It is possible to base reliable systems on either technology. Both copper cable and fibre optic cable data highways are capable of working over distances of up to a few kilometres. This is enough to be able to connect all the equipment of a water purification plant with its associated pump stations, etc. to a central SCADA computer.

18.6.4 RADIO

In the case of a water distribution system which includes remote reservoirs, pump stations, etc. the situation calls for a long-range communication system. The most common solution is to use radio links.

This link now becomes part of the data highway. Radio links are limited in capacity to about 2 400 bits per second with an upper limit of about 9 600 bits per second. This is somewhat slower than the previously mentioned figure of 20 000 bits per second. If care is taken not to transmit any unnecessary data, it is nevertheless possible to monitor and control a very large water distribution system with a radio communications system, without overloading the communications channel.

18.6.5 TELEPHONE LINES

For the long-range communication situation, the use of telephone lines rented from the public service authority can also be considered. If continuous or frequent communications are required, a line can be permanently leased. If the frequency is low, then a dial-up system which uses the standard existing telephone infrastructure, can be considered.

18.6.6 LIGHTNING SURGE PROTECTION

The lightning frequency in South Africa (and on the Transvaal Highveld in particular) is significantly higher than in most other industrialised countries. This requires that all electronic and electrical equipment be protected against surges. The surges are induced in the cables which connect pieces of equipment to each other and, the longer the cable, the larger the induced surge. This applies to all cables, whether they be power supply cables or signal cables of whatever type. All points where cables enter pieces of equipment therefore need to be protected.

Lightning induced surges will very occasionally exceed 50 000 amp, but normally it is not economically justified to protect equipment to such a high level against such a very rare occurrence. In practice a surge protection level of 10 000 amp is found to be adequate for cable runs of typically 100 m. For shorter cable runs, the protection level can be reduced in proportion to the length of the cable. This is required only for cable runs externally to buildings. All buildings which use steel in their wall structures or have metal roofs, act as electromagnetic shields and additional surge protection is normally not required for cable runs which are entirely inside the structure. Structures which are exposed to direct lightning strikes such as radio telemetry antennas, require special treatment which, with current technology, can give a very high level of protection even against direct strikes.

18.7 RELIABILITY AND MAINTENANCE

The design of the instrumentation for a water purification plant is usually based on a primary set of equipment which functions in a convenient, but cost-effective way. Although the mean time to failure of an individual piece of equipment is many years, many such pieces are used in a typical installation. It is therefore necessary to allow for the occasional failure of an instrument, a PLC, etc. and to make back-up arrangements.

18.7.1 SYSTEM RELIABILITY

Compared to many other industrial processes, the following factors make the design of an adequate back-up system relatively easy.

- Water is a relatively benign process medium
- Water treatment processes are rather slow
- Water processing can generally be stopped at short notice.

For these reasons, a back-up system does not require duplication of equipment. One simply reverts to a simpler, albeit less convenient, control mode. It is, however, necessary to design this possibility into the configuration of the system. In a SCADA-controlled system, one would, for instance, not put all the decision-making functions into the central computer. It is better to delegate this as far as possible to the individual PLCs which control a section of the plant. Such a PLC would typically also have a control panel to assist with operator interfacing. Should the SCADA system fail, operators can revert to controlling the plant from the local control panels, still with the assistance of the PLCs. Should even this fail, it must still be possible to revert to completely local manual control.

18.7.2 EQUIPMENT RELIABILITY AND MAINTENANCE

A reliable system depends on reliable equipment. This does not only mean that the equipment should be inherently reliable. It also means that the instrument shall be:

- suitable for the application
- suitable for the environment within which it must operate
- properly installed
- properly calibrated
- used with good cabling practice
- be protected against electrical surges
- properly maintained.

The last-mentioned item justifies further comment.

Electronic equipment does not contain many moving parts. The temptation therefore exists to conclude that it does not require regular maintenance. Experience, however, has shown this to be very wrong. Unmaintained installations invariably fail after a few years.

- Components age and cause calibration drift.
- Dirt blocks instrumentation openings.
- The few mechanical components which

there are, like cooling fans, wear out and fail.

- Insects, water, dirt, etc. find their way into the innards of the equipment.
- Terminals work loose.
- Cable ends fray.
- Rats eat the cable insulation.

It is therefore necessary that such installations be visually inspected and functionally checked twice a year. The best person to do this is usually the contractor who supplied and installed the equipment. The cost of the service will amount to a few percent per year of the value of the original installation, but this expense is amply justified.

19

Design Examples

P J Pybus

The preceding chapters of this book have dealt with the individual operations and processes that are required for the treatment of the water. The purpose of this chapter is to integrate the design of the individual components into a cohesive whole that will produce the required output at the correct quality in an economically acceptable manner.

It is important to remember continually in the design stage that the works will have to be operated by people. This implies that the works should be designed for ease of operation; all equipment requiring manual input should have been designed for spatial comfort and ease of effort; safety of operating staff should be a paramount thought, particularly in view of the onus placed on the designer by the Occupational Health and Safety Act (Act 85 of 1993). The Act should be studied by the designer as it covers the working procedures in different circumstances.

Design is a team effort. The team will comprise the specialists in the different disciplines of water science and engineering as well as representatives of the owners of the works. The people who will have the most to do with the works once it has been commissioned and handed over will be the operator and his staff. The response of the operators to a well thought out layout, and preferably one to which they have been able to make a contribution, will be well worth the effort of bringing someone from the operating staff onto the design team.

It is important to keep good minutes of the design meetings, recording not only the decisions taken, but also the reasons therefore as the designer will ultimately be held responsible for the final outcome. Where possible the cost consequences of any design decisions that depart from the feasibility study should be noted. It frequently happens that these have a significant effect on the final costs, not only the capital cost but also the operating costs. Such decisions may have to be reviewed if the cost changes become too great.

19.1 LAYOUT AND GENERAL CONSIDERATIONS

In the design example that follows it is assumed that the feasibility study will have provided the motivation to build the works as well as have determined the design capacity that will be required to meet the estimated demands for an economically determined design horizon. It is also assumed that the required process will involve only the dosing of chemicals, sedimentation, filtration and disinfection.

19.1.1 GENERAL

The layout of the works is dictated largely by the terrain, which in turn affects the hydraulic design. The steeper the site, the simpler the hydraulic problems, but the greater the complexity of the roads and the site works.

19.1.2 HYDRAULICS

The layout of the pipes and channels should be such as to enable the greatest flexibility of working possible without incurring wasteful expenditure. As a general rule, should one of the units be taken out of service for any reason, the conduits should be sufficiently large to handle their proportion of the incoming design flow without any overflow occurring. It should be possible to by-pass any unit at will.

Each unit should be capable of isolation. Each unit should be capable of being drained, preferably without the use of pumps. If a facility is designed to recycle the washwater and sludge supernatant, the water that is scoured from the units should pass into the recovery system. Care should be taken that the rate of scouring does not overload the recovery system.

The designer needs to examine the hydraulic conditions under all possible flows, not merely the maximum overload rate, but also the conditions that would apply in the event of no flow. This is particularly important in the case of pipelines carrying solids. These should be designed so as not to deposit the solids under normal conditions and with the possibility of flushing the line and creating the conditions that would resuspend the solids.

19.1.3 SOLIDS HANDLING

The handling of solids needs to be considered carefully with the layout. The routes of the trucks delivering the chemicals to store should be as short as possible, steep gradients should be avoided to enable the trucks to be off-loaded at the correct level and according to the correct method of unloading. The routes should be straight through to avoid any reversing movements, especially for the larger vehicles. If reversing is imperative, adequate manoeuvring space should be provided. Curves should be designed in accordance with the latest highway manuals for the heaviest class of vehicle likely to deliver materials to the site.

The layout of the chemical stores area should facilitate the mechanical handling of the chemicals, using trolleys and fork lifts. Sufficient space should be provided to allow these to turn and reverse, even if in the initial stages no mechanised handling is to be provided. It will be demanded in due course.

Consideration needs to be given to the manner of ultimately disposing of the sludge. It could be taken in a dry form off the site to landfill, it could be dried and then disposed of in a landfill close to the site or left in a slimes dam. Whatever means is selected it will have to be done in such a manner that it does not cause pollution or degradation of the environment, especially as the result of seepage of acid leachates.

If off-site disposal is considered, then the road layout must take into account the traffic that this will generate, as well as the mechanical handling facilities that will be needed. The overall layout of the works will therefore be strongly influenced by the need to be able to handle the flow of solids.

19.1.4 ACCESS

The access provided for the people that will operate the works will influence the manner in which they approach and carry out their duties. Too spread out a layout, will cause the operators a great deal of wasted time, lead to the omission of perhaps vital calls around the works and result in malfunction in some respect or another.

The route which the operator follows on the daily round of inspection should be considered when planning the layout. This will vary from case to case but will generally follow from an inspection of the raw water to assess whether any change in the dosing rate is required, to the chemical house and dosing point to check the actual dosing rates and then past the clarifiers to the filters and the disinfection point. A compact layout will assist the operator greatly.

Traffic problems for the handling of the solid materials have been touched upon above. Provision should be made for road access of the people who work on and will visit the works. Security check points will be needed to control the access of visitors both into and around the works.

19.1.5 Construction and layout

The way in which the works is to be constructed must also influence the layout of the works. This means allowing space for construction vehicles between the units, taking into account the stability of the *in-situ* material, as to whether adjacent excavations will collapse into one another and a host of related problems. These considerations which are of a temporary nature, should not be allowed to outweigh the need to produce a layout that will assist the operator.

A factor not often given the consideration that it deserves is that of the cable routes. On completion of the works the cables should be accessible in the event of a fault. They should not be buried so deep that it requires a major excavation to recover them. If there is any significant filling required through which cables must pass, the filling should be completed before the cable laying. It should also be remembered that the open trenches interrupt traffic flow around the site, frequently just at the time as the erection of the mechanical components is in progress. Careful planning and adjustment of the layout can do much to minimise this type of problem.

19.1.6 Selection of materials

This is an appeal to the designers to consider the selection of materials on the grounds of cost, corrosivity of the environment, required mechanical strength and the required life of the item.

Cost is perhaps the most important consideration, not only in terms of initial cost, but also of subsequent maintenance and upkeep. In selecting materials and finishes the future costs of maintaining these need to be considered.

Pay attention to the flow of materials, mainly the chemicals to be used on the works and the off-loading arrangements necessary. These deliveries will be made weekly or monthly and awkward reversing movements or difficulty of access to the loading bay will impede the general operation of the works and make for poor designer-operator relations. Make the layout of the roads simple, with adequate reversing space, if this is necessary; design the curves for the longest vehicle likely to use them and if possible, allow a complete circuit without the need to reverse.

The operator will have to move around the complete works daily and probably much more than once a day. Layouts that look well on a drawing, with plenty of space, may well be creating a problem for the operator, and in consequence the designer. Keep the layout of the units as compact as is practicable. If it is possible, calculate the route which an operator will normally take around the works and ensure that it can be easily traversed. A logical route would be to visit the clarifiers, followed by a call at the chemical house to check on the dosing rates and thereafter to inspect the filters and attend to the backwashing. If the route of the operator can be simplified, the designer will have gone a long way to ensuring that the works will be well looked after.

As a corollary to the foregoing it is frequently wise not to construct any paths until after the works have been commissioned and routes established. It so often happens that the design provides for neat geometric paths which do not follow the natural walking direction and so are seldom used.

The use of bridges between units may save the operator a great deal of time each day and should be considered.

19.2 DETAILED LAYOUT

The works as designed have to be constructed as well as operated. Working space applies equally to both phases and must be allowed for. Situations which should be considered are:

- Space for bolting up flanges, particularly in chambers
- Space around hand wheels of valves - are they close to any other flange?
- Is there space for pipe fittings to be lifted out of position?
- Can a person gain reasonable access to a pump or other mechanical equipment?
- Is there space around to lay out tools and any parts that may be removed from an item of mechanical equipment?
- Are there any lifting movements that require the use of lifting equipment?
- Is there space for the operator of the equipment to perform any manual tasks associated with the equipment?
- Is there space to strip the shutters from conduits or channels?

19.2.1 SAFETY

The promulgation of the Works and Public Safety Act places the responsibility for the safety of the works squarely on the shoulders of the designer. Should an accident occur and injury or death result the designer may be held liable.

The designer needs to consider the layout of the works very carefully and try to imagine potentially dangerous situations for both the public and the operating staff. One method of doing this is to conduct a HAZOP study in which the designers, operators, owners and maybe also the contractors and suppliers, examine every element of the works and determine what could go wrong, and take measures to alleviate the problem. This type of analysis should be done at an early stage of the design phase and again towards the end, and well before any become committed in concrete.

19.2.2 OPERATION

The design of a works will ultimately be judged by:

- The ease with which the works can be operated.
- What procedures must the operator go through in order to effect any changes to the process, to desludge a clarifier or to backwash a filter?
- Are there sufficient instruments and meters?
- Does he know at what rate he is washing?
- What is the head loss through the filters and how is this affected be the flow rate?
- Is backwash rate controllable and how is the operator to effect this?
- Are automatic backwash procedures necessary and if not, are they easy to follow?

- Are the valves geared for ease of operation? Do they need to be?
- What operations does the operator have to perform to draw sludge from a clarifier. Is the rate, frequency and duration of each decanting operation adjustable?
- How is chemical feed rate controlled and more importantly; measured?
- Can all flow meters be checked for accuracy by simple displacement?

The designer needs to visualise the structure that he is designing, seeing the building in its finished state while it is still on the drawing board. This is an art that is acquired by practice and through experience and is an essential tool for the successful designer.

19.3 DESIGN EXAMPLE

A water works is required with a nominal treatment capacity of 20 000 m^3/d which is the estimated sustained peak demand at the design outlook of the scheme of which it forms part. The sustained peak factor is 1,45.

The raw water will be delivered from a dam some distance from the water works through a 500 mm diameter steel pipeline. The pipeline has been marginally oversized to provide additional head at the outlet for in-line static mixing of the chemicals.

The rate of flow through the water works will be controlled by means of a valve on the outlet of the pipe.

19.3.1 INVESTIGATION

A process of experimentation has been undertaken over a period of 12 months to determine:

- the optimum system of treatment,
- the most economic coagulation,
- the required rates of dosage,
- any additional chemical treatment required, and
- the required rates of dosage.

19.3.2 PROCESSES AND OPERATIONS

These consist of:

- coagulation
- flocculation
- sedimentation
- rapid gravity sand filtration
- pH correction
- disinfection.

19.4 COAGULATION AND CHEMICALS

The water is adequately buffered and no post-liming is required.

19.4.1 CHEMICAL DOSAGE

The chemical dosages were to be :

- Alum: average 25 mg/ℓ
  peak 60 mg/ℓ for 8 weeks
  continuously
- A polyelectrolyte could be used at a dosing rate of 0,5 mg/ℓ rising to 1 mg/ℓ for 8 weeks of the year.
- A dosage rate of 50 mg/ℓ of lime is required to adjust the pH to give a positive Langelier Index.

The sustained peak flow demand occurred at the beginning of summer prior to the onset of the summer rains. The effect of the rainfall was to reduce the demand whilst increasing the turbidity of the raw water.

Flow data	Average	Sustained peak
Flow (m^3/d)	14 000	20 000
Flow (m^3/h)*	583	833
Flow (m^3/s)**	0,162	0,231

* required for sizing sedimentation tanks and filters.
** required for hydraulic calculations.

19.4.2 CHEMICAL DOSING RATE

This consists of:

Alum:

Normally @ 30 mg/ℓ = 30 x 15 kg/d
= 420 kg/d
@ peak = 30 x 20 kg/d
= 600 kg/d

Polyelectrolyte:

Normally 0,5 mg/ℓ = 0,5 x 14 kg/d
= 7 kg/d
@ peak = 1,0 x 14 kg/d
= 14 kg/d.

19.4.3 CHEMICAL STORE SIZING

The chemical store can be sized as follows :

Weekly alum demand = 4 200 kg/week
Weekly polyelectrolyte demand = 50 kg/week

Alum store:

Due to transportation problems it may take from 3 to 6 weeks from date of order to delivery.

