Process Design Manual For Small Wastewater Works

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Water Research Commission

PROCESS DESIGN MANUAL FOR SMALL WASTEWATER WORKS

Report to the
Water Research Commission

by

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on behalf of
Waterscience CC

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EXECUTIVE SUMMARY

INTRODUCTION

The official starting date of this project was 1 April 2006 and the project was scheduled for completion in May 2008. However, due to the Project Steering Committee requesting the inclusion of material that was additional to that specified in the original proposal, the completion date for the project was extended to September 2008.

The motivation for conducting such a project was based on the fact that 'A Guide to the Design of Sewage Purification Works' was first published in 1973 by the then Southern African Branch of the Institute for Water Pollution Control (IWPC) and over the years this useful reference document has become known as the 'Black Book'. This guide was revised and republished in 1987, the same year that the IWPC became the Institute of Water and Environmental Management and the same year in which the Water Institute of Southern Africa was formed.

A major decision taken by the Branch Council was that the revised publication should be applicable to sewage works treating up to 5 Mℓ/day of domestic sewage as there was concern that the revised book should not be used as a type of "do-it-yourself manual" by people lacking adequate expertise to design such a works. The purpose of the 1987 revised publication was to update outdated information, include new processes and provide the information in a more user-friendly manner. The 1987 revision of the Manual was intended to be less of a guide to design, and more of a manual to assist firstly designers, and secondly engineers and/or chemists who may be required to approve the designs for smaller domestic sewage works treating up to 5 Mℓ/d.

Since publication of the revised edition of the Manual for Design of Small Sewage Treatment Works in 1987, no further revision has been carried out, despite that fact that since then there have been a number of new technologies introduced into wastewater. It is obvious that the manual is in need of updating as several new processes are available and the understanding of the activated sludge process, in particular, has advanced significantly since then. The manual also does not consider processes used in small plants of the package plant type as are commonly used in housing complexes. It is clear that with technological advances and a number of changes in the procedures used in plant operation, a new manual, covering these changes, is required. Another change to the manual is that the maximum capacity of 5 Mℓ/d stipulated in previous versions is considered arbitrary and irrelevant. Although the manual is designed with smaller wastewater plants in mind, it needs to be borne in mind that much of the information and design calculations are applicable to much larger plants and not confined to works of 5 Mℓ/d or less. Many larger plants have been built in stages over extended periods of time, resulting in plants consisting of a number of smaller modules. In such cases this manual could be of particular relevance. The manual is however written with smaller wastewater works in mind and is intended to assist in designing, approving and operation of these smaller plants.

AIMS

This aims of this project were to:

- Produce a preliminary manual after sourcing literature sources and extracting information from these literature sources. Consult with other specialists in fields where this is considered necessary.
- Produce an internally edited and reviewed draft manual for submission to a steering committee for peer review
- Produce the final Design Manual for Small Sewage Treatment Plants including the comments, recommendations etc. obtained after peer review of the draft manual
- Conduct workshops to disseminate the information contained in the Manual to coincide with WISA 2008.

METHODOLOGY

The initial part of this project involved review of the existing manual. It was proposed to update this manual wherever necessary, particularly the biological filter and activated sludge chapters, and to add chapters on rotating biocontactors (RBC) and submerged media reactors.

Once the manual had been comprehensively reviewed, a thorough literature study was conducted to ensure that wastewater treatment practices used both locally and internationally were covered. This made it possible to identify the key issues that needed to be addressed in compiling a new manual and also highlighted any gaps that existed in the previous manual. The regulatory aspects and new legislative requirements were carefully considered in terms of how they were expected to impact on this manual. A final draft of the manual was proof-read by A Pitman, to check for content and errors, before being circulated to the Steering Committee members for comment.

ACKNOWLEDGEMENTS

The research in this report emanates from a project funded jointly by the Water Research Commission r and entitled:

Design Manual for Small Sewage Treatment Works.

The Steering Committee responsible for this project consisted of the following persons:

Dr HG Snyman	:	Water Research Commission (Chairperson)
Mr DJ Nozaic	:	Waterscience CC (Project Leader)
Mrs SD Freese	:	Waterscience CC (Researcher)
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The financing of the project by the Water Research Commission and the contribution of the members of the Steering Committee are gratefully acknowledged.

The assistance of Mr A Pitman in proof reading the document is also gratefully acknowledged.

A number of diagrams and photographs used in this manual were obtained from the eWISA website. Gareth McConkey and the rest of the eWISA team are gratefully acknowledged for the use of this material. The eWISA website provides excellent information on a wide variety of topics dealing with water and wastewater and is a highly recommended site for anyone involved in the water and wastewater fields. Visit www.ewisa.co.za.

Alistair Hunter is thanked for assisting in obtaining some of the excellent quality photographs obtained in this manual.

Narina Ramdhaw is thanked for her excellent drawings which appear in Chapter 7.

Kim Hodgson is also acknowledged for ensuring that references made to legislation that is currently under revision (Regulation 2834 of the National Water Act) was correct at the time of publishing.

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GLOSSARY

BR	Anaerobic Baffled Reactor
ADWF	Average Dry Weather Flow
BNR	Biological nutrient removal
BOD	Biochemical Oxygen Demand
Bv	BOD Loading
COD	Chemical Oxygen Demand
d	Day(s)
DAF	Dissolved Air Flotation
DO	Dissolved oxygen
DWAF	Department of Water Affairs and Forestry
E. coli	*Escherichia coli*
EIA	Environmental Impact Assessment
FFR	Fixed Film Reactors
F/M	Food to micro-organism
Ft^3/Mgal	Cubic feet per mega gallon
g/Cap.d	grams per capita per day
g/m^3.d	Grams per cubic metre per day
IWPC	Institute for Water Pollution Control
kg	Kilograms
kg/d.h	Kilograms per day per hectare
kℓ	Kilolitre
kℓ/d	Kilolitre per day
kW	Kilo Watts
kW/kg.d	Kilo Watts per kilogram per day
ℓ	litre
ℓ/m^2.d	Litres per square metre per day
m^3	Cubic meters
MBR	Membrane Bioreactor
m^3/d	Cubic meters per day
MF	Microfiltration
mg/ℓ	Milligrams per litre
m/h	Metres per hour
m^3/ha.d	Cubic meters per hectare per day
Mℓ	Megalitres
Mℓ/d	Megalitres per day
mℓ/ℓ	Millilitres per litre

MLSS	Mixed liquor suspended solids
MLVSS	Mixed liquor volatile suspended solids
mm	Millimetres
mm/s	Millimetres per second
m^3/min.m	cubic metres per minute per metre
m/s	Metres per second
NH_4-N	Ammonia nitrogen
P	Phosphorus
OA	Oxygen Absorbed
O&M	Operation and Maintenance
PDWF	Peak Dry Weather Flow
PST	Primary settling tank
PV	Permanganate Value
PWWF	Peak Wet Weather Flow
RAS	Return activated sludge
RBC	Rotating Biological Contactor
RDU	Rotating Disc Unit
rpm	Revolutions Per Minute
RSS	Return Sludge Concentration (Clarifier underflow)
SABS	South African Bureau of Standards
SAR	Sodium Adsorption Ratio
SBR	Sequencing batch reactor
SNL	Supernatant Liquor
SRT	Solids Retention Time
SS	Suspended solids
SSVI	Stirred specific volume index
SVI	Sludge volume index
SWD	Sidewater depth
TKN	Total Kjeldahl Nitrogen
TWL	Top Water Level
µm	Micro metre
UF	Ultrafiltration
USAB	Upward Anaerobic Sludge Blanket
UV	Ultraviolet
WAS	Waste activated sludge
W/m^3	Watts per cubic metre
WWTP	Wastewater Treatment Plant

CHAPTER 1

1. INTRODUCTION

1.1 INTRODUCTION

'A Guide to the Design of Sewage Purification Works' was first published in 1973 by the then Southern African Branch of the Institute for Water Pollution Control (IWPC) and over the years this useful reference document has become known as the 'Black Book'. This guide was revised and republished in 1987, the same year that the IWPC became the Institute of Water and Environmental Management and the same year in which the Water Institute of Southern Africa was formed. This became known as the 'Green Book'.