Size of delivery: 5 t, 10 t or 20 t trucks. Hence allow for 20 t deliveries plus reserve of 2 weeks, should deliveries be delayed.

The required capacity of store = 6 x 4,2 t
= 25 t

1 pallet carries 1 ton or 20 x 50 kg bags
Footprint of pallet = 1,00 x 1,2 m.

Allow 400 mm space between pallets. Stack 2 high, on 2 sides of store.

Length required = 7 x (1,0 + 0,4) + 0,4
= 10,20 m

Allow 3,0 m space between pallets for forklift.

Breadth required = 2 x (1,2 + 0,4) + 3,0
= 6,20 m

Polyelectrolyte store

The selected polyelectrolyte has a shelf life of 3 months.

Delivery can be effected within 4 weeks of order.

Provide storage for 1 month @ peak rate. i.e. 30 x 14 kg = 420 kg

Supply in 25 kg containers.

Require space for 425 ÷ 25 = 17 containers

Diameter = 0,5

i.e. 3 rows, 3 deep on both sides of chemical store

Add 2,6 to length of store.

Store floor area = 12,70 x 6,20 m.

19.4.4 CHEMICAL MAKE-UP AND DOSING

Alum
Provide 10% solution strength for the alum with 24 hour make-up tank for peak condition.

Bulk density of alum = 750 kg/m^3

Volume of tank required

$$= \frac{20\ 000 \times 60}{1\ 000} /0,1/750$$

= 16 m^3

Provide 2 tanks each 8 m^3 active volume, with 200 mm dead storage for sediment accumulation, etc.

Free board to be 300 mm.

Dimensions of tank = 2,0 m x 2,0 m x 2,5 m

Polyelectrolyte
Make-up tanks and mixing equipment and dosing pumps to be provided by chemical supplier.

19.4.5 DOSING PUMPS

The alum dosing pumps are considered here.

Provide two working pumps to handle peak flows plus one standby.

Maximum pumping rate = 16 m^3 over 24 h
i.e. 11,1 ℓ/m for 2 pumps.
Specify 6 ℓ/min.pump @ 100% stroke.

Flash mixing
Provide G value > 600 s^{-1}
Velocity in inlet pipe = 1,18 m/s in 500 mm diameter pipe.

Reduce size to 400 mm for static mixer:
V = 1,84 m/s at sustained peak.

Provide 2 orifice plates

Distance apart = $L/_{do}$ in range 5 to 7.
If first trial of orifice diameter
= 200 mm
then distance apart = 6 x 0,2 = 1,2 m

19.5 FLOCCULATION

Examples of flocculation and flocculators are given in Chapter 8.

19.6 SEDIMENTATION

Provide 2 tanks each to treat 10 000 m^3/d

Floc size = 1,0 mm
Mean density = 1 006 kg/m^3
Water density (20°C) = 998 kg/m^3
Viscosity = 10^{-3} kg/ms

Theoretical settling velocity, Stokes' Law,

$$= \frac{9,81\ (1\ 006 - 998)(0,6 \times 10^{-3})^2}{18 \times 10^{-3}}$$

= 1,57 mm/s
= 5,65 m/h

Flow dispersion value - Circular radial flow tank (Fig. 9.3)
Take as 0,3

$$\text{Then area of tank} = \frac{10\ 000}{0,3 \times 5,6 \times 2,4}$$

$$= 246 \text{ m}^2$$

$$\text{Diameter of tank} = \sqrt{246 \times 4/11}$$

$= 17{,}70$ m
Say $18{,}0$ m

Provide 2 hours minimum retention

$$\text{Thus volume per tank} = \frac{10\,000 \times 2}{24} = 833 \text{ m}^3$$

$$\text{Side water depth} = \frac{833}{(\pi/4 \times 18{,}0^2)}$$

$= 3{,}28$ m
Say $3{,}30$ m

$$\text{Weir loading rate} = \frac{416}{\pi \times 18}$$

$= 7{,}4 \text{ m}^3/\text{h.m}$ of weir
<10

Provide 1:12 floor slope.

19.7 RAPID GRAVITY SAND FILTER

Provide single-media, downflow filters

Filter rate not greater than 8 m/h

Flow control is upstream, therefore division of flow between filter beds will be by means of weirs.

Note on design of filter inlet launder:

It is important to maintain a near constant water level so that the depth of water over the division weirs is constant, thus improving flow division. If the invert of the launder is horizontal, the velocity, and hence the velocity head, reduces as water is drawn into each filter.

Therefore, in order to maintain a constant velocity in the channel, keep the depth proportional to the flow by stepping the invert of the channel.

Provide four filters, with central gallery.

Backwash rate to be 20 m/h with preceding air scour rate.

$$\text{Area of each filter bed} = \frac{20\,000 \times}{6 \times 24} \; \frac{1}{8{,}0}$$

$= 17{,}4 \text{ m}^2$
Say $18{,}0 \text{ m}^2$

Provide beds $4{,}0 \times 4{,}5$
Backwash rate $= 18 \times 20 \text{ m}^3/\text{h}$
$= 360 \text{ m}^3/\text{h}$
$= 100\ \ell/\text{s}$ - reasonable

Provide central filter gallery.

19.8 DISINFECTION

Maximum chlorine demand - by measurement

$= 2 \text{ mg}/\ell$

Maximum rate of dosage

$$= \frac{2 \times 20\,000}{24 \times 1\,000} \text{ kg/h}$$

$= 1{,}7$ kg/h

Size and specify chlorinator to dose 2 kg/h

Note: Legislation Hazardous Substances Act 1973 (Act 15 of 1973).

20

Cost Estimating and Evaluation

P J Pybus

The purpose of this chapter is to introduce the reader to the need for, degree of accuracy required and an outline of the methods used as well as some of the pitfalls in preparing cost estimates. A discounted cash flow method of comparing the capital, operating and maintenance costs of different schemes is briefly presented.

20.1 PROJECT CYCLES AND FUNDING AGENCY

The project cycle runs from the first awareness by the owner of the need for the project, through the first application to a lending agency for funds, the appointment of the engineer, the receipt of the preliminary and feasibility reports, the examination of the project for technical, social, institutional and financial soundness, the design phase, the calling for and adjudication of tenders, construction, commissioning and subsequent operation and maintenance of the works.

The funding agency will need estimates of cost at different stages of the project as well as information on the timing of the expenditure, i.e. cash flow, in order to have the funds available at the right time. The information will be used to evaluate the project for relevance and affordability. The cost estimates will have to be prepared with increasing precision at each successive stage.

The stages will be at the initiation of the project when application is made for funds; at the preliminary report stage when the application is to be processed for the first time and an assessment is made as to whether to proceed with the project or not; at the feasibility report stage when the loan will be formally approved and then the last estimate will be based on the final schedule of quantities as tenders are called for, in order to check that the tenders have been reasonably priced.

20.1.1 EFFECTS ON TARIFFS

The cost estimates will be used at the feasibility stage to determine the effect of the project on the cost of water delivered to the customer as this in turn will have an effect on the political acceptability of the scheme and the need to apply for subsidies.

20.1.2 COMPARISON OF ALTERNATIVE SCHEMES

During the preliminary report and feasibility report stages of the project the costs of alternative treatment systems, possible alternative locations of the works, etc. will be used to evaluate the alternatives. Such financial comparisons take into account not only the capital cost of the project but also the costs of operation and maintenance.

20.2 ACCURACY OF COST ESTIMATES

As the project progresses through the cycle, the precision with which the cost estimates are prepared increases. At the inception of the project an order of magnitude estimate may suffice. By the time tenders are called for, the financial controllers will require estimates that do not deviate from the final cost by more than a few per cent.

It is important to remember that the first preliminary estimate of capital cost is the one that will be remembered by the financial managers and decision-makers. This is the figure they will recall at the end of the project and compare with the final cost. Misunderstanding, loss of confidence and acrimony result from any significant increase in this amount. Such increases may well result from a change in the location of the works or its size as the result of changed circumstances encountered during the feasibility studies. The changes of scope that lead to an overrun tend to be overlooked under these circumstances. It therefore pays to define very closely at every stage what the estimates cover and to document carefully all changes in scope.

A point to remember is that there are fundamental differences in the thought processes of engineers and scientists on the one hand and financiers and accountants on the other. "Estimate" is understood by a financier to mean the amount of money that will be spent in any fiscal period, while to an engineer it represents his best calculation of the cost of a project or component of it. Both interpretations of the word are correct. Keep these differences in training and thinking in mind when discussing costs.

20.2.1 MARKET FORCES

The state of the economy will influence tender prices significantly. Whilst there is a downturn and a shortage of work, contractors are forced to cut prices to the bone. Once the volume of work that is being put out to tender exceeds the capacity of the industry to handle the work, prices will tend to rise. The change in price is out of all proportion to the actual swing in availability of work and may lead to increases of as much as 30% in the space of a few months. It is not possible to predict the precise timing and magnitude of this type of swing but caveats should accompany any estimates when this type of situation is likely to arise. The changes in the reverse direction are not as sudden or as dramatic and the consequences for the financing authority are not as severe. Financiers are nevertheless intolerant of any deviations in estimates.

20.2.2 PRICING PROBLEMS

The engineer may be required to provide a reasonably accurate cost estimate at a stage in the project when it is scarcely defined and perhaps even the site has not been finally determined. Contractors tendering for work are called upon to price final designs against properly prepared schedules of the work to be done. Despite this and the fact that in a competitive situation close pricing is expected, very wide-ranging prices are often received as shown in the tabulation.

Tender	A	B	C	D	E	F	G
Relative price	1,00	1,02	1,13	1,28	1,29	1,58	1,88

When compared with this range for clearly defined work it is a matter of surprise and congratulation that most estimates are sufficiently accurate for their purposes.

20.3 GLOBULAR COST ESTIMATES

Once the need for the project has been recognised, a cost estimate will be required to initiate the project cycle. At this stage the capacity of the required works can only be guessed at, the location of the works has probably not been established and the processes that are to be used are still only a surmise. However, even under these circumstances, a fair estimate can be made.

20.3.1 CONCEPT OF SCALE

In general terms the unit cost i.e. cost/Mℓ.d of design capacity of a water works will decrease as the scale or size of the scheme increases.

A simple formula can be used to relate the cost of works of a similar type and nature for different capacities.

$$C = K.Q^{n}$$

where

C = the cost of scheme of capacity Q

Q = the capacity of the scheme expressed as the throughput and not as a linear dimension.

K = a factor of cost which makes adjust ments for escalation, location, ter rain etc. It needs to be recalculated periodically.

n = a scaling factor related to the type and nature of the works under consideration.

The value of n will lie in the range of between 0,6 and 1,0. The former is generally applicable to pipelines where the effects of scale are the most marked whilst the latter is more appropriate for items that are modular in nature, i.e. pumps or desalination units.

Knowledge of the costs of at least two works of a similar nature, but not size, is required to calculate the values of n and K.

20.3.2 FACTORS AFFECTING THE COST OF THE WORKS

There are a number of factors, both within and outside the control of the engineer, that can influence the cost of any work put out to tender.

The nature of the works and the different process trains that are to be employed will naturally affect the cost estimate and when comparing the cost with that of another works it is important to ensure that only the costs of similar works, incorporating similar process trains, are used. If such estimates are not readily available it may be necessary to go back to first principles to establish a realistic cost. (See the section on feasibility estimates.)

The location of the works will affect the general rates that are applicable. These tend to rise with distance from major centres. Local factors such as difficulty of access, as distinct from the possible need to construct access roads for the project, can have a significant influence on the rates that will be tendered and ultimately the overall costs. These need to be taken into account in the preparation of the first preliminary estimate, even if only subjectively.

Escalation is likely to remain an ever present factor in the costing of engineering works. Therefore the date when actual construction will begin has a major impact on the cost. Delays in the approval of the project, an extended design period and a long construction period will increase the final cost.

An estimate has to be made of the rate of inflation to use. It is probably best to use the current rate.

20.3.3 ENGINEERING JUDGEMENT

Engineering judgement is difficult to define but it must take over when science and circumstances preclude a documented decision. It may not always be possible to define and quantify all the parameters needed to prepare a preliminary cost estimate and yet an answer must be given. The engineer must therefore have a good idea of the cost of works of certain sizes at any time and be able to adjust these in a mental process for factors such as distance, access and time of implementation. Engineering judgement will also come to the fore to prevent gross errors in an estimate. It is an attribute that generally only comes through long and sometimes bitter experience.

20.4 PRELIMINARY COST ESTIMATES

In any project there is generally more than one solution to a problem, each with differing capital and operating costs. The differences may be in type or location of the works or in the phasing, with augmentation at an intermediate stage compared with one single construction. Reasonable and informed estimates of these costs are needed in order to eliminate all but the most favourable schemes.

20.4.1 NEED FOR DATA BASE

An engineering office preparing designs of works needs to establish and maintain a good data base of the prices and costs of water works. The work involved in doing this on a continuous basis is far less than in periodically having to go back to old records and delve for the contract prices of work that may not be relevant to the present project.

The data base should record the basic information such as:

- Client
- Type of works or extension
- Whether the construction was original or an extension of an existing works
- Capacity of works units incorporated in the works
- Comments on extent of site works for that project
- Operating rates of the units
- Costs of each unit
- Date of completion
- Locality of works.

The data base should be updated regularly as each new project reaches its successive stages of implementation.

20.4.2 SIZE OF WORKS

The steps taken to determine the size of the works, be it a new works or an extension, will have the greatest impact on the cost of the works. The optimum design outlook has been found to be a function of the discount rate applicable at the time. Consequently, once the design outlook has been determined, the factor affecting the size of the works is the rate of growth in demand that is assumed. This rate cannot be calculated too carefully as it has a profound influence on the cost of the works.

20.4.3 NATURE OF WORKS

Clearly the nature of the works will affect the overall cost significantly. A straight forward type of works comprising chemical dosing, sedimentation, filtration and disinfection will have a very different basic cost than one with only direct filtration. Distinction needs to be made in data base for these differences. Alternatively the cost record should be for individual types of treatment unit. In this case allowances will have to be made for interconnecting pipework, site works, etc.

20.5 ESTIMATES FOR CIVIL CONSTRUCTION

20.5.1 SITE WORKS

The need for and the nature of the site works must be allowed for in the preliminary estimate. These may vary and provide for general earth-moving and site-levelling, as well as the completion activities of constructing site roads, drainage and final terracing and grassing. The standards of finish to be adopted, i.e. whether the roads will be surfaced or gravelled, should be allowed for and noted in the scope of what the estimate covers.

20.5.2 INTERCONNECTING PIPEWORK

The interconnecting pipework usually forms a significant part of the cost estimate and is frequently the cause of error in the estimate, either by omission or due to underestimating the actual cost. Substantial allowances need to be made for the cost of valves and fittings which occur more frequently on a water works than in a normal pipeline. For a preliminary estimate it may be sufficient to allow a straightforward percentage of the entire works.

20.5.3 MECHANICAL AND ELECTRICAL

The mechanical and electrical components of a scheme may, for a preliminary estimate, be calculated as a percentage of the civil works or else repriced from existing data. A more prudent way of doing this is to obtain a cover price from a company supplying and installing water treatment equipment. Most such companies are generally very co-operative and helpful in furnishing information. However, the information should be used carefully. If you ask for the price of a chlorinator, for example, the price you will be given will cover the cost of supplying and maybe installing the chlorinator, but will not include the gas mask and safety equipment, the extractor fan and the drum change-over panel. It is the responsibility of the person preparing the estimate to ensure that everything that will be subsequently installed is included in the original enquiry or else to make suitable allowances for what is not included.

20.5.4 COMPLETENESS OF ESTIMATE

Some of the worst estimation errors arise due to from the omission of significant items from the total estimated cost. Examples of these are the construction of the roads to the site and the provision of power from a distance. Ancillary structures such as off-loading facilities, chemical stores, offices, laboratories and workshops, should be included in the estimated cost.

20.5.5 INDIRECT COSTS AND ALLOWANCES

Most engineering estimates include allowances for items covering contingencies, escalation and engineering fees. These allowances are frequently based on a percentage of the net estimated cost of the works. The magnitude of the allowance should in all cases be derived from the actual situation. Ten per cent is the most commonly used value for contingencies. Once an escalation rate has been established and the date for the start of construction, and the construction period have been settled, calculation of the percentage to be allowed for escalation is straightforward.

$$F = R{*}T_1/12 + R{*}T_2/12/2$$

where

F = the factor to be applied for calculation of the escalation amount, expressed as a percentage of the net estimated cost

R = escalation rate expressed as a percentage per annum

T_1 = the period in months from date of calculation until construction commences.

T_2 = the period of construction in months.

Engineering and professional fees can be calculated from statutory fee scales or from agreements reached with the owners of the scheme. A common error is to allow a fixed percentage for the costs of any site staff that are to be appointed. A realistic method of calculation is to take the typical cost of the class of person per month multiplied by the period of construction.

20.6 FEASIBILITY ESTIMATES

A very much higher order of accuracy is required at the feasibility stage for it is on this figure that final approval for the expenditure of the funds will be made. Subsequent changes will not be regarded as acceptable and will prejudice the confidence of the client and financiers in the ability of the Engineer and his profession. Accuracy at this stage should be within 10% of the final cost of the project.

As different schemes will be compared as part of the feasibility process it is important to calculate the costs of all the different schemes in exactly the same manner or else the comparisons will be invalid. Only the person preparing the estimate will be aware of any differences in the assumptions made in preparing the estimates for the alternatives. Bias must be avoided.

The accuracy of the estimate will finally be determined by the prices tendered. For this reason a close watch needs to be kept on market forces and the degree of competition that exists to obtain work.

20.6.1 CLASSIFICATION OF COSTS

Costs of a differing nature should be identified at an early stage in the investigation as this will simplify subsequent calculations. Three such costs are illustrated.

Non-depreciable costs that will remain constant throughout the life of the project and do not have maintenance or depreciation costs allocated against them. An example is the cost of the land or once-off legal or Parliamentary costs.

Depreciable costs covering all items of construction or plant that will during the lifetime of the project require maintenance and will depreciate and require replacement at some stage.

Dependent costs, being those items that are calculated as a percentage of all the depreciable costs.

20.6.2 BASIC QUANTITIES

In order to achieve the required degree of accuracy the engineer should take out at least preliminary quantities of the different units based on an abbreviated Schedule of Quantities. Such a schedule need only cover some 10 to 15 items selected for their impact on the total cost. A typical schedule for a filter building is shown at the conclusion of this chapter. It may seem daunting to measure the quantities of a multiplicity of buildings and structures that differ only slightly from one another. Do the job once only by preparing spreadsheets for different types of structure. The spreadsheet can combine basic process design with calculations of quantities, presenting these in bill format to which rates can be applied and extended. The format of the sheet should be sufficiently flexible to cover all reasonable variations of that type of structure. Bear in mind that in this day and age it is not possible to design each and every unit from first principles. Standardisation of the conceptual design of the unit is essential but without limiting the scope for design improvements.

An important benefit gained by the creation of a spreadsheet for cost estimating purposes is the ability to optimise the cost of the structure by adjusting the proportions of the structure, e.g. the length and breadth of a rapid gravity filter bed. It also enables the designer to see immediately the cost consequences of any design decision.

20.6.3 COMPREHENSIVE RATES

The net rates tendered for any class of work are not suitable for use in an abbreviated schedule as advocated here. Too many other items of work are then not allowed for and errors occur.

The technique is to select the basic rate that is to be used to cover an item or class of work. Then connect all the smaller work-related items, sum the costs of these and add this total to the cost of completing the basic item. Then divide this total cost by the quantity of the basic item to derive the comprehensive rate.

As an example for the calculation of earth works quantities it will be necessary to define the height if the structure above some given ground level as well as the depths to intermediate material and hard rock. The rates can then be simplified to:

- excavate in any material
- excavate in any material and backfill
- extra over for
    - intermediate material
    - rock
- form embankments on site
- cart away excess material.

The rates for carting away would be as per any existing tendered rate, as would those for intermediate material and rock. The rate for excavation and backfill would be calculated from the total cost of all similar items including trenching, and for working space to erect shuttering, divided by the total volume scheduled for such items. The rate to be used for forming embankments would be the sum of the costs of all related items such as topsoil and grass, double handling, etc., divided by the total volume of embankment material. The rate for general excavation would be the sum of the costs of the remaining items in the bill, including excavation in confined spaces, etc. divided by the total volume of the bulk excavation. Thus the many small items are catered for in the single comprehensive rate calculated for the major item.

Suitable allowances need to be made for preliminary and general items as a percentage of the net value of work. This value should be similar to the preliminary and general of the contract from which the rates were taken.

20.6.4 SITE-SPECIFIC ASPECTS

At the feasibility stage it will be possible to identify the site specifics that affect the cost.

The distance of the site from a major centre will add significantly to costs, especially if the site is remote and there is no direct telephone link with the contractor's suppliers. Rates will have to be recalculated to take into account the locality of the works and the local difficulties that will affect the price.

The nature of the terrain will impact quite severely on the magnitude of the site works required as well as add to the general construction difficulties. Allowances should be made for the extra terracing needed where steep slopes are encountered. The feasibility of erecting the mechanical and electrical plant needs to be considered at this stage, and space must be allowed for the transport and crane vehicles.

The subsoil conditions, the presence or absence of rock or special geotechnical features, need to be carefully evaluated and any additional costs arising, included in the estimate.

20.6.5 COMPLETENESS OF ESTIMATE

The following list suggests some of the items that may have to be taken into account:

- Access road
- Power transmission line
- Power transformer and HT switchgear
- Sewer connection or disposal system
- Telephone connections
- SCADA control system
- Telemetry links
- Security measures
- Site fencing
- Cost of land
- Administration buildings
- Servitude acquisition.

This list should not be regarded as complete or comprehensive.

20.7 COST OPTIMISATION

As mentioned above the use of spreadsheets incorporating the calculation of quantities for design can bring benefits in respect of cost optimisation. They can be used to test virtually every design decision that is taken and built into the spreadsheet. The value of the spreadsheet rises exponentially with the degree of sophistication that is incorporated into its design. Careful thought should be given as to the final results that are to be expected from it when commencing a new spreadsheet.

It is unfortunate that at the feasibility report stage a great many "nice to have" features that are later included in the tender documentation are not dealt with at this stage. These may be for additional equipment, features to allow for improved operational flexibility, or improved quality as a result of over-specification. Once again a clear definition of scope and quality standard needs to accompany the feasibility estimates. The additional features may well be justified extras but it is as well to know that they are extra to the original budget should there be a query in the event of a budget overrun.

20.8 CALCULATION OF REFERENCE UNIT VALUE OF WATER

The reference unit value is the cost of treating a unit volume of water over a fixed period of time taking into account all the costs that will be incurred throughout the period. It is represented by the net present value of all the costs that will be incurred, divided by the discounted volume of water that will be treated over the same period.

Financial comparisons are made by calculating all the costs that would be associated with the construction and development of the scheme for its useful life, summing these and discounting to some common base date. Allowance is made in the calculations for maintenance as well as the depreciation that will occur on the equipment and structures. As the major expenditure is likely to be on the civil engineering components which should have a life of from 40 years and upwards the calculation of the present worth of most water works schemes is taken over this or greater periods.

The useful life of any component of the scheme will be limited, notwithstanding any maintenance work that may be carried out. The value of the item is generally written off linearly over its life. At the end of this period the component will have to be replaced. Therefore, in calculating the life-cycle costs of the scheme, allowance will have to be made for both depreciation, and once the residual value of the component has reached zero, for the replacement cost. At the end of the period any residual value must be included and discounted as a credit to the base date.

The life of the component will depend on its type, make and quality standards to which it was manufactured, as well as the quality of the maintenance that will be carried out on it. Higher quality mechanical equipment should have a longer life than a cheaper substitute, provided that the maintenance is equal for both items. This, however, is difficult to quantify and in the calculations that follow standard values as used by earlier government departments are used for the life of the type of equipment are used.

There is surprisingly little information and documented costs available in the public domain on this aspect.

20.8.1 DEPRECIATION, MAINTENANCE AND REPLACEMENT COSTS

The annual maintenance costs are frequently and conveniently expressed as a percentage of the initial cost. Again there is little published information available justifying the percentages that are in common use.

The annual factors and accepted life for various categories of work are set out in Table 20.1.

TABLE 20.1
LIFE AND MAINTENANCE FACTORS OF COMPONENTS OF WATER WORKS

Component	Life years	Annual maintenance factor
Concrete structures	40	0,25% to 0,5%
Buildings	40	0,5% to 1,0%
Dams and similar structures	45	0,05%
Pipelines	45	0,05%
Mechanical components	10-15	5,0% to 10,0%

20.8.2 STAFFING

It will be necessary in any cash flow analysis to estimate the number of staff that will be required to operate the water works. In accordance with Government Gazette Notice R2 834 of 27 December 1985 the numbers and categories of operators are laid down. In terms of this Notice the Director General may allow fewer persons or persons with a lower educational qualification to be employed for the operation of any water care works for the period and subject to the conditions determined by him, if he is of the opinion that in the particular circumstances the attainment of the objectives of the regulations will not be frustrated by such employment.

The costs of these people will have to be determined in conjunction with the employer or owner of the water works. An additional overhead component to cover such items as normal and sick leave, pension scheme, medical benefits as well as the administrative costs associated with the employment of that person, should be added to the net wage or salary. This overhead also varies according to the nature of the employment agreement, i.e. whether hourly, weekly or monthly paid.

The annual cost should be calculated including the full overheads as set out in the preceding paragraph.

20.8.3 COST OF CONSUMABLES

The treatment of water will involve the application of certain chemicals in doses that can be predetermined, the use of power, as well as the use of certain items that will be consumed.

It will be necessary to calculate the annual chemical consumption over the full life of the scheme, frequently taken as 40 years, and pro rated for actual volumes of water to be treated in each year of consideration. This is most easily done on a spreadsheet, especially prepared for this purpose.

20.8.4 CALCULATION OF PRESENT WORTH AND REFERENCE UNIT VALUES

An example of the calculation of the reference unit values is given at the end of the chapter. It has been prepared on a spreadsheet and takes into account the design and construction periods in calculating the cash flow in the years of construction.

20.9 DESIGN OUTLOOK

In the early 1960s it was conventional to design a water works or a reservoir for an outlook of 12 years whereas a pipeline, which exhibits greater advantages of scale, would be constructed with an outlook of up to 25 years. In the 1980s and later, these periods have been conventionally reduced, seemingly without any supporting calculations or theory to justify this change. The following section endeavours to provide a more rational approach to the selection of the design outlook.

20.9.1 GROWTH SITUATION

The population of South Africa is growing at an overall rate of approximately 3% per annum. Economic and geographical factors are causing most of the growth in the urban centres as people move there from the rural areas where opportunities for economic advancement are fewer. In consequence there are urban areas growing at considerably faster rates than the national average. It is not the purpose of this chapter to discuss means of forecasting water demand or how to predict what the future growth will be. It does assume that for any scheme a particular logarithmic growth rate has been accepted. This single decision affects the cost of the scheme more than any other. The next major decision is to determine what the optimum outlook of the next stage of the scheme should be.

20.9.2 COMPETITION FOR CAPITAL

A reduction in the design outlook of a scheme appeared to come from the increased competition for available funds or in simplified economic terms, with an increase in the discount or borrowing rate. The former may be termed the opportunity cost of the capital or the return that could be obtained by optimal alternative investment. Another concept is that the discount rate is the difference between the current borrowing rate and that of the annual inflation rate and represents the true return on a simple fixed investment. In the years when the inflation rate was higher than the borrowing rate the discount rate would thus have been negative. It remained at between 8% and 10% according to the economist consulted. A simplified explanation is that it is an index, expressed as a percentage indicating the availability of capital for development projects.

Before finally recommending a scheme on economic grounds it would be advisable to seek the advice of the owner's economist as to what the correct rate should be.

20.9.3 EFFECTS OF GROWTH RATE AND DISCOUNT RATE ON DESIGN OUTLOOK

The next most important decision having selected the growth rate in future demand and the discount rate to be used, is the outlook for which the design should be made. This is defined as the period in which the growth in demand will rise to equal the design capacity of the works and before which the next extension should be completed.

It has been found that this period is a function of the nature of the works and the effect of scale on the cost of construction.

20.10 CHECKLIST FOR ESTIMATING COST OF WATER WORKS

A checklist is given here to aid in estimating the cost of a water purification works. The steps include:

- determination of components of the scheme
- capacity of works
- costs

20.10.1 DETERMINE COMPONENTS OF THE SCHEME

For raw water procurement consider the following:

- Dam or river
- Gravity of pumping main
- Raw water pump station
- Balancing tank/storage
- Control valves

Water purification components are:

- Inlet structure and hydraulic control
- Chemical house and chemical storage
- Pre-chlorination/ozonation
- Mixing and coagulation
- Flocculation
- Sedimentation and/or dissolved air flotation
- Filtration : Rapid gravity
  : Pressure
  : Dual media
  : Slow sand
- Post-liming stabilisation
- Disinfection : Chlorination
  : Chlorine dioxide
  : Ozonation
- Post-filtration storage and balancing tank
- Clear water pump station
- Delivery pipeline
- Site works and stormwater

Infrastructural requirements include:

- Fencing, roads, water supply, sanitation
- Power supply : Establish source
  : Powerline
  : Substation/mini-sub
- Access : Length and category of road
  : Site roads
  : Township roads
- Telephone : Costs from Telkom
- Sewerage : Reticulation
  : Treatment system

Appurtenant buildings consist of:

- Offices
- Workshops : Size
  : Lifting equipment
  : Equipment
- Staff accommodation : Number, area and level of accommodation
- Guard rooms and security

20.10.2 DETERMINE DESIGN CAPACITY OF WORKS OR EXTENSION

- Establish future increases in water demand over a period of time (as long as possible)
- Establish capacity of current installation
- Determine design outlook for different components of the scheme based on current discount rate and appropriate scaling factor
- Finalise capacity of works to be constructed.

20.10.3 DETERMINE COSTS

- Prepare abridged schedule of quantities appropriate to different units
- Ascertain global rates appropriate to schedule
- Prepare net estimate of costs
- Add contingency allowance
- Calculate allowance for inflation taking into account inflation rate and period and duration of construction
- Calculate professional and design fees including site staff, S & T costs and disbursements, calculate total
- Add percentage of total for value added tax
- Calculate total estimated budget cost.

21

Operation, Control and Management

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The purpose of operating a water purification works is to produce the required quantity of potable water of acceptable quality. The various aspects involved in the operation of a water treatment works consist of operation, maintenance, supervision, management, control and safety.

Operation consists largely of the activities involved in the production of treated water. These activities will normally be performed by the assigned operating staff. Maintenance would involve the activities carried out by designated personnel in keeping the plant operating in good mechanical and electrical condition.

Supervision entails the overseeing of the operating activities by the person or persons responsible for the plant such as the works manager, superintendent or shift supervisor.

Control includes the processes and routines instituted to ensure that operation is carried out to suitable standards. Safety operations at the work should comply with safe procedures and relevant legislation.

Management comprises the higher level activities necessary in order to ensure continuous adequate production of potable water for the supply area. These are not necessarily based at the works.

Public relations is an important aspect of management of a water works as it provides a feedback on acceptability and thus on the quality and quantity of the product delivered to the public. The water works at many towns are of interest to various groups of people such as nurses, public health workers and school children. Some of the time of the works operating and supervisory staff may be taken up by conducting tours of the works and explaining the various processes involved.