A major decision taken by the Branch Council was that the revised publication should be applicable to sewage works treating up to 5 Mℓ/day of domestic sewage as there was concern that the revised book should not be used as a type of "do-it-yourself manual" by people lacking adequate expertise to design such a works. The purpose of the 1987 revised publication was to update outdated information, include new processes and provide the information in a more user-friendly manner. The 1987 revision of the Manual was intended to be less of a guide to design, and more of a manual to assist firstly designers, and secondly engineers and/or chemists who may be required to approve the designs for smaller domestic sewage works treating up to 5 Mℓ/d.

No further revision has been carried out since publication of the revised edition of the Manual for Design of Small Sewage Treatment Works in 1987, , despite that fact that since then there have been a number of new technologies introduced into wastewater. It is obvious that the manual was in need of updating as several new processes are available and the understanding of the activated sludge process in particular, has advanced significantly since then. The 1987 edition of the manual also did not consider processes used in small plants of the package plant type as are commonly used in housing complexes. It is clear that a new manual, covering these changes was required, .with technological advances and a number of changes in the procedures used in plant operation.

Another change in this new edition of the manual is that the maximum capacity of 5 Mℓ/d stipulated in previous versions is considered arbitrary and irrelevant. Although the manual is designed with smaller wastewater plants in mind, it needs to be remembered that much of the information and design calculations are applicable to much larger plants and not confined to works of 5 Mℓ/d or less. Many larger plants have been built in stages over extended periods of time, resulting in plants consisting of a number of smaller modules. This manual could be of particular relevance in such cases. The manual is however written with smaller wastewater works in mind and is intended to assist in designing, approving and operation of these smaller plants.

1.2 HOW TO USE THE DESIGN MANUAL

Even a highly-qualified engineer would not attempt to design a car or an aeroplane by reading a book. Even a large number of books on the subject would not provide the background know-how which is garnered by experience over a period of time. One would also need to have access to the tools of the trade that may be essential to the design process.

Whilst a wastewater works is relatively simple conceptually, the same argument can be applied. Designing a plant requires experience and know-how which needs to be accumulated by exposure to the field over a period of time, and using a single reference book to design a plant would be a risky approach.

Like its IWPC predecessors (the Black Book and the Green Book) this manual does not, therefore set out to be an all-embracing guide to design. There are many other comprehensive works and manuals of practice that could and should also be consulted. However it does provide an overview of South African practice and design criteria.

The manual is intended to be a guide to persons already in the field, who may use it for cross-checking purposes, and a starting point to persons interested in wastewater treatment and would like to be involved in works design.

It could also be a useful reference for persons who have a need for a plant and who have had a design presented to them for approval. With a few quick checks it should be possible to assess if the plant appears to be about the right size, and whether suitable process selection has been carried out.

It would also be of use as a prescribed book for a number of the higher-level courses being offered in wastewater treatment, where a brief design overview is to be included.

CHAPTER 2

2. DESIGN PRELIMINARIES

2.1 GUIDE TO ESTABLISHING A WASTEWATER WORKS

2.1.1 INTRODUCTION

It is not feasible to just go ahead and design and build a sewage or wastewater works. Permission has to be granted by a number of authorities before work can commence. If the works is in a municipal area it will be necessary to apply to the municipality for permission. Compliance with various regulations and planning provisions will be needed and approval may be subject to negotiation over a fairly extended period of time.

The Department of Water Affairs and Forestry (DWAF) will also have to be approached for approval and permission. The formalities are less stringent for small works (less than 2 Mℓ/d) where the General Authorisation limits apply, but there are still procedures that need to be met. If DWAF has delegated some of its powers to the Municipality this will be an additional step that has to be gone through with them.

DWAF (and/or the Municipality) will be interested in the treatment sequence proposed, in order to satisfy themselves that the effluent will comply with the required limits. In certain catchments listed by DWAF there are higher standards applicable (Special Standard) which may be applicable to designated conservation areas, or there may be phosphate limits on the effluent where eutrophication is a downstream problem. Irrigation use of effluent also has different requirements.

In addition to all of this an EIA (Environmental Impact Assessment) is required, regardless of the size of the plant, and this involves compiling an EIA and submitting it to the Provincial Department of the Environment.

The negotiations may be time-consuming and complex and it may well be advisable to obtain the assistance of a specialised consultant, if this is not a routine activity for you or your organisation.

2.1.2 EFFLUENT STANDARDS

The effluents from wastewater plants have to comply with Gazetted Standards for Discharge as set by DWAF. The General Authorisation limits apply to works of 2 Mℓ/d and less. Larger plants have to have a survey and report done on the potential effects of effluent discharge on the river, and in certain cases, more stringent limits may apply.

The provisions of some of the applicable regulations extracted from the relevant Government Gazette are given later in this document.

2.1.3 SLUDGE UTILISATION AND DISPOSAL STANDARDS

A WRC project was established in which all role players could contribute following a certain amount of controversy over the years as to the applicable standards for sludge application to land.,. The result of this has been the publication of a five part set of guidelines which set out permissible disposal routes for sludges of various qualities depending on the level of treatment and stabilisation. This guide has been adopted by DWAF and the Department of Health, and copies are available from the WRC

2.1.4 MAINTENANCE

The selection of processes for a works should take into account the level of infrastructure that exists at the proposed site. There are many plants in existence that have been poorly selected through specification of relatively sophisticated machinery and processes. In many country areas there are virtually no maintenance contractors or facilities available, and plants need to be kept as simple as possible. Sophisticated PLC controls and automatic valves may reduce the need for operator input but they increase the need for high level maintenance. Many plants as a result cease to perform effectively after a period of time.

2.1.5 OPERATOR LEVEL

The more sophisticated a plant, and the larger the works to be operated, the higher will be the works classification and the higher will be the level of skills and qualifications required by the process controller and supervisor operating the works. The Department of Water Affairs and Forestry (DWAF) has had a system of Works and Operator classification in place for many years. It is currently in the process of revision. Until now this was governed by Regulation 2834 of the National Water Act, but in future will fall under Regulation 17 under Section 9 of the Water Services Act. Works designers should be aware that the processes they select will entail certain requirements in terms of process controller education and experience and ongoing training requirements which will need to be complied with for the process controller to obtain a licence and to operate the works. Process controlling can be considered to be a scarce skill, and may be virtually unobtainable in rural areas.

2.2 POPULATION AND PROJECTIONS

2.2.1 INTRODUCTION

For correct sizing of a works it is necessary to have good estimates of existing or likely wastewater flows and biological loads over the period encompassed by the design. When the design is an extension or refurbishment of an existing plant one is usually able to access some data regarding flows and wastewater strengths as a basis for process sizing, but for new plants, in areas which are still in the process of planning or development, it is necessary to make a number of design assumptions.