21.1 OPERATION

Operation generally covers the work carried out by the operators or shift workers at the works. Among the duties to be carried out by the operators would be the following:

- Set flow rates
- Set dosages and chemical feed rates
- Check plant and equipment
- Desludge clarifiers or sedimentation tanks as required
- Backwash filters as required
- Carry out treatment of sludge
- Test water quality and take corrective action
- Carry out basic maintenance
- Record activities
- Handle chemicals
- Order chemicals, parts and tools
- Prepare and work to budgets
- Works calculations
- Communicate with other operators and supervisors
- Safety inspections
- Start-up and shut-down plant

21.1.1 Setting flow rates

This entails the adjustments of raw water flow into the works based on the requirements of the area reticulated in the town or the levels of the reservoirs downstream of the works. Setting flow rates entails adjusting gravity flow from a river or a dam into the works or operating the raw water pumps which would pump water to the works at certain discrete feed rates.

When setting raw water flow rates to the works it should be borne in mind that better process stability is achieved if flow-rate changes to the works are minimised. Ideally with large water storage in the reservoirs it may be possible to run 24 hours per day at an essentially constant flow rate and allow the reservoirs to rise and fall, accommodating the peaks and troughs in demand. Not all works are fortunate enough, however, to have such a situation and adjustment to meet daily variation in demand may be required.

If this is necessary attempts should be made to minimise the changes as far as possible. Two changes per day to give day and night treatment rates would be preferred if this can be achieved. The fewer the changes made to the flow rate and the more standardised operation can become, the less chance there is of errors occurring in the setting of chemical feed rates and dosages and hence in maintaining good quality purification at the water works.

21.1.2 Setting of dosages and chemical feed rates

At a water purification works, depending on the processes involved, a number of different chemicals may have to be added. These would include coagulants, floc aids, pH correction chemicals, pre-oxidants and disinfectants. All chemicals added have to be adjusted if the flow rate through the works is changed, hence the recommendation given previously that changes to works operation should be made on a planned basis and should be minimised as far as possible.

Coagulant addition is traditionally the main treatment chemical added at a works as this destabilises colloidal material in the raw water, enabling it to be settled out and removed in the settling tanks and filters. A number of coagulants are available for use. These include inorganic coagulants such as aluminium sulphate (alum), ferric chloride and polyaluminium chloride. There is also a fairly wide range of organic coagulants which are commonly known as polyelectrolytes. Sometimes polyelectrolytes are blended with polyaluminium chloride or ferric chloride to produce a mixture which yields good results on certain types of water. Coagulation is dealt with in detail in Chapter 6.

Coagulant demand is determined by the requirements of the water and is normally tested using a flocculation test machine or jar test. The jar test may be carried out either by the operating staff or by the chemical services department of the water authority. In a jar test a number of samples of water of known volume are dosed with increasing dosages of the chemical under consideration. The dosage at which optimum results are obtained is then used on the plant. This produces the most economical treatment costs for full-scale operation.

It should be noted that jar tests do not always perfectly reflect the conditions which give optimum performance on the plant and slightly higher or lower dosages will need to be determined by experience on a particular water works.

Lime can also be used for coagulation purposes under certain conditions. On certain waters the addition of lime up to a pH of

10,5 to 11,5 will produce a floc which can be settled out and the water recarbonated with carbon-dioxide to reduce the pH. This process is not widely used except for Vaal River water; as it requires specialised equipment.

Chemical addition for pH control for coagulation purposes varies depending on the coagulant being used. When using alum which is amphoteric, optimum flocculation generally occurs within a fairly narrow pH range, usually between 6,5 and 7,5. One would therefore add sufficient soda ash, lime or caustic soda to bring the pH to an optimum point within this range after the alum addition. Further pH correction would then take place after filtration (when all alum floc has being removed) to raise the pH to a level suitable for distribution.

Aluminium sulphate, ferric chloride, polyaluminium chloride and polyelectrolytes i.e. all coagulant chemicals, are added as solutions. They are normally fed into the water using a dosing pump; the most common type of dosing pump is the diaphragm pump which adds the solution in a series of pulses.

It is important to even out these pulses in the dosage line to ensure a steady feed of coagulant into the water. Otherwise successive cycles of overdosage and underdosage occur which may not be recovered from in the treatment process and will lead to higher chemical consumption and poorer water quality.

Lime does not dissolve to any great extent and is commonly added as a slurry. Lime is often fed from a dry feeder which drops it into a dilution water stream which in turn is conveyed to the dosage point of the treatment stream. If lime is used for post-pH correction, a lime-dissolving tank may be necessary to prevent solids accumulating in the reservoirs.

Pre-oxidant chemicals such as potassium permanganate and hydrogen peroxide are also added as solutions with dosing pumps.

Disinfectants are added to the water to remove pathogenic or harmful micro-organisms and to render it safe for human consumption. Chlorine is the most widely used disinfectant and in South Africa it is virtually the only one used. In other countries chlorine dioxide is sometimes used as it also gives a residual in the water which persists through the reticulation system. Ozone is extensively used in France and Germany and to some extent in other parts of the world. However, some chlorine needs to be added to the purified, disinfected water, in order to ensure an active residual in the distribution system. Ultraviolet(UV) disinfection is sometimes used but this is not recommended on large-scale systems as it does not produce a residual which protects the system.

Chlorine, when added to water disinfects extremely rapidly under the correct pH conditions, - the lower the pH the more effective the chlorine disinfection within normal limits. At a pH of 8,5 or greater the effectiveness of chlorine is reduced somewhat although good results can still be obtained if adequate storage is present in the reticulation system.

Chlorine may be added either from gas cylinders or from a solution of sodium hypochlorite or as calcium hypochlorite (HTH). On works of reasonable size chlorine gas is the most common either in 68 kg or 900 kg cylinders. The rate at which chlorine can be fed from a cylinder or drum is limited and where large quantities are required a number of cylinders or drums are connected to a manifold to ensure an even draw at the required rate.

Chlorine is drawn as a vapour from the cylinders and is passed through a measuring tube to an ejector which is driven by a high-pressure water pump to prevent suck-back of water into the gas system. The solution is then added to the water at the pre- or post-chlorination dosage point. If the pH of the water is sufficiently low 30 minutes of contact in a well-designed contact tank should be adequate for disinfection purposes. However, if the pH is high, several hours' contact may be necessary for complete disinfection. This may mean that storage in a reservoir is required before completion of disinfection.

21.1.3 CHECK PLANT AND EQUIPMENT

It is the duty of the operator to check all plant and equipment at routine intervals during his period of duty. This includes (i) inspection of the raw water and intake channels, (ii) operation of all dosage pumps and chlorinators, (iii) inspection of the sedimentation tanks or clarifiers to ensure that carry-over is not occurring and (iv) inspection of the filters to see that they are performing satisfactorily and to ascertain when backwashing is necessary.

The operator should also check whether the raw water and purified water pumps are operating normally and whether the backwash system for the filters is functioning satisfactorily. On works of fairly large size it may be beneficial from a cost and control point of view to provide automatic monitoring of conditions on the works with alarm indication when equipment trips out or goes out of specification. On a small works this function relies on the diligence of the operator and operators should therefore be encouraged to walk around their plants and get to know them well.

21.1.4 DESLUDGE CLARIFIERS AND SEDIMENTATION TANKS

All raw water contains suspended solids to a greater or lesser degree. These are coagulated and flocculated by the addition of coagulant chemicals and settle in sedimentation tanks, or are removed in clarifiers. Algae, silt, clay, etc. are suspended solids.

Obviously, with the passage of time there will be an accumulation of solids at the bottom of tanks, or in sludge compartments, and periodic desludging is needed. Most works have tanks with hopper bottoms or other suitably designed outlet arrangements so that sludge can be removed by opening a desludge valve or valves for a suitable period. The method of desludging depends on the type of tank installed.

For a sedimentation tank desludging will take place until virtually no solids are present in the desludge liquid. A floc blanket clarifier, however, would require that a floc blanket is maintained in the clarifier and only sufficient solids should be removed to prevent excessive build up in the unit. Control is often achieved in proprietary designs by having a submerged overflow basin within the clarifier. Desludging then takes place on a similar basis to a sedimentation tank i.e. desludging should be continued until the water runs relatively clear.

Desludging of clarifiers and sedimentation tanks should be carried out routinely with the period and frequency of operation depending on conditions at the works. Units which clarify highly turbid river waters may require desludging several times a day, whereas a works which is treating a water of low turbidity may only require desludging once a day or even less often. The frequency will be based on operating experience and recommendations of the designer of the plant installed.

21.1.5 BACKWASH FILTERS

It is assumed that most of the works under consideration will be served by rapid gravity filters with only a limited number of plants having pressure filters. Some works may, however, have slow sand filters. Obviously if a works has slow sand filters these will require attention in a different way from rapid filters.

Slow sand filters operate for a period of several weeks at a time before requiring attention. When the end of the filter run is reached, i.e. the head loss through the bed increases to the limit, the filter is taken offline, allowed to dry out, and the surface layer is removed by scraping. The filter is then recommissioned at a slow rate to allow a new film to build on the surface after which operation continues normally.

For a normal rapid gravity works the filter runs or period of operation of the filters may vary quite widely. This will depend on the nature of the raw water, especially turbidity and the performance of the clarifiers or sedimentation tanks upstream of the filters. There are also a number of works where the raw water is always of low turbidity, where clarifiers are not provided at all, and the coagulated, flocculated water passes directly to the filters. Algae-laden water usually require treatment ahead of the filters.

On a works without clarifiers, backwashing of filters is normally required two to three times per day. On works which are provided with clarifiers or sedimentation tanks backwashing may be required once a day or less. Filter runs of up 96 hours are not uncommon, but the temptation to prolong the filter runs should be avoided as secondary problems may occur. Under normal conditions filter runs of 24 to 48 hours are practised.

A filter may be backwashed either on a timed basis or on headloss. Headloss indication is provided on many proprietary makes of filter and if this is reliable it can serve as a suitable basis for the initiation of backwashing. It is not infrequently the experience however, that headloss indication is somewhat unreliable. In this case it may be preferred to backwash the filters on an elapsed time basis.

Backwashing procedures will vary depending on the type of filter installed. Most filters have air plus water backwash while some have water wash only. In the case of water wash only, the expansion of the bed is of significant importance. At the end of the backwash cycle the filter is returned to duty.

Rapid gravity filters in South Africa are normally provided with an air-scour system. The procedure is then to close off the feed and outlet, allowing the water level above the filter to reach a suitable level for the scour operation, start the air blower, and scour the filter for a number of minutes. The purpose of the air scour is to dislodge the dirt which has settled on or become attached to the sand particles in the filters. Air-scouring loosens the dirt and enables the backwash water to remove it more easily. Backwash water is then passed through the filter until backwash water is reasonably clean. This will also take several minutes but varies depending on the depth of the filter, the backwash rate and the amount of material to be removed.

Pressure filters and some rapid gravity filters only have water backwash without air scour facilities. In this case all that is required is to close off the feed and the outlet, to start the backwash pumps, and to backwash the filters for a certain period of time, and then to close the backwash valves and restart operation.

21.1.6 TREAT SLUDGE

As will be evident from the discussion in the two preceding sections, sludge production takes place at a water works through the processes of coagulation and flocculation. Most of this sludge is settled out in sedimentation tanks or clarifiers and the remainder has to be removed in the filters.

On many works, backwash water recovery is provided by collection from the filters and pumped back to a point near the head of the works upstream of the sedimentation tanks. The sludge in the backwash water from the filters then settles out in the tank resulting in a single point of sludge removal from the works. Separate sludge thickening can also be provided. This aspect is dealt with in Chapter 15.

The sludge removed from the clarifiers and possibly from the filters has to be disposed of. For many years it was common practice to return water works sludge to the river or supply point from which the water was abstracted. As the sludge from a water works is composed mainly of solids from the raw water this would not be a harmful practice, provided that it did not comprise a major portion of the receiving flow.

However, with increasing eutrophication and urbanisation this practice has fallen into disrepute. In any event it does not comply with the provisions of the General Standard of the Water Act. In terms of the Act a water works is an industry and certain standards have been promulgated for discharge of waste streams to a water course. In nearly all cases it has therefore become necessary to provide some form of sludge handling at the works, as well as for formal disposal.

In its simplest form, sludge handling would be done by means of sludge lagoons or drying beds. However, these have a land area requirement. Large works therefore usually use mechanical dewatering methods; the sludge then still has to be disposed of.

A small works with large land area can be provided with three or more sludge lagoons such that one is being filled, one is settling and drying out, and one is having its solids removed at any particular time. In this way continuous removal of sludge solids can be catered for. Most works however, do not have sufficient land for lagoons or drying beds and some form of mechanical dewatering is therefore necessary. As mentioned previously this is particularly the case in works of large capacity.

Normal design would be to provide for some form of sludge thickening of the clarifier scour water and backwash water prior to dewatering. The thickener would be either a gravity thickener or a dissolved air flotation cell. The dewatering system would consist of either a centrifuge or a belt press. Tubular filter presses have been used on some works and plate and frame filter presses are used in Europe although this technology is falling into disuse.

The centrate from the centrifuge or the filtrate from the belt press can be returned together with the thickener overflow of water to the head of works if complete conservation of water resources is necessary. However, these streams are usually somewhat contaminated. It might therefore be prudent to discharge them to evaporation ponds. Final removal of sludge would then be necessary from site to landfill, or other suitable means of disposal.

21.1.7 TEST WATER QUALITY AND TAKE CORRECTIVE ACTION

It is the duty of a plant operator to ensure that the water he is producing is always of adequate quality. This is established by means of routine testing of samples. Whilst complete analysis cannot be carried out at a works on a routine basis, there are a number of tests which can be carried out which provide an adequate check for routine purposes. Thus for potable water the turbidity, conductivity, colour, pH and residual chlorine can be measured quite easily.

Turbidity is measured with a purpose-made turbidimeter. This functions by measuring the amount of light scattered by a water sample in a specially designed instrument. The scattered light is picked up by photocells and amplified, and provides a signal which provides a digital or analogue read-out of turbidity in the sample.

Purified water for potable purposes, should comply with suitable quality parameters. According to the SABS 241 of 1984 the recommended turbidity level for treated water is 0,5 turbidity units; the objective should be to have a lower final turbidity. Raw water and clarified water turbidities should also be measured on a routine basis for record and control purposes.

Operators should familiarise themselves with the levels of turbidity at various points on their works and be aware of deviations from the average. This is particularly important on a works which treats water from a river where large changes in turbidity can occur in the raw water. This in turn significantly affects the coagulant feed requirements to the process. A high turbidity water generally requires higher coagulant dosages.

A change in the raw water turbidity can result in massive changes in the turbidity of the treated water if corrective action is not taken. An early sign of this is the appearance of the water from the clarifiers and the filter clear wells and operators should therefore observe their works units on a routine basis. However, the turbidity should also be measured as this provides an indication of the actual quality which can serve as an accurate record and a guideline for chemical dosages.

Conductivity is indicative of the dissolved solids content of a water. The total dissolved solids (TDS) can be calculated directly from the conductivity, using a factor. Conductivity is measured using a conductivity meter which comprises a conductivity cell in which the water is placed and which measures the current moving across the electrodes. This is amplified and converted into a conductivity signal. Conductivity is the reciprocal of resistance to current and is expressed in millisiemens per metre (mS/m). The conductivity of a water should be relatively low, if the water is of good quality. High dissolved solids can have undesirable side-effects in potable use, and the limiting conductivity for potable use is set out in SABS 241 of 1984. Unless very high chemical dosages are being practised on a water works it is unlikely that the conductivity of the treated water will differ significantly from that of the raw water. This should be tested by the operating staff and a decision made whether both need to be measured on a routine basis or whether just the treated water will provide an adequate indication.

As the term implies colour is an indication of the amount of colour-causing matter in the water. Colour is present through organic, inorganic or suspended substances in the raw water and the colour of raw water can be quite high at certain times. The coagulation process in the works usually removes colour to a great extent and if colour removal is successful, it should barely be detectable in the purified water.

Colour is measured using a deep cell comparator with a white light source. The units of colour are Hazen units which are measured on a platinum cobalt scale and indicate a yellowish brown colour present in the water. It is not abnormal for raw water supplies to have colour values in excess of 100 Hazen units; such colour would indicate industrial pollution.

The limiting colour for treated water is set in SABS specification (241 of 1984). A well-purified water should accordingly have a colour of 5 Hazen units or less.