In the case of residential areas or developments where there is no existing data, the best estimates of flow and load are usually based on population projections. The developers of the residential areas or estates will have an overall plan of the number of stands and for smaller developments may even have knowledge of the size of houses to be built, or the number of bedrooms in the flats or apartments planned. From this it should be possible to obtain a reasonable estimate of the likely population and in turn from this, an estimate of likely wastewater flows and biological loads.

Population and flow projections for areas served by a wastewater treatment plant should be made before sizing of treatment processes and piping. Where possible, designs should be based on a 10-year design period for any one phase of construction. However shorter periods or staged developments often need to be implemented to match predicted growth patterns. In considering staged development the ultimate development of the collection area should be assessed to determine how the layout for the plant may appear if the area is fully developed.

The population per dwelling unit depends on a number of factors. Obviously the size of the house is relevant and a four-bedroom house is likely to have a higher occupancy than a one or two bedroom unit. The area is also relevant. A holiday area is likely to be much closer to being fully occupied in season than a development which houses permanent residents. Income level and family size are also important. Many lower income areas have larger families than high income areas and populations of 6 to 8 persons per household are common in poorer areas compared to 4 persons per unit in a high income area.

Population projections should be based on historic growth in the catchment area, together with projected housing and industrial developments and the programmes of construction. The population projections should also take into account non-permanent residents and seasonal changes in populations (that is, heavy tourist areas or commercial areas). The sum of the non-permanent and the permanent population is considered the functional population and is the basis for design flow. Although a seasonal population may be present in a service area for only a portion of the year (for example, the summer months, or the winter months), it may have a significant effect on the waste-water flow treated by the plant. If large fluctuations in flow are expected, the design may need to include provisions for isolating parts of the plant during periods of low flow, and bringing these back into operation as the need arises during periods of high flow or organic loading.

Consideration of service area characteristics should be included when estimating flow per capita. When available, water consumption for an area should be used to estimate wastewater flow generation. At least 50 to 80% of the water consumption reaches the sewer system (the lower percentage is applicable in semi-arid regions). If water consumption data are not available or an area is undeveloped, an estimation of flow per capita can be used to generate expected wastewater flow. This is discussed in the following section (Chapter 2, Section 3).

When estimating population one therefore needs to establish the total number of residences, and to allocate a population per unit depending on income level, size of units, and type of development. This then forms the basis for estimating the flows and biological loads which will be used to size the works.
In purely residential areas the wastewater will consist of domestic sewage, the strength of which is related to the demographics of the area. The water consumption per capita depends on income level so that sewages from low income areas are of higher strength than those from higher income areas. This aspect is discussed later (Chapter 2, Section 4).

Commercial areas may contribute significantly to flow, yet not have a significant population count., The type of industry or commercial business should therefore be identified whenever possible. If the area is zoned for commercial use but is not developed, an estimate based on the wastewater generation from similar businesses in the surrounding area offers a starting base for flow projections.

In mixed areas such as where a new town or city is being planned there will be mixed use of water and the possibility of effluent from wet or dry industries contributing to the wastewater mix. Usually some information will be available regarding this, as new towns are generally built for a purpose such as to serve a mine or a large industrial development, and assumptions can be made to estimate likely non-domestic flows. Collaboration with the designers of the potable water works or water supply authority can avoid duplication in such cases and ensure that the water supply capacity and the wastewater treatment capacity are matched.

New developments are sometimes planned to be constructed over extended periods which may be five, ten, or even twenty years. Whilst a good argument can be made for building a plant with the ultimate capacity for a five year development, it is usually more appropriate to build works for longer-term developments in stages or modules. The number of stages required will depend on the programme and the rate of population or flow increase, balanced by the cost penalties incurred in building a works in a relatively large number of small stages. The optimal number of stages of development is therefore decided by a combination of economic and engineering judgements together with input from the client as regards affordability over the development period.

2.3 FLOW ESTIMATION

2.3.1 INTRODUCTION

When designing a new sewage works or extending an existing one, the design should be based on the flow rate and sewage strength. Estimation of the flow rate is therefore a critical factor in design. If an existing works is to be extended and the flow records and plant performance data of the plant are available, then careful assessment of the rate of flow, daily and seasonal fluctuations in flow rate and sewage strength and operational data for the various process units can be used to determine the design parameters of the extended plant. However if no reliable data are available, as is often the case in designing new works, then estimates have to be made. This chapter provides guidelines in estimating flow for design purposes. This is achieved by establishing the size of the population for the contributing area to the works, thereby making it possible to obtain a reasonable estimate of likely flows to the works.

2.3.2 ASSESSMENT OF WASTEWATER FLOW

2.3.2.1 Domestic Sewage

This component is normally the major contributor to the total flow and a reliable estimate is therefore important. There are various ways in which this may be determined. Domestic sewage flow may be expressed in terms of litres/household.day. Typical figures for fully-sewered housing units with bathrooms, basins and kitchen sinks, taken from SABS Publication 0252-2 are given in **Table 2.3.1** below. **Table 2.3.2** then gives reduced flows for dwelling units that do not have full in-house water reticulation.

TABLE 2.3.1: Typical figures for flow estimates for fully-sewered households.

Description	Sewage Flow ℓ/person/day
Low income group:	
Per dwelling unit, or	500
Per person per dwelling unit	70
Middle to upper-income groups:	
Per person per dwelling unit, or	160
Dwellings with 2 bedrooms	750
Dwellings with 3 bedrooms	900
Dwellings with 4 bedrooms	1 100
Dwellings with 5 bedrooms	1 400
Dwellings with 6 bedrooms	1 600
1) An allowance of 15% for storm-water infiltration and other contingencies should be incorporated in the design figures to be used for dwelling houses	

TABLE 2.3.2 : Typical flows for dwellings without full in-house water reticulation

Level of water supply	**Sewage flow** ℓ/person/day
Public street standpipes	12 to 15
Single on-site standpipe with dry sanitation system	20 to 25
Single on-site standpipe with a WC pan connected to water supply	45 to 55
Single in-house tap with a WC pan connected to water supply	50 to 70

Flows in excess of those tabled above are likely where water supplies are not metered, while lower flows will prevail in residential areas without normal internal plumbing in the dwellings. As an alternative it has been established in many areas that a figure of about one kℓ/d per stand for fully reticulated sites provides a reasonable estimate of flow. No reliable correlation between the sewage flow and the number of sanitary fittings in a dwelling has yet been established.

Water consumption records, where available, are a possible alternative basis for estimating domestic sewage flows. About 70% of domestic water consumption may be expected to reach the sewer; but this percentage will be higher from dwellings where there are no gardens, and lower where there are large gardens, especially if lawn sprinklers are used. Some areas have a return as low as 40% to 50%

Flows from other units contributing to domestic sewage are estimated as listed in **Table 2.3.3.** and are taken from SABS publication 0252-2.

In providing for the full design life of a sewage treatment works the upward trend in per capita sewage contribution should be kept in mind and an annual increase of 1.5% to 2.5% could be expected in below-average areas. Persons intending to design wastewater plants and appurtenances would be well advised to obtain the relevant SABS publication as, apart from flow guidelines, it also contains much of the necessary detail requirements.

2.3.2.2 Other Criteria

Guidelines for expected flows and loads are also available from other publications in this country such as Government Departments. In addition there are criteria from the UK, Europe and USA. These can be helpful but care is needed in applying them as different lifestyles and per capita consumptions may exist. In the section on load estimation (Section 2.5), tables from UK (British Water) and USA (Washington State) are given as examples.

TABLE 2.3.3: Typical figures for daily sewage flow estimates for units other than households.

Type of establishment	Unit	Daily sewage flow ℓ/unit
Airports	Passenger	10
Bars	Customer	8
Boarding houses	Person	110
(Additional kitchen wastes for Non-resident boarders)	Person	23
Cocktail lounges	Seat	70
Country clubs	Visitor	370
	Employee	50
Day schools	Student	37
Department stores	Toilet	1 850
	Employee	40
Dining halls	Meals served	30
Drive-in theatres	Car space	9
Factories (exclusive of industrial waste)	Worker / shift	140
Hospitals, medical	Bed	500
	Employee	40
Hospitals, mental	Bed	400
	Employee	40
Hotels without private bathrooms	Person	110
Hotels with private bathrooms	Person	140
Motels	Bed	90
Offices	Worker / shift	70
Restaurants (toilet and kitchen wastes)	Person	20
Service stations	Parking space	10
Shopping centres	Employee	5
Swimming baths	Person	9
Theatres	Seat	10
Tourist camps or caravan parks with central bathhouse	Person	90

2.3.2.3 Non- Domestic Wastewater

Although this design manual is intended for relatively small works treating mostly domestic sewage the situation often arises, even on small plants, that there is a significant non-domestic component. This applies to settlements or towns established to serve a single factory, mine, sugar mill, brewery, or agricultural industry, or a relatively small industrial township. In this case, the proposed industrial activity needs to be established together with planned production levels and the expected water consumptions and effluent generation. It will then be necessary to consult the literature for the specific industry or industries involved to establish likely wastewater flows and loads and characteristics. Information should be available from the industrial entrepreneurs and there are a number of useful publications in this field. The series of WRC reports in the published in the NATSURV series some years ago would still be useful in this regard. The nature of the non-domestic wastewater component needs to be carefully considered as it may well have a major effect on process selection and plant sizing

2.3.3 FLOW VARIATION

The flows given above are average flows. On top of this there is a diurnal variation over 24 hours and variation during wet and dry conditions. Infiltration of groundwater and ingress of surface stormwater into the sewerage system will affect the rate of flow significantly. Hence flow recording during both wet and dry weather conditions is necessary to establish peak factors.

2.3.3.1 Average Dry Weather Flow (ADWF)

The flow used when designing treatment units will be the total domestic sewage flow and is known as the ADWF and is measured in kℓ/d. The figures given in the tables above represent ADWF.

2.3.3.2 Peak Dry Weather Flow (PDWF)

The normal daily peak flow in dry weather (PDWF) exceeds the AWDF by a factor that is generally highest when the population and the area served are small. The larger the town and the more mixed the population the lower the likely peak factor. Conversely in a settlement serving one industry where nearly everyone follows the same timetable, the peak factor is likely to be high.

The following formula is often used to calculate the factor:

$$\text{Peak Factor} = 1 + \frac{14}{4+\sqrt{p}} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.(2.1)$$

where p is the equivalent population (1000's) contributing to the system.

The Table 2.3.4 below gives typical peak factors. Peak flows are typically measured in ℓ/s.

TABLE 2.3.4: Typical peak factors for estimation of peak flows in ℓ/s.

Population Served	Peak Factor (ℓ/s)
2 000	3.5
10 000	3.0
30 000	2.5

The effect of long sewers is to attenuate and thereby decrease the peak factor. Also a sewered area with an elongated shape and flat slopes will likewise tend to lower the peak factor. The peak dry weather flow for a works receiving sewage by pumping may vary considerably from that of a fully gravitational system. These cases should be investigated in more detail.

2.3.3.3 Peak Wet Weather Flow (PWWF)

The average dry weather flow (ADWF) should exclude any provision for groundwater infiltration or stormwater entering the sewerage system. Capacity should, however, be provided for this peak wet weather flow. Special circumstances, such as high rainfall intensity, high natural water table, water-tightness of pipe systems, frequency of drainage inspections, etc., should be taken into account in assessing the peak wet weather flow.

Estimates of peak wet weather flow (PWWF) vary so widely that no fixed criteria can be given. The allowance in an arid region may be effectively zero. Areas close to sea level or flat areas with high water tables may have continuing infiltration of groundwater which may have high salinity. Unstable ground conditions may lead to frequent sewer fractures and high levels of ground and stormwater infiltration. The geology and climate of the area therefore needs to be taken into account when planning for wet weather peaks.

One empirical method for estimating increased flow under wet weather conditions is to size the hydraulic design to allow for a flow rate equal to one and a half times the peak dry weather flow (PDWF).

2.4 SEWAGE STRENGTH AND CHARACTERISTICS

2.4.1 INTRODUCTION

Wastewaters can have a wide range of strengths and varying characteristics depending on their origin. When considering wastewater one normally starts with domestic sewage and this manual is basically for the design of plants for treating sewage of domestic origin. However even a small town may have an industrial component to its wastewater and knowledge of the expected characteristics is essential for a suitable design of treatment works.

Different industries produce effluents which differ in strength and characteristics. These have been extensively surveyed and reports published in the NATSURV series issued by the Water Research Commission. The reports cover most of the types of wet industry likely to be encountered. However if the industry in question is already in existence, there is no substitute for a local investigation into the quantity and nature of effluent being produced. This is normally the work of a specialist and particular care should be taken when designing plants if experienced assistance is not available.

Further discussion on industrial effluents is beyond the scope of this manual and the discussion below is confined to domestic sewage.

2.4.2 CHARACTERISTICS OF INTEREST

Sewage characteristics can be divided into four main categories:

i. the concentration of oxidisable organic material, or substrate,
ii. the concentration of nutrients present and
iii. the solids concentration.
iv. the pH and alkalinity value

The **concentration of oxidisable material** is normally expressed as an oxygen demand and is a measure of the strength of the sewage. The **nutrients** generally refer to nitrogen and phosphorus present, and is a measure of the propensity for the treated effluent to give rise to eutrophication (algal growth) downstream from the works.

The **solids concentration** is an indicator of the relative amount of sludge likely to be produced. The **alkalinity** needs to be adequate to sustain full nitrification (oxidation of ammonia to nitrate).

2.4.2.1 Strength of Sewage

The strength of sewage arriving at treatment works varies considerably, depending largely upon the domestic living standards of the contributing population. The following are the most common parameters for measuring organic strength:

1. Chemical oxygen demand (COD)
2. Five-day biochemical oxygen demand (BOD)

The selection of one of these parameters depends largely on practical considerations and personal preferences. However, guidelines to assist in selection of these parameters are presented below.

BOD is intended to be a measure of the amount of oxygen required by bacteria and organisms to break down the sewage. However the normal test is the 5 day BOD, and it only estimates about half the total biological demand. It is possible to carry out an ultimate (complete) BOD but this takes about 50 days and is a difficult test that is highly prone to inaccuracies. We therefore tend to use the BOD (5 day) as a measure of the biodegradability of the sewage or effluent and the COD as a better estimate of the oxygen demand for purification. Many design criteria are in terms of BOD but these have a correction factor built in. However the trend has been towards using COD for the design criteria in recent years, and this manual generally expresses its design criteria in terms of COD.