Certain waters are difficult to treat, however, as far as colour removal is concerned. This applies particularly to waters in the Western Cape which have high concentrations of humic substances. Colour removal from these waters is somewhat more complicated and usually involves coagulation at reduced pH or ozonation.

Measurement of pH is carried out using a pH meter which comprises a specialised electrode together with a millivoltmeter calibrated in pH units. pH readings range from 1 to 14 with the neutral point normally being considered to be 7,0. Thus, pH values less than 7 are regarded as acid and pH values higher than 7 are regarded as alkaline. This is not strictly the case in water treatment however, where the neutral pH value or pHs may be as high as 9,0.

Every water should be analysed such that its pHs value can be determined as this will indicate the desirable pH for discharge from the works. If the water has a pHs of 9 and the water is discharged at a pH of 7 the water will be strongly corrosive and will corrode the reticulation system. Conversely, if the pHs is 7 and the pH of water treated from the works is 8, the water will be mildly scale-forming and this can cause problems in boilers and hot water systems. As a general rule the water discharged from the works should be about 0.5 of a pH unit below the pHs value.

Residual chlorine is of importance in the protection of the reticulation system from re-infection. Chlorine is normally added at a water works, sometimes as pre-chlorination, and almost always as post-chlorination, to disinfect the water and remove pathogenic organisms. A residual chlorine concentration is kept in the water to ensure that this protection persists throughout the reticulation system. The operating authority should establish what residual chlorine concentration would be adequate to protect the entire reticulation system. The normal aim is to ensure that a residual concentration of 0,05 to 0,1 mg/ℓ is present at the furthest point. This may mean that consumers right next to the water works may be receiving water with residual chlorine concentrations in excess of 0,5 mg/ℓ. Should the residual chlorine concentration in the final water need to be in excess of 1 mg/ℓ, to ensure protection of the whole system, other arrangements need to be considered, such as re-chlorination in the reservoirs or the use of chloramines to ensure persistence of chlorine throughout the reticulation system.

Residual chlorine can be measured with a comparator which includes a coloured disk and uses DPD tablets or powder. The water in the test tube is dosed with DPD powder. Chlorine in the water reacts to produce a pink colour which is compared with the colour on the disk, and the concentration is read off. Chlorine dosage on the works should always be set to maintain an effectively constant residual chlorine concentration (usually >0,2 mg/ℓ) in the reservoirs and reticulation system.

21.1.8 CARRY OUT BASIC MAINTENANCE

The amount of maintenance to be carried out by the operating staff at the water works needs to be decided by the water authority based on the qualifications and skill of the operators and the complexity of the tasks to be performed. Most operating staff can be relied on to carry out certain basic routine activities such as cleaning and scouring of dosage pipelines, cleaning of in-line filters and routine minor lubrication.

However, additional work should be planned based on the qualifications of the operators. For example, some works operators may be qualified artisans capable of carrying out fairly advanced maintenance, for example, installation of stand-by pumps, should this be required.

21.1.9 RECORD ACTIVITIES

It is essential for the controlled operation of a water works that the work carried out is recorded in suitable form. This would comprise a log book for the Superintendent or Works Manager, log sheets for the operators, and analytical result sheets for either the operating staff or chemical staff associated with the works. The log book would be in diary form and would record any unusual occurrences at the works during the day in question such as works visitors, breakdowns in the plant, or anything else of note.

The log sheets would record the operation of the works. Pumping hours, flows and meter readings should all be recorded on a routine basis, either hourly, every two hours, once or twice a shift or daily, depending on the variation and the importance of the item being recorded. An example of a typical log sheet is shown in Figure 21.1.

The Works Manager or Superintendent should inspect the logs on a daily basis to ensure that these are being filled in properly as well as to assimilate the information contain in them. Based on information in the log sheets and reports from the operators, he would then take whatever action is necessary to correct any abnormal situation which might be taking place.

Similar comments apply to results of analysis. It is recommended that operating staff measure and record turbidity, conductivity, colour pH and residual chlorine. These results should be recorded on a routine basis at least daily and if the plant operates on a shift basis, preferably once per shift. These results should also be inspected to ascertain whether there are any trends which need to be followed up and action taken thereon.

The water engineer will call for a monthly/quarterly or annual summary of the recordings.

21.1.10 HANDLING CHEMICALS

The use of chemicals is usually necessary in water purification and treatment. Many of these chemicals are difficult to handle and may be toxic or irritating to the skin. Operating staff should ascertain the precautions necessary for the handling of all the chemicals in use at a particular works. These are normally obtained from the suppliers of such chemicals who have manuals of practice and codes to handle chemicals. In particular chlorine requires great care and the assistance of the suppliers in handling this chemical safely should be obtained. Other chemicals such as polyelectrolytes or ferric chloride may be toxic in large quantities or be acid, and the suppliers of such chemicals should provide information on the best means of handling them. This may require modifications to the system originally designed to be installed at the works and this should be discussed with the design engineer.

21.1.11 ORDER CHEMICALS, PARTS AND TOOLS

The superintendent or manager of the water works should establish a reporting system whereby he is timeously informed of the need to order chemicals so that appropriate action can be taken.

Potable water is an essential commodity and its supply should not be jeopardised through poor procurement systems. Auto-

OPERATIONAL DETAILS

CHEMICAL,PHYSICAL AND BACTERIOLOGICAL ANALYSIS
MONTH.............. YEAR ....

DATE	WATER : RAW							CLARIFIED WATER						WATER : FINAL								
				mg/l as CaCo$_3$				1		2		3				mg/l as CaCo$_3$				LANGELIER INDEX		
MONTH	°C	N.T.U. TURB.	pH	COND mS/m	HCO$_3$	T.H.	Ca.	N.T.U. TURB.	pH	N.T.U. TURB.	pH	N.T.U. TURB.	pH	°C	N.T.U. TURB.	COND mS/m	HCO$_3$	T.H.	Ca.	pH	PHS	SAT INDEX
01																						
02																						
03																						
04																						
05																						
06																						
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26																						
27																						
28																						
29																						
30																						
31																						
TOTAL																						
AVE.																						
MIN.																						
MAX.																						

Figure 21.1
Typical log sheet

OPERATIONAL DETAILS

QUANTITIES OF WATER, CHEMICALS USED & DAILY STOCK
PUMPS AND EQUIPMENT MAINTENANCE

WATER : FINAL				RAIN FALL IN mm	HUMIDITY		TEMP		QUANTITIES			CHEMICALS CONSUMED						MAINTENANCE	
CHLORINE mg/l		BACTERIO-LOGICAL			MIN	MAX	MIN	MAX	RAW	LOSS	FINAL	POLY-C		ALUM		Cl_2		PUMPS	EQUIPMENT
FREE	TOTAL	TOTAL	E-COL		%	%	°C	°C	m^3	m^3	m^3	kg	mg/l	kg	mg/l	kg	mg/l		

Figure 21.1
(continued)

matic order levels and stock control should be instituted so that whenever stock reduces to a certain level an automatic order is placed with the suppliers for replacement.

Parts and tools are also necessary as consumable items in a water works. The Superintendent/manager should ensure that spare parts are available for critical equipment so that repairs can be carried out quickly. This duty may form part of the maintenance function in larger works but in smaller works it often falls to the Superintendent to ensure that spares are available.

The operating authority should ensure that its procurement systems are adequate to cope with any emergency situation.

21.1.12 PREPARE AND WORK TO BUDGETS

The assistance of operating staff, superintendents and works managers is necessary to prepare budgets for the operation of water works. This is assisted by good record-keeping. If one has a record of chemical consumptions over the past year it is much easier to estimate likely quantities to be required for the following year. Budgets are required in any authority to plan expenditure over the coming financial year. Good estimates of proposed chemical consumption are necessary as these are frequently large items in the overall budget.

The assistance of operating staff is also necessary for capital budgeting. Problems may be experienced with the operation of certain parts of the works or the works may be showing signs of coming up to full capacity. It is the duty of operating staff to inform the higher authority when this is happening so that plans can be made in good time to carry out extensions or modifications.

21.1.13 CARRY OUT CALCULATIONS

Staff on any water works should be able to answer queries as to what the relevant dosages are at a particular time. Thus, if asked what the chlorine dosage is, the superintendent or operator should be able to calculate this from the water flow and the chlorine feed rate. This type of calculation is normally covered in the training of operators and these calculations should be used in ensuring that the operating staff have a full appreciation of dosages as well as consumptions on the plant.

21.1.14 COMMUNICATE WITH OTHER OPERATORS AND SUPERVISORS

Many works are operated in isolation and operators may therefore be unaware that the problems they are experiencing may be similar to those in adjacent towns or in other authorities elsewhere in the country. Operators should therefore, through the authority, make contact with other works in the vicinity or which are treating a similar type of water, and exchange ideas and operating experiences. Operators associations exist to cater for this and operating authorities should encourage their staff to belong to these.

21.1.15 SAFETY INSPECTIONS

Safety is an important operational aspect. A works should comply with safety procedures and all relevant legislation. However, unless continually reinforced, practices can become corrupted and working procedures become slack. The site may become untidy, demarcations may wear out with time or things may be moved, and it is therefore necessary that routine and non-routine safety inspections be carried out. In a larger authority there will be persons whose main responsibility is safety. On a small works it may be necessary for the works manager or superintendent to carry out the safety inspections with the assistance of anyone who has some experience in this type of work.

The condition of safety equipment should be inspected and respiration apparatus, ladders, scaffolding, walkways, machine guards and all similar items should be inspected on a routine basis. As it is primarily for their sake, operators should be involved in the procedures. It should be ensured that staff are using gloves and protective clothing when handling toxic or dangerous chemicals. It should be noted that with the new Occupational Health and Safety Act the responsibility for safe practices falls squarely on management. It is incumbent on management to ensure that the works is safe and that safe working procedures are being adhered to. Should an accident occur it is common practice for the Chief Executive/responsible officer to be charged with negligence. To avoid this every possible action should be taken to ensure that safe procedures are being followed.

21.1.16 START-UP AND SHUT-DOWN PLANT

Operators running a works which operates continuously may not be familiar with the procedures for shutting down or starting up. It is not always possible just to switch on the feed pumps, switch on the dosing and find that everything works perfectly. The works operating manual will give procedures for start-up and shut-down of the plant and these should be familiar to the operating personnel and used accordingly. On works which only operate for a limited number of hours in the day, start-up and shut-down procedures become more important, but as they are more routine they tend to be routinely followed more correctly.

21.2 MAINTENANCE

Once a works has been built and handed over for operation, it is incumbent on the operating authority to maintain it in optimum operating condition. This requires maintenance of machinery and equipment, buildings, roads and grounds. Each operating authority needs to review the type of maintenance that should be carried out and to decide who should carry it out in practice. Maintenance can be carried out using in-house staff, by subcontracting certain aspects, by full privatisation and by sending out for specialist work.

In this part of the chapter we shall focus on the maintenance of machinery and methods of handling this problem.

The maintenance of equipment and machinery at a works can be carried out in several ways. These can be classified as routine preventative or planned maintenance, breakdown maintenance, and as an example of a structured approach or Reliability Centred Maintenance (RCM).

21.2.1 ROUTINE PREVENTATIVE OR PLANNED MAINTENANCE

In many ways planned maintenance is the ideal method of coping with the maintenance problem on a works. If all wearing parts could be replaced before their useful life was up, breakdowns would be reduced to a minimum and unforeseen disruptions to the process could be virtually eliminated.

Unfortunately, it is not always possible to accurately predict the service life of many mechanical items. Bearings for example, can last from a few hundred hours to many hundreds of thousands of hours.

If the replacement period is not carefully determined this can lead to breakdowns in the event of early failure, or to the replacement of parts which still retain a good portion of their service life. Nevertheless, for strategic items of equipment it may be preferable to replace items early rather than run a higher risk of breakdowns.

Preventative maintenance is normally regarded as being a high-cost option as it is associated with relatively high replacement costs and man-hours. The tasks involved in preventative maintenance can range from basic procedures which can be carried out by works operating staff through normal mechanical and electrical activities capable of being carried out by competent fitters or electricians, to specialised work such as servicing of centrifuges which requires specialised workshops and trained personnel. Each operating authority needs to decide what level of staffing can be justified by the scope of work available at the particular site and whether this warrants full-time staff or whether the work should be subcontracted or sent out.

21.2.2 CORRECTIVE OR BREAKDOWN MAINTENANCE

This differs in philosophy from planned maintenance in that the equipment is serviced regularly e.g. oil changes for gearboxes at required intervals, but certain items such as bearings may not be replaced after a fixed number of hours but may be allowed to operate until they become noisy or are on the verge of failure. Similarly pump impellers would be allowed to gradually wear and are replaced when efficiency drops below a certain level. Breakdown maintenance is usually less costly than planned maintenance as the intervals of servicing are normally greater. However when the maintenance is required it is usually quite urgent. It also may be complicated by the fact that two or more items of plant may break down almost simultaneously, necessitating extra manpower or prioritisation of jobs.

Another potential problem is that allowing certain items to operate to the verge of breakdown may result in more expensive repairs than if wearing items were routinely replaced.

All in all corrective or breakdown maintenance is not always the cheap solution. It can, however, be countenanced if stand-by equipment is available and spares are readily obtainable. However, any operating authority should list all of its equipment and make decisions as to which can be allowed to break down, which can be easily replaced, and which are critical to operation and require an additional stand-by.

21.2.3 RELIABILITY-CENTRED MAINTENANCE (RCM)

In-between the extremes of full preventative maintenance and breakdown maintenance there are a number of possibilities. It is usually economical for an operating authority to carry out maintenance with a mixture of the two. This requires listing all equipment and deciding whether the piece can be allowed to fail or not or whether it should be serviced routinely. RCM is one of a number of programmes which assist this sort of decision-making process, to set up a structured maintenance policy.

Water works authorities should investigate whether this sort of programme would be a useful tool to them in setting up a maintenance system at their plants.

21.3 SUPERVISION

Even on a very small works it would be very unlikely that all work could be performed by one person. Stand-by is always essential and assistance through the employment of staff is therefore necessary. In practice this could range from the employment of at least two labourers in order to carry out basic operations, to a complete staff structure on a large water works under the control of a Works Manager, which would consist of senior operators, operators, shift attendants, labourers and support staff.

The Department of Water Affairs and Forestry has laid down qualification requirements for water works operators; these should be adhered to by the water works authority.

Operating authorities should recruit staff who comply with these requirements or can be trained as is necessary. Under most conditions it will be possible for qualifications to be obtained whilst training on the job.

21.3.1 CONTROL OF LABOUR

Staffing on a small works would be under the control of a superintendent and would comprise a few relatively unskilled staff to provide assistance with the unloading of chemicals, batching of solutions, and checking that equipment continues to operate. Labourers will also be required to clean the buildings, garden and grounds.

Although the tasks at this level are limited in nature it is necessary to structure the jobs as much as possible so that each person has clearly delineated duties and a reporting structure. Some interchangeability would be necessary for stand-by purposes and this needs to be built in through job rotation.

It is possible that even on a small works it may be necessary for shift operation to be carried out. Under these conditions a somewhat higher level of staff would be required. This leads to the question of qualifications which will be discussed in subsequent sections.

21.3.2 CONTROL OF OPERATORS

The plant on a larger works would be operated by staff under the control of a superintendent or works manager.

Most large works and many smaller ones operate on a shift basis and the shifts would be controlled by a senior operator, supervising a number of operators and shift attendants.

Depending on the design of the works, it may be possible to schedule many of the tasks to be carried out by day staff. This would comprise the receiving of chemicals, preparation of dosage solutions, changing of chlorine cylinders and similar duties. Many of these tasks are best carried out in daylight when greater resources are available. The duties of the afternoon and night shift staff should however be structured in such a way that a number of useful tasks require to be performed at suitable intervals during these shifts. Tours of the works should be carried out at regular intervals.

Operators, senior operators, superintendents and works managers are required to comply with certain qualification standards depending on the size of the works. These are discussed further in a subsequent subsection.

The jobs to be carried out by the various operators and shift attendants and all persons on the works, should be structured as far as possible. Each person should have a job description and be responsible for a specific area with defined activities. Job enrichment through job rotation is beneficial to both the employer and employee.