BOD is widely used in the design of lagoon and pond systems, and this has been retained in this manual. The design of biological filters used to be expressed in terms of the old Oxygen Absorbed (OA) or Permanganate Value parameters but this, and the activated sludge process are now increasingly being designed in terms of COD.

The COD test is increasingly used in the design of the activated sludge process. The test itself takes about 2 hours and is an oxidation of sewage at high temperature (reflux) under strongly acid and oxidising conditions. With only minor exceptions, the COD effectively oxidises all the organic matter and is a good measure of the total strength of the sewage.

There is a fairly constant COD/BOD ratio of about 2:1 in domestic sewages. As such, treatment processes for domestic sewage can be designed using either COD or BOD as a design parameter. Only where final COD’s and BOD’s are being considered for pond designs does the difference become material.

2.4.2.2 Nutrient Concentration

The two most important nutrients in sewage treatment processes are nitrogen and phosphorus.

Nitrogen concentration is generally determined as Total Kjeldahl Nitrogen (TKN), which measures the sum of the free and saline ammonia plus the organic nitrogen concentrations. Nitrogen is not usually the limiting nutrient for eutrophication but is important because it exerts an oxygen demand on the activated sludge process, and consumes alkalinity when converted from ammonia to nitrate (nitrification).

Sewage phosphorus in most domestic is present in both the poly-phosphate and the ortho-phosphate forms. It is therefore important, when measuring the total phosphorus concentration present, to select a test that converts all the phosphate into one form, usually the orthophosphate. Phosphorus is the limiting nutrient for eutrophication. As little as 0.1 mg/ℓ phosphate in an impoundment can lead to algal growth. The limit for sewage effluents in sensitive areas is 1 mg/ℓ, and normal sewages have about 10 to 12 mg/ℓ of phosphate present. Phosphate removal by biological means can be built into the activated sludge process but this requires specialised design. If phosphorus is limited, most small plants use metal salt addition to remove phosphate (iron or aluminium)

2.4.2.3 Solids Concentration

The removal of suspended solids in the raw sewage by primary sedimentation gives a reasonably good estimate of the sludge solids which will be requiring treatment in the sludge plant at the works. Typical removals are of the order of about 250 mg/ℓ (0.25 kg/m^3). These solids are incorporated into the waste sludge discharged from the process for extended aeration activated sludge plants.

2.4.2.4 Alkalinity

The oxidation of ammonia to nitrate also requires about 7mg/ℓ of alkalinity per mg/ℓ of nitrogen. If the total alkalinity of a wastewater is not more than seven times the TKN, there will be a shortage of alkalinity for nitrification, resulting in low pH effluent with residual ammonia. The two options available to counter this are to promote denitrification, which recovers just over half the alkalinity lost per mg/ℓ, or to add alkali (lime or soda ash).

2.4.2.5 Typical Sewage

A typical domestic sewage or a wastewater with a high domestic component will have typical characteristics, as shown below :

BOD (as O_2)	250 to 350 mg/ℓ
COD (as O_2)	500 to 700 mg/ℓ
Settleable solids	8 to 10 mℓ/ℓ
Suspended solids	200 to 350 mg/ℓ
TKN	60 to 85 mg/ℓ
Ammonia (NH_4^+-N)	40 to 50 mg/ℓ
Phosphate (P)	10 to 13 mg/ℓ

An increase in concentrations should be anticipated if the major contribution to the sewage is from lower income groups. Conversely if the contributing area is a high-income area, the sewage will be of lower strength.

2.5 LOAD ESTIMATION

2.5.1 INTRODUCTION

The strength of sewage arriving at treatment works varies considerably, depending largely upon the domestic living standards of the contributing population. Sub-economic housing areas tend to produce strong sewages due to low per-capita water consumptions and high occupation densities. Luxury housing areas produce weak or dilute sewages due to high specific water consumption patterns, and the use of washing appliances.

Wastewater characteristics can be divided into three main categories:

i. The concentration of oxidisable organic material, or substrate
ii. The concentration of nutrients present
iii. The solids concentration.

2.5.2 ORGANIC CONCENTRATION

The following are the most common parameters for measuring organic strength:

1. Chemical oxygen demand (COD)
2. The five-day biochemical oxygen demand (BOD)

The COD test is increasingly used in the design of the activated sludge process and fixed film aerobic processes. The design criteria in Europe and America sometimes use COD and sometimes BOD but the use of COD seems to be gaining ground. The BOD test has been, and is still used in the design of lagoon and pond systems, and occasionally of biological filters and the activated sludge process. However the test itself has a number of disadvantages, making the result dependent on the reaction rate as well as the substrate concentration.

In domestic sewages the range of COD/BOD ratios varies between about 1.8 and 2.5 and is commonly assumed to be about 2:1. The COD/OA (Oxygen Absorbed) ratio is usually about 10:1. As such, treatment processes for domestic sewage can be designed using either COD, BOD or OA as a design parameter. In practice one would normally measure or estimate the COD concentration or load, and divide by 2 if the design criteria were expressed in terms of BOD, or by 10 if old OA or PV (Permanganate Value) criteria were being used. In this design guideline we will generally be using COD as a basis, although BOD will be retained for pond design.

2.5.3 NUTRIENT CONCENTRATION

The two most important nutrients in sewage treatment processes are nitrogen and phosphorus. Phosphorus is usually the limiting nutrient when wastewater effluents are discharged to watercourse and impoundments and its presence leads to the proliferation of benthic algae in rivers (attached algae), and phyto-plankton (floating algae) in dams and lakes. The phenomenon is known as eutrophication and is a cause of secondary pollution. Certain species of algae can become toxic to livestock. For this reason the phosphorus concentration in effluents is limited in certain catchments, and processes which remove this nutrient are necessary on wastewater plants on such catchments. The designer of a wastewater works should therefore ensure the status of the catchment into which the works effluent will be discharged before finalising the design of the plant.

High nitrogen concentrations are also limited, as there is a General Authorisation limit on ammonia and nitrate concentrations in effluent discharges. High concentrations of nitrate are harmful when consumed by infants, which is a consideration when there is informal abstraction downstream of a plant. Ammonia in effluent is toxic to various aquatic organisms, including many species of fish. Nitrogen concentration is generally determined as Total Kjeldahl Nitrogen (TKN) which measures the sum of the free ammonia and ionised ammonium concentrations, as well as the organic nitrogen concentrations.

2.5.4 SOLIDS CONCENTRATION

The solids component of wastewater, and particularly of domestic sewage, comprises a significant portion of the overall biological load (approximately 40%). On large works it is usually preferable from an operating cost point of view to treat the solids separately in a separate sludge treatment section of the plant. On small works the additional capital cost and operating complications of separate sludge treatment are not justified, and, apart from septic tanks, the solids are usually treated in the aerobic process.

Solids in wastewater are generally measured using two tests. The first and oldest test is the **Settleable Solids** test where the wastewater is poured into a steep-sided conical flask known as an Imhoff Cone (similar to a rain gauge, with similar volumetric graduation marks from the base of the cone). It is then settled for a fixed period (typically one hour) and the volume of settled solids read off. The settleable solids test gives an approximate value for the volume of sludge likely to be produced from a particular wastewater. For a domestic sewage this usually ranges from about 5 to 15 mℓ/ℓ.

The second more reliable test is the **Suspended Solids** (SS) test where the wastewater is filtered and solids removed, dried and weighed. For sludge treatment design the suspended solids removal is relevant, rather than the absolute value in the wastewater. This would be determined by measuring the suspended solids in the raw wastewater and the suspended solids after settling the wastewater for an hour or two hours. SS results are expressed as milligrams per litre (mg/ℓ) which is equivalent to parts per million. A typical suspended solids removal would be between 150 and 350 mg/ℓ.

2.5.5 LOAD FROM TYPICAL DOMESTIC SEWAGE

The daily load per capita can be expressed by various parameters and these will also vary depending on the diet and social structure of the population served. The suggested loads for design purposes are given in **Table 2.5.1** below.

TABLE 2.5.1: Suggested loads per capita for typical domestic sewage.

Parameter	Design Loads g/Cap.d
Oxygen Demand	
BOD (as 0_2)	50 – 80
COD (as 0_2)	100 – 160
Solids	
Settleable solids	60
Suspended solids	30
Nutrients	
Ammonia nitrogen (NH_4 -N)	15
Phosphate (P)	4

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2.5.6 MIXED WASTEWATER AND INDUSTRIAL EFFLUENTS

The existing or planned factories need to be considered for their contribution to the works. Norms exist for nearly all industries relating expected wastewater flows, strengths and characteristics to production levels and types of product. In addition to this samples can be taken from an existing factory and effluent volumes estimated. If the factory is still in the planning stage it may also be possible to take samples from a similar existing factory to obtain an idea of the expected effluent characteristics.

The industrial load and volume would then be added to the domestic component to arrive at a total volume and load. Allowance must also be made for the domestic sewage load arising from the factories particularly if many of the employees come from outside the area.

2.5.7 OTHER CRITERIA

In addition, many criteria are available on the Internet, particularly from the USA. Comprehensive tables from the British Water website and the Washington State (USA) website are shown below as examples.

TABLE 2.5.2: British Water: Table of Loadings for Sewage Disposal Facilities

Expressed per person / activity / day (unless otherwise specified) (Ammonia as N)

	Flow ℓ	BOD g	Ammonia g
DOMESTIC DWELLINGS			
Standard residential	200	60	8
Mobile home type caravans with full services	180	75	8
INDUSTRIAL			
Office / Factory without canteen	50	25	5
Office / Factory with canteen	100	38	5
Open industrial site, e.g. construction, without canteen	60	25	5
*Full-time Day Staff	90	38	5
*Part-time Staff (4 hr shift)	45	25	3
SCHOOLS			
Non-residential with canteen cooking on site	90	38	5
Non-residential without a canteen	50	25	5
Boarding school (i) residents	200	75	10
(ii) day staff (including mid-day meal)	90	38	5
HOTELS, PUBS & CLUBS			
Hotel Guests (Prestige hotels)	300	105	12
Hotel Guests (3 & 4 hotels)	250	94	10
Guests (Bedroom only – no meals)	80	50	6
Residential Training/Conference Guest (all meals)	350	150	15
Non residential Conference Guest	60	25	2.5
Drinkers	12	15	5
HOTELS, PUBS & CLUBS continued			
Holiday camp chalet resident	227	94	10
Resident Staff	180	75	10
Restaurants - Full Meals - luxury catering	30	38	4
- pre-prepared catering	25	30	2.5
- Snack Bars & bar meals	15	19	2.5
- Function Rooms including buffets	15	19	2.5
- Fast Food i.e. (roadside restaurants)	12	12	2.5
- Fast Food Meal (burger chain and similar)	12	15	4
Students (Accommodation only)	100	56	5
AMENITY SITES	10	112	2.5
Toilet Blocks (per use	10	12	2.5
Toilet (WC) (per use)	5	12	2.5
Toilet (Urinal) (per use)	10	19	4
Toilet Blocks in long stay car parks/lorry parks (per use)	40	19	2
Shower (per use)	20	19	5
Local community sports club, e.g. squash, rugby & football	40	25	6

TABLE 2.5.2: Continued	Flow ℓ	BOD g	Ammonia g
AMENITY SITES			
Swimming (separate pool without sports centre)	10	12	2.5
Health Club/Sports Centre	50	19	4
Tent sites			
Caravan Sites - (i) Touring not	100	44	8
(ii) Static not	100	44	8
(iii) Static fully	180	75	8
HOSPITALS & RESIDENTIAL CARE HOMES			
Residential old people / nursing	350		
Small hospitals	450		
Large hospitals	Assess		

**Staff figures also apply to other applications*

TABLE 2.5.3: Design Basis for New Sewage Works (Washington State)

DISCHARGE FACILITY	Flow ℓ/d	BOD g/d	SS g/d	Flow Duration h/d
Dwellings per person	380	90	90	24
Schools with showers and cafeteria per person	60	18	18	8
Schools without showers and with cafeteria per person	40	11	11	8
Boarding schools per person	285	90	90	16
Motel (rooms only) per room	490	118	118	24
Trailer (caravan) courts at 3 persons per caravan per site	1100	272	272	24
Restaurants per seat	190	90	90	16
Highway restaurants per seat	680	318	318	24
Highway rest areas per person	20	5	5	24
Service stations per vehicle serviced	40	5	5	16
Factories per person per 8-hr shift	55-130	14-32	14-32	8
Shopping centres per 100 m^2 of floor space	750-1140	5	5	12
Hospitals per bed	1140	272	272	24
Nursing homes per bed	750	136	136	24
Homes for the aged per bed	380	90	90	24
Doctor's office in medical centre per 100 m^2	1800	45	45	12
Laundromats, 9 to 12 machines per machine	1800	130	130	16
Community colleges per student and faculty	60	14	14	12
Swimming pools per swimmer	35	0.5	0.5	12
Theatres, drive-in type per car	20	5	5	4
Theatres, auditorium type per seat	20	5	5	12
Picnic areas, per person	20	5	5	12
Resort camps, day and night, with limited plumbing per campsite	190	23	23	24
Luxury camps with flush toilets per campsite	380	45	45	24

Care should be taken when applying these criteria as they do not necessarily relate to local conditions. In particular, the USA criteria for dwellings have a high per capita load due to the widespread use of kitchen waste-disposal units. Where no local criteria appear to exist however, they can serve as a guide and can be related to other criteria for which a local equivalent is available.

2.6 PROCESS SELECTION

2.6.1 INTRODUCTION

Process selection is sometimes straightforward, but is often complex and involves a large number of factors. The result depends on the primary objective which may either be the best technical solution at an efficient cost, or the best available solution for the finance available. It also depends on other considerations such as the available operating expertise, the maintenance and engineering capabilities of the client, the availability and cost of power and chemicals, the distance from large cities where technical and backup resources are available, and the climatic factors involved.

As a general principle, process selection should arrive at the simplest practical solution that will achieve the desired objective. Although there are often complex biochemical processes involved, wastewater treatment processes are generally not all that High-Tech in nature. However the ability of clients to operate activated sludge processes (especially nutrient removal plants), and to maintain machinery in efficient working order needs to be established before making a decision. The ability of operating staff to operate complex processes such as occur in a reclamation plant needs also to be considered before making an appropriate decision on re-use of water or effluent.

In general the objective of wastewater treatment is the removal of suspended solids and reduction of COD or BOD, and ammonia. Choice of the process most applicable will be influenced by the proposed plant size, type of waste to be treated, and degree and consistency of treatment required. Some of the considerations involved are listed below.