21.3.3 ANCILLARY STAFF

Apart from the operators and the staff involved with purely operational aspects on a water works, there is also a need for ancillary staff who perform a number of service functions.

Depending on the size of the works there may be one or a number of maintenance staff such as fitters, electricians or instrument technicians allocated to the plant. In addition to this, if the works is automated and has a SCADA system, there may be a need for a data processing specialist. It should be noted that even on a relatively small works automation and instrumentation usually pays for itself through quality control and possibly reduction of the staff complement.

Staff are essential from a quality control point of view. Laboratory technicians or a works chemist, may be required to monitor the chemical quality of the water depending on works operation and the size of the works. Responsibility for operation and control may also be extended beyond the works itself back to the supply point, which might entail control of operation of a dam and also forward through the reservoirs to the reticulation system. This may require additional specialised staff.

The bacteriological quality of the water also has to be monitored. This can be done using an in-house, microbiological laboratory, or samples can be sent out to a suitable pathology lab or to the relevant health department. The frequency of such monitoring and the extent of sampling naturally depend on the size of the works.

Control of ancillary staff may require careful consideration. A superintendent with only minimal qualifications from an operating point of view, cannot be expected to competently supervise laboratory technicians, or chemists, or even maintenance staff. Similarly a works chemist would have the qualifications to supervise laboratory staff but not maintenance staff.

Depending on the capabilities of the staff available it may be necessary that certain works staff report to managers based outside the works; this solution will depend on the size and disposition of the operating authority.

21.3.4 TRAINING

Operators have to comply with certain statutory requirements as regards qualifications. At most works this would normally be the Operator's Diploma in Water Purification and Treatment. Training for these qualifications is obtainable at technical colleges, in various centres throughout the country. Certain technikons also offer qualifications at a higher level; this is desirable for superintendents controlling larger works.

Training is also necessary on site. New staff should be put through an orientation programme which familiarises them with the works and the operations taking place there. Thereafter they should be allocated in turn to specific jobs on the plant and learn how to operate the plant in accordance with the operating handbook and normal good operating practice. Small organisations may not have the need or the financial resources for formal training facilities. Nevertheless, some form of on-the-job training is necessary. Outside assistance should be called in if necessary.

21.4 MANAGEMENT

The functions of management consist of planning, organising, controlling and directing staff.

21.4.1 PLANNING

Planning is one of the main functions of management. The planning process should start even before the works is designed. Planning would include the water source, method of abstraction, methods of purification, size of the works, and all other aspects with regard to its design, construction and installation. This sort of planning is normally performed by a consulting engineer or specialist organisation. However, client input is invaluable at the planning stage. Planning is also required to operate the works and this is the subject under consideration in this section.

When a works is planned it is necessary to prepare for the operating situation. Apart from setting staff requirements, staff structure, and duty procedures one must also plan for emergency contingencies. Thus, a plan of action should be drawn up to deal with specific problems which may arise at the plant. This could, for example, be failure of a pump station, or of the dosage equipment, power failure, a shortage of chlorine or other chemical, or anything else which would lead to an emergency situation.

21.4.2 STAFF REQUIREMENTS

One of the functions of management is to set staff requirements and the structure of such staffing. Some aspects of this have been discussed in the previous subsection. Operation of the works obviously requires more than just operators. Maintenance staff are also required to keep the plant operational. Chemical services staff are necessary to monitor and correct treatment problems. Data processing specialists may be necessary if fully automated control via a SCADA system is installed. Management needs to develop a staff structure and organisational chart so that all these disparate elements can operate harmoniously. Whilst structures obviously need to be modified on the basis of operating experience, an initial plan is necessary prior to commissioning.

The various duties of the operating staff need to be worked out in detail. Jobs require to be structured and a job description should be prepared for each post so that every person has a clearly delineated responsibility and reporting structure. It is the task of management to anticipate all the duties which will need to be carried out once the works is fully operational, and to set these out in coherent form. Obviously, once the plant is running, unforeseen eventualities will occur and modification to the duties will probably be required. This should be done on a routine basis to update all the working programmes.

21.4.3 PREPARE PROCEDURES

Having set out the duties of the operating and maintenance staff, actual operational methods need to be worked out.

It is necessary to have fairly standardised procedures so that the work is done in a uniform and coherent manner. Thus, when receiving chemicals for example, a certain person should have signing authority to receive the chemicals, should supervise the off-loading and check that all the chemicals have been received. When preparing a batch of chemicals for dosage, certain persons should have the authority to check and ensure that the correct quantities of chemicals have been put into the batching tank and that these have been properly mixed. When changing chlorine cylinders, certain designated persons who are adequately qualified should be present to ensure that safe working procedures are being carried out. All these procedures should be reduced to simple instructions in writing so that in the absence of the superintendent or person in authority, the procedures can be adhered to in a standardised form.

21.4.4 Log sheets

It is a function of management to ensure that adequate records are kept. This starts with the log sheets which should be filled in during shift by the operators and senior operators. Log sheets should be designed to report all the important items which may require looking at, on a routine basis. If a retrospective search needs to be done on the operating history of the plant, the weather conditions, flow rate through the works, turbidity and pH of the raw water, turbidity and pH of the final purified water, quantity of water leaving the reservoir, residual chlorine in the final water, ammeter readings on certain pumps and equipment, hourly meter readings on certain parts of the plant and many other aspects, these need to be meticulously recorded.

21.4.5 Operating manual

An operating manual should be produced by the consulting engineer and detailed manuals of plant equipment should be referred to. These, however, are inadequate if they do not cover all the procedures necessary for works operation. Management should ensure that an operating manual is available which can be used to ensure that works operation is carried out consistently and coherently. The manual should be updated as plant equipment changes. This requires reducing all the tasks to their simple elements and itemising these in the working procedures.

The procedures prepared under Section 21.4.3 can be incorporated into the manual, together with such other information as is necessary for works operation. The operating manual should contain as much information as is necessary without making it so large and cumbersome that operational staff tend to be put off by its size. The manual should contain plant layout drawings and thumbnail drawings of each unit, showing essentials.

21.5 Control

Control of waterworks operation includes control of plant conditions, monitoring of log sheets and records, carrying-out of analysis of the raw and final water quality to check that operation is satisfactory, and control of the staff working on the plant.

21.5.1 Monitoring of plant condition

Part of the duties of the senior operator or superintendent should be to monitor the condition of the plant. The works should be inspected at least once per shift to establish that all equipment is operating satisfactorily and that no abnormal conditions have occurred. Inspection of the plant should include the checking the appearance and the headloss through each filter. The flocculator and clarifier or sedimentation tank should be inspected to see whether floc and clear water appear normal and whether desludging is being carried out properly. On-line indication of incoming raw-water turbidity is an advantage on works which treat water directly from the river particularly during periods of high rainfall when the water quality can change rapidly. Filters should be inspected for backwash affection, possible short-circuiting, and the quality of the sand in the beds.

The chlorination equipment should be inspected to check when the chlorine cylinder is likely to run empty and will require changing. Facilities for automatic change-over of chlorine cylinders is almost essential on water works as this ensures a continuous feed of chlorine and allows removal and replacement of cylinders after the switch-over has occurred.

21.5.2 MONITORING OF LOG SHEETS AND RECORDS

Log sheets and records are of little use unless they are inspected and acted upon. The log sheets prepared by management and filled in by the operating staff need to be inspected routinely. This should ideally be done by the superintendent as one of his first duties in the morning, to ascertain whether anything abnormal has occurred during the previous twenty-four hours. Similarly the logbook should be inspected to see whether anything unusual has occurred. Corrective action can then be taken.

21.5.3 LABORATORY ANALYSIS AND CONTROL

Laboratory analysis of water samples is necessary if the performance of the process is to be properly evaluated. To do this, a sampling schedule should be drawn up which would routinely analyse samples for various determinands.

Certain determinands such as turbidity and colour need to be measured on a frequent basis. Raw water turbidity is particularly important on works which receive raw water directly from a river. However, even in impounded supplies the turbidity of the raw water can change significantly in a fairly short time and this has major implications for the chemical doses required for coagulation and flocculation. On any works it is therefore necessary to keep track of the movement of turbidity. On relatively large works it may well be advisable to install an in-line turbidimeter to monitor changes instantaneously. On smaller works, samples should be taken for turbidity at least on a shift basis and preferably more frequently.

It is also necessary to check the residual chlorine leaving the works frequently as this is important as regards the bacteriological quality of the final water.

The residual chlorine in the final water and in the works reservoir should be measured at least once per shift and if the works is operated only on a day-shift basis, it should be measured first thing in the morning and last thing in the afternoon.

A comprehensive analysis of the water is required occasionally in order to assure that the water remains suitable for potable purposes in all respects and that no additional changes to treatment processes are required.

Before a works is even constructed the raw water should be submitted to a full SABS 241 analysis. This checks all the different determinands, including metals and trace organics to ensure that the water is inherently safe for potable use. A full SABS 241 analysis should also be carried out at infrequent intervals while the plant is in operation, say once per year. In between these two extremes a number of determinands should be measured routinely. Total and calcium hardness, alkalinity, dissolved solids or conductivity, ammonia content, chloride, sulphate, and various other determinands need to be monitored on a weekly or monthly basis, and records kept of their levels in the raw and treated waters.

Analysis and record keeping are necessary. Action also needs to be taken when certain determinands exceed or fall below certain preset action points in their range of expected values. Thus, for example, should the raw water turbidity increase substantially, steps should be taken to increase coagulant dosage. This in turn may mean that additional jar tests may need to be carried out at the works in order to estimate the increasing coagulant demand. Jar tests should also, in any event, be carried out on a routine basis to establish the treatment requirements of the water. (Refer to Chapter 6 for jar test procedures.)

Should the hardness or dissolved solids content of the water change, there is little that can be done about this but records should be available to be able to answer any queries that may arise from industrialists in the reticulated area.

The bacteriological quality of the water is extremely important. Faecal coliform counts and total plate counts should be carried out routinely on the treated water, the frequency being determined by the size of the works. In addition to this, other bacteriological indicators may be necessary from time to time. Faecal streptococcus should be measured occasionally in the purified water, and there should be an occasional check for viruses. *Giardia* and *Cryptosporidium* are also becoming of concern at various works and it may be advisable to check for these on the raw and purified waters from time to time.

If faecal coliform counts or other counts are received intermittently or frequently on the purified water, and increasing the chlorine dosage does not appear to help, it may be necessary to upgrade the purification and treatment processes or to make the necessary modifications.

21.5.4 EVALUATION OF PROCESS

The design of any works, is based on certain assumptions. In the course of time, these assumptions may no longer be valid. Thus a works designed to treat a silt-laden water would have emphasis on good coagulation, clarification and filtration facilities. In the course of time the river supply may, however, become eutrophic or a dam may be installed upstream and the water received at the works might become clearer but more highly enriched with organic material. Under these conditions modifications to the coagulation process may be necessary, pre-oxidation with chlorine or ozone may be required, and in some cases activated carbon may be necessary to remove taste and odour.

It is necessary when operating a water works to periodically evaluate the process at the plant and to assess whether this is most suitable for the water being received. Such an evaluation should be carried out on a routine, but infrequent, basis. Some changes are subtle and may not be appreciated and it is only when this type of exercise is carried out, that the need for process modifications may become apparent.

21.6 SAFETY

It is incumbent on the operating authority of any water works to ensure that safe working practices are adhered to. This is necessary not only for the obvious reasons, but also because statutory requirements are such that severe penalties will be imposed if a plant is not operated safely. The implications are further discussed below.

21.6.1 REGULATIONS

In South Africa the Act or Machinery and Occupational Safety Act has now been replaced by an OHASA the Occupational Health and Safety Act (Act 85 of 1993). Previous regulations which had already been stringent have been further tightened. Operating authorities should, as a matter of policy obtain a copy of the Act and familiarise themselves with the regulations and implications. The Act provides for safe working procedures in a large number of instances and is fairly complex. It may be beneficial to obtain the advice of specialists in complying with the regulations of the Act.

Under the Act, responsibility for unsafe conditions or procedures now devolves to the chief executive or highest levels of an operating authority and it is not unusual for the chief executive of a company to be prosecuted for an accident which takes place on his factory premises. It is therefore in the interests of all operating organisations to ensure that they comply with the requirements and operate within the spirit of the regulations.

21.6.2 NOSA

The National Occupation and Safety Association (NOSA) exists to cultivate an awareness of safety and safe working procedures. This encompasses making the work place safer, improving record keeping and complying with regulations, as well as inculcating a safe attitude to working procedures. As far as the work place is concerned NOSA encourages safe demarcation of work areas, protective equipment on machinery, safety guards, barriers, hazard warnings and all other physical measures which improve safety.

As far as record-keeping is concerned registers need to be kept of inspections on safety equipment such as ladders, scaffolding and other equipment of this nature. In addition, safe working procedures need to be followed. It is recommended that an operating authority makes contact with NOSA to avail himself of the expertise which is available.

21.6.3 HAZARDOUS CHEMICALS

Water works personnel may be exposed to hazardous chemicals under certain conditions. Gaseous chlorine which is delivered in cylinders is an extremely toxic and hazardous substance. Great care should be taken when connecting and disconnecting cylinders and operating chlorination equipment. The major chlorine suppliers in South Africa frequently conduct training courses to instruct staff who are to use chlorine in the safe use of the equipment and the hazards of chlorine. It is strongly recommended that the operating authority of any water works sends relevant staff on such courses.

Care should be taken in the event of chlorine leaks. The gas is toxic and heavier than air. Care should be taken to have all chlorine installation areas well-ventilated and to have self-contained breathing apparatus available for people who have to enter the area to close off leaking cylinders or equipment. As was mentioned previously, information on the methods of doing this safely is available from the suppliers of chlorine, and should be carefully studied.

A number of other chemicals are used on water works and these should be regarded as hazardous to a greater or lesser extent. Ferric chloride is a strongly acidic substance and should be handled with care as the material is strongly corrosive and requires equipment which is resistant to chloride attack. Goggles and gloves should be worn when loading or off-loading ferric chloride.

Aluminium sulphate is less hazardous but is also an acidic substance and should be handled with care. It is recommended that gloves and eye protection should also be worn when handling this material.

Lime is not particularly hazardous but can cause skin problems for people exposed to it on a long-term basis. Protective clothing should therefore be worn. Polyelectrolytes are mildly acidic and are toxic in large quantities; care should be taken in their handling. Most chemicals are supplied by organisations which can advise on the safe handling of the materials concerned. The operating authority should therefore contact the suppliers of materials used at water works for advice on the safe handling.

21.7 PUBLIC RELATIONS

Even a relatively small works can from time to time be affected by public relations issues. It is as well therefore for the operating authority and the staff of the works to be aware of this and to be prepared for contact with the public. Organised visits to the water works should be encouraged, especially by school children.

21.7.1 VISITORS AND TOURS

Although it normally only applies to relatively large works, even small works may be the subject of tours to demonstrate the purification processes. Tours of water works are often carried out for school children, nurses and health inspectors in the course of their training. The superintendent or suitably designated person should be available to make arrangements for such tours and to show people around the plant. A well-conducted tour, suitable literature and diagrams will serve as a good advertisement for a water works and lead to an awareness in the community of the activities which are required to produce a potable water. Larger organisations may find it necessary to have a formal organisation to handle tours and visitors.

21.7.2 INFORMATION AND PRESS RELEASES

Irrespective of the size of the works, water supply or quality problems may occasionally occur which require notification of the public. A large organisation operating a number of water works may find it necessary to have a permanent public relations officer. Smaller organisations should, however, be aware of the need to inform the media when problems are occurring and to reassure the public.

Typical examples of this are taste and odour problems caused by an algal bloom in the water course, leading to geosmin or methyl isoborneol being present in the water.

The public react negatively to unpleasant tastes and odours or coloured water fearing that the water is unsafe. This requires a speedy press release or public statement to the effect that the water is being properly treated, that it is free of hazard and can be drunk safely and that the problem is being attended to and will be corrected as soon as possible. Good public relations can create a positive image in the minds of the public.