The selection of primary solids separation automatically involves the need for separate sludge treatment and dewatering processes. This is not justified on small plants for reasons of cost and complexity. As a reasonable generalisation for cut-off, plants of 10Mℓ/d and smaller should not have primary sedimentation and many plants larger than this are also of the extended aeration type.

Notwithstanding the above, very small plants of package plant size for housing estates are often best served by the installation of septic tanks as a first stage for solids removal and sludge accumulation, as well as some biological load reduction.

Effluents of high strength are often well suited to a first stage anaerobic process which reduces the biological load without high power consumption. Effluents from the anaerobic first stage or high rate aerobic process are then treatable by either activated sludge in one of its variants, or by a low rate fixed film process as a second stage to achieve the necessary discharge standards. High rate plastic media filters are a reasonable alternative to primary anaerobic treatment as they are low in energy requirement, but the plastic medium is relatively expensive.

Nutrient removal processes are less successful as a second aerobic stage of treatment than as a single stage aerobic process, due to the removal of readily biodegradable COD in the primary stage.

The selection of process depends on land availability. With unlimited land a pond system has much to recommend it. Smaller areas could use aerated lagoons and maturation ponds, or mechanical plants followed by a pond or a wetland. Highly built-up areas will require mechanical plants, sometimes on two levels and using high-rate compact processes.

The level of operating skills available is important. No plant can be fully automated, but some processes require less attention than others. For very small plants, fixed film processes with septic tanks may often be preferable to variants of the activated sludge process, although sequencing batch reactors (SBR's) reduce some of the operating attention required.

The positioning of the plant is relevant. Plants adjacent to residences should be quiet and odour-free and this will influence process selection.

Where sludge treatment is necessary, the choice of sludge treatment process, thickening and/or dewatering will depend on the land availability and the disposal route available. Land disposal, landfill disposal, composting, and incineration all have different requirements.

The cost of power is important. High power costs move the choice towards anaerobic and fixed film processes. Low power costs favour aerobic digestion and activated sludge.

Climatic factors also need to be considered. Cold winters would necessitate long sludge age activated sludge plants for continued nitrification, and well-insulated mesophillic digestion of sludge. Warm climates would allow lower sludge ages and cold digestion.

Mechanical complexity is an issue. It would not be advisable to install complex machinery in a plant for a small municipality in a remote rural area. Conversely a plant for a mining village where there is mine maintenance staff available would tolerate a mechanised approach using machinery rather than a large labour force.

Finally, but importantly, the process selection should be suitable for achieving the discharge or effluent standards that are required for the particular application (see Chapter 2.8).

2.7 TREATMENT SEQUENCES

2.7.1 INTRODUCTION

In designing a wastewater plant some process choices limit the number of alternatives possible for subsequent treatment, or predicate the need for additional processes such as sludge treatment or dewatering. As in many other fields, the guiding principle should be to keep it simple. This is particularly the case for very small plants where skilled operation may not be available. On much larger plants it may be economically beneficial to have primary sedimentation, anaerobic digestion and mechanical dewatering or sludge drying beds. However on small plants it may be best to have a septic tank and a fixed film aerobic process.

The tables below are not fully comprehensive but are intended to give a guide as to options available to a designer at various stages of the selection of processes.

2.7.2 VERY SMALL PLANTS (SINGLE DWELLING UP TO APPROXIMATELY 500 Kℓ/D)

2.7.2.1 Primary Selection

No Head of Works (No Screens, degritters or flowmeter).

2.7.2.2 Process Choices Available

1. Pond system
2. Septic tank followed by soakaway
3. Septic tank followed by RBC or Submerged Media Reactor
4. Septic tank followed by activated sludge (Usually slightly larger plants in this range).

2.7.2.3 Disinfection Choices Available

1. None (soakaway, maturation ponds, wetlands, irrigation)
2. Chlorination and discharge to watercourse or irrigation (HTH (calcium hypochlorite) pills, sodium hypochlorite solution)
3. Ultraviolet (UV) Irradiation and discharge to watercourse or irrigation.

2.7.3 SMALL PLANTS (500 Kℓ/D UP TO 2 000 Kℓ/D)

Plants in this size range will have a head of works, with screening and degritting, and usually either a flowmeter or some other form of flow measurement.

2.7.3.1 Primary Selection

No primary sedimentation.

2.7.3.2 Process Choices Available

1. Extended aeration activated sludge
2. Sequencing Batch Reactor (SBR)
3. Maturation ponds or wetlands after aerobic treatment
4. Sludge lagoons
5. Sludge drying beds.

2.7.3.3 Disinfection Choices Available

1. Chlorination by sodium hypochlorite solution or commercial granulated calcium hypochlorite with solution feed
2. UV irradiation.

2.7.3.4 Effluent Disposal Choices

1. Discharge to watercourse or wetland
2. Low grade re-use.

2.7.4 MEDIUM SIZED PLANTS (2 Mℓ/D TO 10 Mℓ/D) – NO PRIMARY SEDIMENTATION

Plants in this size range will invariably have a head of works that will probably be mechanised

2.7.4.1 Primary Selection

No primary sedimentation.

2.7.4.2 Process Choices Available

1. Extended aeration activated sludge
2. Maturation ponds or wetlands after aerobic treatment
3. Sludge lagoons
4. Sludge drying beds
5. Mechanical sludge dewatering (DAF, belt press, centrifuge).

2.7.4.3 Disinfection Choices Available

1. Chlorination by sodium hypochlorite solution or gas chlorination

2. UV irradiation.

2.7.4.4 Effluent Disposal Choices

1. Discharge to watercourse or wetland
2. Low grade re-use
3. Re-charging groundwater.

2.7.5 MEDIUM SIZED PLANTS (2 Mℓ/D TO 10 Mℓ/D) – PRIMARY SEDIMENTATION PROVIDED

Plants of this size will usually have a mechanised Head of Works.

2.7.5.1 Primary Selection

Primary sedimentation provided.

2.7.5.2 Process Choices Available

1. Conventional Activated Sludge or Variants
2. Biological (Trickling) Filters
3. Maturation ponds or wetlands after aerobic treatment.

2.7.5.3 Sludge Processing Choices

1. Anaerobic Sludge Digestion (cold, mesophillic, or thermophillic)
2. Sludge drying beds
3. Mechanical sludge dewatering (DAF, belt press, centrifuge).

2.7.5.4 Disinfection Choices Available

1. Chlorination by sodium hypochlorite solution or gas chlorination
2. UV irradiation.

2.7.5.5 Effluent Disposal Choices

1. Discharge to watercourse or wetland
2. Low grade re-use
3. Re-charging groundwater.

2.7.6 LARGE PLANTS (> 10 Mℓ/D)

These are beyond the scope of this manual but most of the choices for medium-sized works remain valid. There is more likelihood of re-use being made of the effluent from larger works, as this represents significant quantities of water, and the scale of use justifies the more sophisticated treatment processes that are required.

2.8 EFFLUENT DISPOSAL AND DISCHARGE STANDARDS

2.8.1 INTRODUCTION

Discharge of wastewater effluent to a water course or by irrigation is controlled by Government legislation, namely Section 39 of the National Water Act, 1998 (Act No. 36 of 1998). This is available from the Department of Water Affairs website and as it is periodically revised, it is advisable to access the latest version from the website., , The latest version at the time of publishing this manual is provided below for ease of use..