21.8 DESIGN AND ITS EFFECTS ON OPERATION

The design of a purification works can significantly effect operation in a number of ways. Among the points of relevance are the following:

- Layout
- Access
- Weather protection
- Aesthetics
- Headloss
- Maintenance and operation
- Buildings
- Lighting
- Service water
- Instrumentation and automation
- Storage of chemicals
- Dosage of chemicals
- Operating manuals
- Plant efficiency.

21.8.1 LAYOUT

The layout of a water works should be logical in terms of flow paths and services. It is difficult and confusing to operate a plant where the flow paths are convoluted or do not follow a logical progression. The layout should also be compact so that operators do not have to walk or move equipment over excessive distance when carrying out routine procedures. The works layout should also cater for future extensions. The design of many works may originally seem good, but becomes untidy and inefficient when the capacity of the plant is increased.

Headloss through the plant and sizing of pipework, weirs and channels should be adequate to cater for all envisaged operating possibilities. It is sometimes found that flows cannot be increased through the remaining unit when one unit is off-line because of tight design or inadequate headloss. It is normally relatively inexpensive to slightly increase the size of the pipework or channels and this can greatly increase operational flexibility once the plant has been commissioned.

21.8.2 ACCESS

Access to the various items of the plant is necessary for both people and equipment. It is often beneficial to have walkways along the channels in the works following the hydraulic gradient through the works and providing access to various parts of the plant. Attempts should be made to avoid too many stairs on access ways. Walkways should be sufficiently wide to enable equipment to be transported along them and ramps should be provided to assist the moving of machinery for maintenance and repair purposes.

Sufficient space should be provided at places where it might be necessary for lifting equipment to be installed. Thought should be given to the routing of materials through the plant for maintenance purposes and for routine operation, for example, suitable access should be provided to the rapid gravity filters or pressure filters on a works so that the sand can be removed and replaced.

21.8.3 WEATHER PROTECTION

Protection against the weather may be necessary for various unit processes. This will depend on the climate in the particular part of the country where the works is situated and also on the type of process which is installed. For example, dissolved air flotation is sensitive to heavy winds, hail and heavy rain and it is therefore desirable that flotation tanks be covered to protect operation.

If the water being treated is eutrophic, the covering of clarifiers and filters will prevent the algal growth which tends to occur due to the action of sunlight on the water in uncovered units. The covering of walkways and units also facilitates the inspection of plant under all weather conditions.

21.8.4 AESTHETICS

The awareness of aesthetic considerations has increased in recent years. The proposed construction of relatively large works and even smaller works in sensitive areas should be the subject of environmental impact assessments (EIA). An EIA considers the effect of the works on the environment and produces an arrangement such that the plant has minimum impact and creates minimum disturbance to the environment.

Care should be taken to ensure that the architecture of a works fits in with the surroundings. Choice of colour and finishes should be such that the works is attractive and modern in appearance while blending with its environment. Pump stations, pipelines and water towers represent a special challenge.

21.8.5 HEADLOSS

In most systems the take-off point for the raw water supply will be fixed by technical and geographical considerations and that of the distribution areas by reason of the populations settled there. The water purification works and storage fit in between these two fixed points. For economical design the number of pumping stages should be reduced to a practical minimum as these constitute a work load to both operations and maintenance.

In the design of the water works it is normally advantageous to obtain gravity flow throughout the works. This avoids the necessity for extra pumping and sumps and gives greater operational flexibility. If, however, advanced treatment processes such as granular activated carbon columns are under consideration for retrofitting at a works it may be necessary to have an additional pumping stage.

21.8.6 MAINTENANCE AND OPERATION

Maintenance and operations can be assisted by good detailed design in the purification works. As an example, clarifiers and sedimentation tanks should be provided with scouring facilities and a good drainage system so that the tanks can be emptied in a fairly short period of time. This reduces the time that units need to be off-line and facilitates maintenance procedures.

Maintenance can also be assisted by choosing equipment which is readily available locally. If possible, equipment made locally should be installed rather than imported equipment. Certain imported plant does, however, have a proven record of availability and service. The provision of duty and stand-by pumps and equipment should be considered wherever operation of these is critical to the process. It is often not possible to carry out maintenance operations immediately, particularly at night and weekends. The ability to switch to the stand-by unit and to carry out repair work at a later stage are therefore strongly desirable.

21.8.7 BUILDINGS

Low maintenance finishes should be used on buildings unless there are specific reasons to do otherwise. Although painted finishes are attractive, they are expensive in the long run as they require frequent attention. Face brick and good off-the-shutter finishes can be attractive provided the aesthetic considerations discussed previously have been taken into account.

The works should also be designed for ease of cleaning. In some areas it should be possible to hose down and squeegee floors and walkways but this should be limited to areas where damage cannot be caused to controls and electrical or mechanical equipment. There is also the possibility that washing down certain chemicals can cause problems.

21.8.8 LIGHTING

Most water works operate for 24 hours a day and lighting therefore needs to be adequate. Powerful lighting is required over clarifiers and filters as there the conditions inside the tanks or in the filters should be clearly observable at night. Good illumination should also be provided of instrumentation and controls and particularly where there are hazards. Lighting suitable for the application should be chosen bearing in mind that certain areas will be damp and may not suit certain types of equipment.

21.8.9 WATER

Service water is needed on a water works to make-up solutions, dilute water, generate a vacuum at chlorine ejectors, occasionally hose down tanks, clean, and fight fires.

The choice will normally exist on a works between gravity and pumped pressure supply. Gravity has the advantage that once the header tank is full, water is available under pressure, even should power fail. However, providing adequate pressure and adequate volumes, may require relatively expensive construction. A chlorine ejector, for example, will generally require 250 to 400 kPa pressure.

The pressure required for fire hoses should be discussed with the supply authority to be consistent with the equipment being used.

21.8.10 INSTRUMENTATION AND AUTOMATION

A decision needs to be made about the level of instrumentation to be provided on a works. In general, one would expect that smaller works would have a relatively low level of instrumentation and automation whereas larger works might be highly automated. However, at present-day labour costs it has recently been found to be economical to automate even fairly small works. Other factors such as available expertise and capital availability then become relatively more important.

Automatic control of coagulant dosage is possible within limited ranges of raw water turbidity by using on-line streaming current detectors. Automatic control of chlorine dosage can be effected by using continuous residual measurement of concentration leaving the contact tank to control the chlorine feed rate. Automatic pH control can be carried out using an on-line pH meter to control the pH correction dosage. The use of automatic control for the various chemical feeds can save significantly on supervision. This is particularly the case when reckoned over a full 3-shift plus stand-by staffing situation.

The decision as to whether to automate a works therefore requires to be considered on the basis of staffing considerations, complexity of the works and availability of qualified maintenance staff and back-up expertise from suppliers. Suitably qualified operators and maintenance staff are essential if a large degree of instrumentation and automation is to be successful on a plant. Caution should, however, be exercised with regard to instrumentation and automation of a works, particularly if the a plant is remote from major centres and it is difficult to get expert help from agents and suppliers.

Flow measurement is required at different points on a works for different reasons. The types of meter installed should suit the application.

For sales meters which need to provide accurate record keeping, devices such as magnetic flow meters which are accurate to better than 2% would be preferred. For control of dosage on the plant however, flow meters such as ultrasonics which are normally accurate to about 5% would be adequate. Sufficient flow meters should be installed with recording and integration facilities to enable the total quantity of water entering and leaving the works to be determined by means of a mass balance. If a SCADA system is installed on the works, record-keeping can be built into the system. Otherwise chart recorders and integrator readings will be necessary for major flowmeters.

A SCADA system should be considered on works of reasonable size as this enables the plant to be controlled from one point and reduces staffing requirements. As mentioned previously, however, adequate expertise should be available to provide backup for this equipment.

21.8.11 STORAGE OF CHEMICALS

Provision needs to be made on the works for adequate storage of all chemicals used in the treatment process.

In planning storage facilities information should be obtained on the safety and other requirements for storage of the various chemicals. Incompatible chemicals should not be stored together or next to each other.

The quantity of storage provided should be based on a number of factors including estimated usage requirements, availability of supply and stability. Chemicals which deteriorate on standing should not be stored in large quantities. Sodium hypochlorite for example loses strength on standing and if this chemical is being used on the plant, arrangements should preferably be made for regular deliveries of fairly small quantities. Chemicals which keep indefinitely can be stored for longer periods. Storage facilities for up to 3 months or even longer may be required for certain chemicals if the suppliers are remote from the works or if economical order quantities warrant this. Some chemicals such as powdered activated carbon which may be necessary when taste and odour problems occur, require to be held as strategic stocks in case of need. In this case the storage facilities should be such as to minimise deterioration of the material on standing.

Depending on the size of the works, investigation into bulk storage of chemicals might be of benefit to works operation. Bulk storage is generally only of significant benefit if the material can be delivered in bulk as well. Taking delivery of the material in bags and then off-loading into a bulk silo has a labour requirement which is little different than if bag supply were used directly on the plant. Bulk storage is usually more economical as packing in small quantities adds to the cost of the chemicals. Before making a decision on bulk storage the local conditions at the works should be considered. Certain materials deteriorate on standing. Others may cake or harden on standing in silos in humid areas. The advice of the suppliers should be sought when considering the type of storage required, and factors such as locality and turnover should be stipulated.

21.8.12 DOSAGE OF CHEMICALS

The various types of chemicals used in water treatment are fed into the process in a number of ways depending on whether they are soluble or insoluble solids or whether they are supplied in gaseous or liquid form.

Coagulants and flocculants are usually dosed as solutions and this is normally carried out using dosing pumps. Aluminium sulphate is sometimes supplied as a solution and ferric chloride is always supplied in liquid form. This can be diluted or dosed directly as supplied using pumps and possibly diluted near the point of addition. Aluminium sulphate is also supplied as solid granules which require to be made up as a solution and dosed using dosage pumps. Both alum and ferric chloride are acidic and require the provision of materials which are resistant to acid attack. Ferric chloride is particularly aggressive and corrodes stainless steel. Polyelectrolytes are supplied as solutions (polyamines and DIMDAACS) where they are used as primary coagulants and in solid form (polyacrylamide) for floc aids.

Solutions of about 5% are used for coagulants and of about 0,2 - 0,5 % for the polyacrylamide. The feeding of these chemicals is normally carried out from dilution tanks but they can be added directly and diluted near the point of addition.

In dosing coagulants it is strongly preferred that a continuous feed be used rather than the pulsed dosage which generally occurs from diaphragm or plunger dosing pumps. If a pulsed delivery of coagulant is added to a raw water stream where plug flow conditions prevail, one obtains successive zones of over dosage and zero dosage. Although the average dosage may be correct the water does not flocculate well because little back mixing takes place. Continuous feed through pulse damping should therefore be carried out.

Lime dosage is normally added as a slurry. This can be from a dry feeder or from a slurry tank. Slurry from a dry feeder is satisfactory provided suitable gravity feed from a point not too remote from the dosage point is available. Lime should not be pumped for long distances through pipes as these tend to clog up and to scale up with time. The addition of lime slurry to a point below the surface of the water often causes carbonate formation which eventually blocks the feed pipe. The use of lime is preferred for pH control prior to sedimentation. After filtration soda ash or caustic soda has advantages.

Chlorine is normally dosed from gas cylinders or one tonne drums. Care requires to be taken in the design of the installation, as the gas is highly toxic and leaks can cause wide spread harm. Suitable breathing equipment should be available and on a works of any size, chlorine detectors with alarms should be fitted to the chlorine house. On smaller works it may be preferable to use sodium hypochlorite for chlorination dosage as this chemical is safer and easier to handle. The advice of chlorine suppliers should be obtained with regard to a chlorine dosage installation.

Dosage of powdered activated carbon for taste and odour control may be necessary where water is eutrophic. Powdered activated carbon dosing systems normally require fairly specialised design to prevent large scale dusting of the chemical which is difficult to handle.

The arrangements should be such as to prevent explosion hazards as the activated carbon dust can be explosive under certain conditions.

Lime can be used for pH correction prior to sedimentation where the insolubles particularly from low grade lime can be removed. After filtration the use of lime increases turbidity and causes a build-up of settleable solids in reservoirs. For post-pH correction the use of soda ash which is completely soluble is therefore preferred on most plants. It is normally added as a solution using dosing pumps. As an alternative, sodium hydroxide can be used for pH correction after filtration. Its use is theoretically advantageous as it has a greater effect at relatively low dosages. It is, however, hazardous to handle being a highly aggressive caustic alkaline substance. Its use is therefore only recommended on relatively large works where fairly sophisticated handling facilities can be provided.

21.8.13 PURIFICATION AND TREATMENT UNITS

The unit operations carried out on the water purification works are fairly standardised. However, different types of equipment can impinge on operating aspects. A number of these are discussed below.

21.8.13.1 Flash mixing

Flash mixing can be carried out in a number of ways. The designer of the plant should bear in mind the likely operating conditions when deciding which method to use. Flash mixing can be carried out either by gravity when the incoming water is passed over a weir, by using the energy of the water by passing it through an orifice plate, or by mechanical flash mixers using a chamber with a high energy mixer. The orifice plate type of unit is least favoured where variable rates of flow are to be treated as the energy dissipation varies strongly with the incoming flow. However, where constant flow rate is to be treated it can be quite successful. Flash mixing by overflowing a weir is good for variable flow rates in that roughly the same amount of headloss will occur whatever flow rate is passing through. If at least 500 mm to 750 mm of headloss is provided adequate coagulation using most chemicals takes place.

Mechanical mixers may have the disadvantage that they do not provide plug flow conditions unless the hydraulics is carefully designed. They do, however, have the advantage of operational flexibility. The amount of energy for coagulation can be adjusted by changing the motor and paddle speed to suit the chemical required. These can be quite successful particularly when using polyelectrolytes for coagulation.

21.8.13.2 Flocculation

Flocculation can be carried out using multi-pass channels, baffled chambers or tanks with motor-driven flocculator paddles. Multi-pass channels are best suited to essentially constant flow conditions as headloss and hence mixing energy are strongly flow-dependent. They have the theoretical advantage of providing plug flow conditions, thus avoiding short circuiting. Motor-driven flocculators, may not provide plug flow conditions unless particular care is taken of the hydraulics but are usually more flexible in practice. The amount of mixing energy can be regulated by adjusting the motor and paddle speed which is of assistance if water conditions change or if different chemicals are used.

21.8.13.3 Clarifiers and sedimentation tanks

Designers should ensure that the type of clarifier provided is suited to the type of water being treated. Thus, for example, blanket clarifiers should be carefully considered for treatment of highly turbid river waters. Sedimentation tanks or hopper bottomed tanks could then be preferable. Based on operating experience many clarifiers need to be cleaned out periodically. The frequency may vary between every few months or once every few years. Clarifiers should be designed in such a way that they can be emptied easily and should be provided with scour valves and draining facilities to enable them to be taken off-line fairly quickly. On larger works a suitable number of clarifiers should be provided so that it is possible to take one unit off-line for cleaning and maintenance while still maintaining production.

21.8.13.4 Filters

Slow sand filters can be provided on relatively small plants where the water quality is suitable (relatively low turbidity) and where labour is available. Normally, however, small works may be provided with pressure filters and larger works with rapid gravity filters. Pressure filters need good instrumentation and an inspection facility.

Both types of filter can operate satisfactorily for extended periods of time without special attention, provided operating conditions are satisfactory. Mud-balling of filters may occur if backwashing is not up to standard or if slime growth is taking place in the media. The facility for continuous or shock-dosing of chlorine into the water prior to filtration will usually prevent this occurring. The design of the chlorination facilities at the works should therefore allow for dosage of chlorine at different points.

Depending on the size of the works it should be possible to take one filter (or 10% of filters on a larger works) off-line for maintenance or repair purposes while maintaining production. The design of the filters should therefore have a limited reserve capacity. This takes the pressure off operations and maintenance when essential work needs to be carried out. In the event of major problems it is frequently necessary to wait weeks before essential spares can be delivered. Production obviously needs to be maintained during such periods.

21.8.13.5 Chlorination

The provision of flexibility in the points of chlorine dosage is strongly recommended. Chlorine may be needed at the head of works, between the clarifiers and filters, and after filtration. Provision should be made in the design of the chlorine installation for dosing at these three points.

The changing of chlorine cylinders is associated with certain hazards and the frequency should be minimised as far as is practicable. In a marginal case the decision should be made to install one ton drums rather than 68 kg cylinders on manifolds. Adequate lifting and delivery facilities should be provided as well as suitable weighing facilities.

21.8.13.6 Storage

Reservoirs are normally provided at the works and at various points in the reticulation system. It is recommended that the reservoir provided at the works be subdivided into two separate sub-units so that half the tank can be taken off-line for cleaning purposes on a periodic basis.

21.8.14 Operating manuals

The provision of adequate operating manuals is essential if operation is to be carried out in a controlled manner. It should be the responsibility of the project or consulting engineers who design the works to provide adequate operating instructions. It is not always possible to foresee all eventualities and the manual may need to be modified after the works has been commissioned and operated for a period of time, when operating experience has been gained. It should, however, be adequately researched so that the fundamentals are sound and that the plant can be adequately operated using the manual from the start-up situation.

It is frequently found in practice that operating manuals tend to be confined to maintenance of and instructions for the equipment being provided. Whilst this is necessary for maintenance purposes, it does not provide any guidelines to the operators themselves as to how the plant should be operated. Instructions on how to carry out routine tests, set flow rates, desludge tanks, carry out backwashing operations on the filters and how to install and remove chlorine containers, should all be included. The manual should be concise and succinct otherwise it will not be used in practice.

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List of Symbols

A	Area (m^2)
	Cross-sectional area
B	Filter ripening coefficient
b	Width
β	Bulking factor
C	Chezy coefficient
C_d	Drag coefficient
C	Particle volume concentration
C_f	Final particle concentration in water (volume concentration, mg/ℓ or turbidity units)
C_0	Initial particle concentration in water (volume concentration, mg/ℓ or turbidity units)
D	Diffusion coefficient, depth
	Water depth
d	Diameter (mm)
d_c	Critical depth
d_{10}	Sand effective size
d_{eq}	Equivalent diameter
E	Dielectric constant of water
e	2,71818
ε	Porosity
f	Porosity, orthokinetic flocculation rate
g	Acceleration due to gravity
G	Velocity gradient (s^{-1})
Ga	Galike number
	Galileo number
H,h	Head loss (m)
H′	Dimensionless headloss
λ	Filtration coefficient
I	Ionic strength
K	Kinematic viscosity
K,k	Constants
K_S	Solubility product
v	Kinematic viscosity
L	Length (m)
	Bed depth
λ	Filter coefficient
	Filtration coefficient
ℓ	Litre
M	media mass
μ	Dynamic viscosity, micrometre (micron)
	Kinematic viscosity
N	Hazen N value
n	Number of particles per unit volume or baffles in a flocculator

θ, φ	Constant
P	Power dissipated or added in coagulation and flocculation
P_O	Original number of organisms
ρ	density
π	Pi
σ	Specific deposity
$\sigma\psi$	Sphericity
ψ	Sphericity
φ	Constant
Q	Flow rate (Mℓ/d, m^3/h)
Q^1	Dimensionless flow rate
Q′	Dimensionless flow rate
q	Process loading rate
R	Radius of sphere, hydraulic radius
R_e	Reynolds number
R_{eB}	Blake Reynolds number
	Blake's modified Reynolds number
ρ	Density; with subscript l for liquid, s for solids and w for water
s	Standard deviation
	Specific deposit
T	Period of detention or temperature in °C
TDS	Total dissolved solids
THM	Trihalomethanes
t	Time (hours, minutes, seconds)
	Filter time
t_f	Filter run time (h)
UC	Coefficient of non-uniformity
V	Volume (m^3),
	Filter bed volume
v	Velocity (m/s)
v_{mf}	Minimum fluidisation velocity
W	Width (m)
x,y,z	Empirical expert
	Empirical exponent

Subscripts

e	Expanded
i	Lateral entry
l	Lateral
m	Manifold
o	Initial condition, or upstream end
s	Media
t	Total, or after time t
u	Ultimate condition
w	Water

Glossary of Terms

This glossary of terms were compiled by reference to the AWWA/WPCF/APHA/ASCE text on water and wastewater control engineering and the ASTM filtration definitions. The appropriate South African context is used especially as regards legal terms. The terminology is in alphabetical form as follows:

absorb	- To take up or to drink, such as a sponge sucks up water.
adsorb	- The act of selectively attracting and holding a gas, vapour, liquid or solid on the surface of a solid.
adsorbent	- A solid material which adsorbs, such as clay, carbon, and activated alumina.
adsorption	- The act of adsorbing.
alkaline	- The condition of water, or soil which contains a sufficient amount of alkali substances to raise the pH above 7,0.
alkalinity	- The capacity of water to neutralise acids and is imparted mainly by carbonates, bicarbonates and hydroxides, occasionally by borates, silicates and phosphates.
alluvial	- Refers to material that has been deposited by streams.
aquifer	- A porous water-bearing geologic formation. Generally restricted to materials capable of yielding an appreciable supply of water.
arable	- An area of land which has sufficient potential for sustained productivity in irrigation development.
area	- The available surface exposed to the flow of a fluid. Also referred to as surface area.
arid	- A term applied where rainfall is so deficient in quantity, or occurs at such times that agriculture is impracticable without irrigation. In climatology, it is a term applied to climates which have rainfall insufficient to support vegetation.
attrition	- Wear cased by rubbing or friction.
backwash	- To reverse flow of fluid (air, liquid) through the filter element to effect deposited solids removal.
bacteria	- A group of ubiquitous, rigid, unicellular, microscopic organisms, lacking chlorophyll.
baffle	- A plate to deviate flow entering a vessel.
basin	- The surface area within a given drainage system.
bed	- The bottom of a water course or any body of water.
bentonite	- A clay containing appreciable amounts of montmorillonite.
berm	- A horizontal strip or shelf built into an embankment or cut to break the continuity of an otherwise long slope, usually to reduce erosion or to increase the thickness or width of cross section of an embankment.
biota	- Animal and plant life, or flora and fauna, of a stream or water body.
blind spot	- A place in a filter where little or no filtration takes place.
blinding	- The reduction or shutting off of flow due to solid particles filling the openings in the filter media or septum.
blowdown	- The use of pressure to remove liquids and/or solids from a vessel.
bridging	- Particles being removed, arch over individual openings in the filter septum or between the individual filter elements.
cake	- Solids deposited on the filter medium.
canal	- An open channel constructed to transport water.

capacity	The quantity that can be contained exactly or the exact rate of flow.
carbonate alkalinity	Alkalinity caused by carbonate ions.
carbonate hardness	Hardness caused by the presence of carbonates and bicarbonates of calcium and magnesium.
catchment	The area tributary to a stream or lake.
clarity	Clearness of a liquid as measured by a variety of methods.
clay	A natural occurring material, usually being activated and used as an adsorbent.
coagulation	Aggregation of microparticles into microfloc, by charge neutralisation.
coagulant	A substance added to destabilise colloids in a stable suspension.
colloid	Finely divided solids, less than 0,001 mm in size; intermediate between a suspension and a true solution.
crest	The top of a dam, dike, spillway or weir.
cycle	Filtration interval; length of time filter operates before cleaning.
dam	A barrier constructed across a watercourse for the purpose of creating a reservoir.
demand	Used in conjunction with modifying terms, e.g. biochemical oxygen, chlorine, oxygen, water.
density current	A flow of water through a larger body of water, retaining its unmixed identity because of a difference in density.
depth filtration	The use of a deep bed of medium to remove particulate matter by filtration.
diatomaceous earth	Diatom skeletons of silica in the form of finely particles; used in cake form as a filter medium.
discharge	The rate of flow of water in a stream or conduit at a given place and within a given period of time.
disinfection	Killing or inactivation of the larger portion of microorganisms, with the probability that all pathogenic bacteria are killed by the agent used.
earth dam	A dam, the main section of which is composed principally of earth, gravel, sand, silt and clay.
ecology	The branch of biology dealing with the relationships between organisms and their environment.
effective storage	The volume of water available for a designated purpose.
effluent	Liquid or wastewater flowing out of a reservoir, a sewer or a purification works.
erosion	Wearing away of land by running water and by wave action.
evaporation	The process by which water becomes a vapour at a temperature below boiling point.
evapotranspiration	Water withdrawn from soil by evaporation and plant transpiration.
feed	The mixture of particles and fluid that is deposited on a septum in a diatomaceous earth filter.
filter	A filtration device or a structure consisting of supported filter medium and equipment for the subsequent removal of the deposited solids.
filtration	To filter i.e. the operation of passing a liquid through a porous or open textured medium in order to remove suspended particles by leaving them behind in the pores of the filtering medium or at the surface of the filter.
filter aid	A material added to improve the effectiveness of filtration.

filter medium - The permeable material that separates particles from a fluid passing through it.

flocculation - Aggregation of micoflocs into flocs by collisions due to hydrodynamic operation.

flocculant - Substances added to a destabilised suspension to accelerate flocculation, densify or strengthen flocs.

flood - A relatively high flow as measured by either gauge height or discharge quantity.

flow rate - The unit rate at which a product is passed through a system.

Fuller's earth - Clay i.e. a hydrous aluminium silicate.

geohydrology - The branch of hydrology relating to subsurface or subterranean waters.

geology - The science that deals with the origin, history and structure of the earth, as recorded in the rocks, together with the forces and processes now operating to modify rocks.

groundwater - Subsurface water occupying the saturation zone from which wells and springs are fed.

hardness - A characteristic of water, imparted by salts of calcium, magnesium and iron such as carbonates, bicarbonates, sulfates, chlorides and notates, that causes e.g. curdling of soap and deposition of scale in boilers.

head - The height of the free surface of fluid above any point in a hydraulic system; a measure of the pressure or force exerted by the fluid.

hydraulics - That branch of engineering science which deals with water or other fluid in motion.

hydrogeology - The branch of hydrology that deals with groundwater, its occurrence and movements, its replenishment and depleting, the properties of rocks that control groundwater movement and storage, including for the investigation and use of groundwater.

hydrologic cycle - The circuit of water movement from the atmosphere to the earth and return to the atmosphere through various stages or processes such as precipitation, interception, runoff, infiltration, percolation, storage, evaporation and transpiration.

hydrology - The engineering science concerned with the waters of the earth in all their states, including the physical, chemical and physiological reactions of water with the rest of earth and its relation to the life of the earth

hydrophilic - Water accepting - antonym is hydrophobic.

hydrostatic pressure - The total force or force per unit area, exerted by a body of water at rest.

hyetology - The science which deals with rainfall.

impermeable - Not allowing, or allowing with only great difficulty, the movement of water; impervious.

impounding reservoir - A reservoir wherein surface water is retained for a considerable period of time but is released for use at a time when the ordinary flow of the stream is insufficient to satisfy requirements.

infiltration - The absorption, flow or movement of water into or through the interstices of soil or porous medium.

inorganic - Substances which are not of basically carbon structure.

interstice - A pore or open space in rock or granular material, not occupied by solid matter.

ion - A charged atom, molecule, or radical, the migration of which affects the transport of electricity through an electrolyte or, to some extent through a gas.

isostatic - Subject to equal pressure from every side; being in isostatic equilibrium.

jet - The stream of water under pressure issuing from an orifice, nozzle, or tube.

laminar flow - The flow of a viscous fluid in which particles of the fluid move in parallel layers each of which has a constant velocity but is in motion relative to its neighbouring layers; streamline flow.

Langelier index - The hydrogen ion concentration that a water should have to be in equilibrium with its content of calcium carbonate.

leach - To cause water to percolate through something.

leakage - The uncontrolled loss of water from artificial structures as a result of hydrostatic pressure

limnology - Scientific study of fresh water, in lakes, ponds, and streams, with reference to their biological, geographical, physical and other features.

liquid - A substance that flows freely, by movement of the constituent molecules, but without the tendency for the molecules to separate from one another.

marsh - A tract of soft, wet land, usually vegetated by reeds, grasses and small shrubs.

mean annual precipitation - The average over a period of years of the annual amounts of rainfall, MAP.

mean annual runoff - The average over a period of years of the annual amounts of runoff discharged by a stream; MAR.

micron - The unit of length equivalent to 10^{-6} m.

micro-organism - Minute organism, either plant or animal, invisible or barely visible to the naked eye.

milligrams per litre - A mass/volume unit of concentration of constituents of water or wastewater.

mineral - Any substance that is neither animal nor vegetable; obtained by mining.

nappe - The sheet of water overflowing a weir or dam.

normal flow - The flow that prevails for the greatest portion of time; the mean or average flow.

organic - Chemical substances of animal or vegetable origin, basically carbonaceous.

orographic - Precipitation caused by the interference of rising land or air in the path of moisture laden wind.

oxidation - The addition of oxygen to a compound or a reaction which results in loss of electrons from an atom.

parts per million - Milligrams per litre.

peak - The maximum quantity that occurs over a short period of time.

percolate - When water, under the force of gravity, passes through soil or rocks, along the line of least resistance.

perennial - A stream flowing through all seasons of the year, during wet and dry years.

permeable - A material permitting appreciable movement of water through when it is saturated, under normal hydrostatic pressure.

pH - The reciprocal of the logarithm of the hydrogen ion concentration.

phreatic	The upper boundary of the water table in soil; the phreatic line lies between the capillary zone and the saturation zone.
porous	Having small passages; being permeable to fluids; porosity is the ratio of volume of voids to bulk volume.
precoating	The deposition of a filter aid on the surface of a filter or septum before filtration takes place.
purification	The removal of undesirable or objectionable matter from water by natural or contrived means - an extractive process.
recycle	The return of clarified liquid for another cycle of phase separation.
septum	A permeable material used to support filter medium.
supernatant	Liquid above settled solids.
reservoir	A basin, lake, pond, tank or other space which is used for storage; impoundment.
river	A large stream of water that serves as the natural channel for the drainage of a basin of considerable area.
river basin	The area drained by a river and its tributaries.
rock	(1) In engineering, a natural aggregate of mineral particles connected together by strong, permanent forces of cohesion.. (2) In geology, the material that forms the essential part of the earth's solid crust (3) In groundwater hydrology, all formations in the lithosphere.
saturation	A condition reached by a material holding another material in an amount such that no more can be held within it in the same state.
sediment	Solids, settled from suspension in a liquid.
sedimentation	The process of deposition of sediment from water.
semi-arid	Neither entirely arid nor strictly humid, but with a tendency toward an arid character.
sewage	The spent water of a community - wastewater.
silt	Soil particles of diameter 0,004 to 0,062 mm.
soil	Earth material that has been so modified and acted upon by biological, chemical and physical agents that it will support rooted plants.
storage	The impounding of water, either in surface or in underground reservoirs, for future use.
stream	A course of running water, flowing in a definite channel, in a particular direction.
synergism	The improvement in performance achieved because two agents are working together.
turbidity	A measure of the scattering and absorption of light rays, caused by the presence of fine suspended matter.
turbulence	The fluid property caused by irregular variation in the speed and direction of movement of individual particles or elements of the flow.
unaccounted-for water	Water taken from a source into a distribution system, but is not delivered to the consumers or otherwise accounted for.
unit process	In which chemical changes take place.
unit operation	In which physical changes take place.
water consumption	The consumptive use of water for any particular purpose.
water demand	A schedule of water requirements for domestic, industrial, irrigation, mining and power generation.

water purification	- The removal of undesirable substances from water.
water requirement	- The need for water, e.g. for plants, recreation and the environment.
watershed	- The divide between drainage basins.
water supply	- In general, the sources of water for public or private uses.
water treatment	- The addition of substances to water in order to render it more suitable for any particular purpose.
water works	- As defined under the Water Act, 54 of 1956.
yield	- The quantity of water, expressed as a rate of flow, that can be collected for a given use, from surface or groundwater sources on a watershed. Yield would vary with the proposed use, assurance of supply andeconomic considerations.
Zeta potential	- Directly proportional to electrophoretic mobility of a particle and a measure of colloid stability.

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