The Act specifies the volumes of wastewater effluent that can be irrigated to land, or discharged to a water resource daily, the volume being limited by the quality of the effluent and its impact on affected water resources, land, and health and safety of the population. Tables listing chemical, physical and bacterial limits are provided, together with a table of listed water resources. The special wastewater limits apply when discharging wastewater effluent to a listed water resource

The Act also specifies monitoring frequency, analytical and record-keeping requirements, precautionary procedures and procedures for registering as a user with the Department of Water Affairs.

2.8.2 SECTION 39 OF THE NATIONAL WATER ACT, 1998

GAZETTE NO 26187

GOVERNMENT NOTICE

DEPARTMENT OF WATER AFFAIRS AND FORESTRY

NO. 399 26 March 2004

REVISION OF GENERAL AUTHORISATIONS IN TERMS OF SECTION 39 OF THE NATIONAL WATER ACT, 1998 (ACT NO. 36 OF 1998)

IRRIGATION WITH WASTEWATER

2.7. A person who-

(a) owns or lawfully occupies property registered in the Deeds Office as at the date of this notice;

(b) lawfully occupies or uses land that is not registered or surveyed; or

(c) lawfully has access to land on which the use of water takes place, may on that property or land:

(i) irrigate up to 2000 cubic metres of domestic and biodegradable industrial waste water on any given day provided the-

Faecal coliforms do not exceed 1000 per 100 mℓ;

Chemical Oxygen Demand (COD) does not exceed 75 mg/ℓ;

pH is not less than 5.5 or more than 9.5 pH units;

Ammonia (ionised and un-ionised) as nitrogen does not exceed 3 mg/ℓ;

Nitrate/nitrite as nitrogen does not exceed 15 mg/ℓ;

Chlorine as free chlorine does not exceed 0,25 mg/ℓ;

Suspended solids does not exceed 25 mg/ℓ;

Electrical conductivity does not exceed 70 milliSiemens above intake to a maximum of 150 milliSiemens per metre (mS/m);

Ortho-phosphate as phosphorus does not exceed 10 mg/ℓ

Fluoride does not exceed 1 mg/ℓ; and

Soap, oil or grease does not exceed 2.5 mg/ℓ.

(ii) irrigate up to 500 cubic metres of domestic or biodegradable industrial wastewater on any given day, provided the-

(a) electrical conductivity does not exceed 200 milliSiemens per metre (mS/m);

(b) pH is not less than 6 or more than 9 pH units;

(c) Chemical Oxygen Demand (COD) does not exceed 400 mg/ℓ after removal of algae;

(d) faecal coliforms do not exceed 100 000 per 100 mℓ; and

(e) Sodium Adsorption Ratio (SAR) does not exceed 5 for biodegradable industrial wastewater.

(ii) irrigate up to 50 cubic metres of biodegradable industrial wastewater on any given day, provided the-

(a) electrical conductivity does not exceed 200 milliSiemens per metre (mS/m);

(b) pH is not less than 6 or more than 9 pH units;

(c) Chemical Oxygen Demand (COD) does not exceed 5 000 mg/ℓ after removal of algae;

(d) faecal coliforms do not exceed 100 000 per 100 mℓ; and

(e) Sodium Adsorption Ratio (SAR) does not exceed 5 for biodegradable industrial wastewater,

if the irrigation of wastewater-

(a) does not impact on a water resource or any other person’s water use, property or land; and

(b) is not detrimental to the health and safety of the public in the vicinity of the activity.

Registration of irrigation with wastewater

2.8.(1) A person who irrigates with wastewater in terms of this authorisation must submit to the responsible authority a registration form or any other information requested in writing by the responsible authority for the registration of the water use before commencement of irrigation.

(2) On written receipt of a registration certificate by the Department, the person will be regarded as a registered water user.

(3) All forms for registration of water use are obtainable from the Regional offices of the Department as well as from the Departmental web-site at http://www.dwaf.gov.za

Location of irrigation with wastewater

2.9. Wastewater irrigation in terms of this authorisation is only permitted if the irrigation takes place-

(a) above the 100 year flood line, or alternatively, more than 100 metres from the edge of a water resource or a borehole which is utilised for drinking water or stock watering, whichever is further; and

(b) on land that is not, or does not, overlie a major aquifer (identification of a major aquifer will be provided by the Department, upon written request).

Record-keeping and disclosure of information

2.10. (1) The water user must ensure the establishment of monitoring programmes to monitor the quantity and quality of the wastewater to be irrigated prior to commencement of irrigation and thereafter, as follows-

(a) the quantity must be metered and the total volume recorded weekly; and

(b) the quality must be monitored monthly as at the last day of each month by grab-sampling at the point at which the wastewater enters the irrigation system for all parameters listed in Subparagraphs 2.7.(i) and (ii).

(2) The methods for the measurement of specific substances and parameters in any wastewater must be carried out-

(a) by a laboratory that has been accredited under the South African National Accreditation System (SANAS) in terms of SABS Code 0259 for that method; or

(b) as approved in writing by the responsible authority.

(3) Upon the written request of the responsible Authority the water user must-

(a) ensure the establishment of any additional monitoring programmes; and

(b) appoint a competent person to assess the water use measurements made in terms of this authorisation and submit the findings to the responsible authority for evaluation.

(4) Subject to Paragraph 2.10. (3) above, the water user must keep a written record of the following wastewater irrigation and related activities, for at least three years-

(a) demarcate the location of the irrigation area on a suitablly scaled map, and indicate the extent of the area under irrigation on a 1: suitably scaled map;

(b) details of the crop(s) and the area under irrigation;

(c) the irrigation management techniques being practiced;

(d) quantity of wastewater irrigated;

(e) quality of wastewater irrigated;

(f) details of the monitoring programme;

(g) details of failure and malfunctions in the irrigation system and details of measures taken, and such information must be made available upon written request to the responsible authority.

(5) Any information on the occurrence of any incident that has or is likely to have a detrimental impact on the water resource quality must be reported to the responsible authority.

Precautionary practices

2.11. (1) The water user must follow acceptable construction, maintenance and operational practices to ensure the consistent, effective and safe performance of the wastewater irrigation system, including the prevention of-

(a) water-logging of the soil and pooling of wastewater on the surface of the soil;

(b) nuisance conditions such as flies or mosquitoes, odour or secondary pollution;

(c) waste, wastewater or contaminated stormwater entering into a water resource;

(d) the contamination of run-off water or stormwater;

(e) the unreasonable chemical or physical deterioration of, or any other damage to, the soil of the irrigation site; the unauthorised use of the wastewater by members of the public; and

(f) preventing of people being exposed to the mist originating from the industrial waste.

(2) All reasonable measures must be taken for storage of the wastewater used for irrigation when irrigation cannot be undertaken.

(3) Suspended solids must be removed from any wastewater, and the resulting sludge disposed of according to the requirements of any relevant law or regulation, including-

(a) “Permissible utilisation and disposal of sewage sludge” Edition 1, 1997. Water Research Commission Report No TT 85/97 as amended from time to time; and

(b) “Guide: Permissible utilisation and disposal of treated sewage effluent”, 1978. Department of National Health and Population Development Report No. 11/2/5/3, as amended from time to time (obtainable from the Department upon written request).

(4) All reasonable measures must be taken to provide for mechanical, electrical, operational, or process failures and malfunctions of the wastewater irrigation system.

(5) All reasonable measures must be taken to collect stormwater runoff containing waste or wastewater emanating from the area under irrigation and to retain it for disposal;

Inspections

2.12. Any property or land in respect of which a water use has been authorised in terms of this notice must be made available for inspection by an authorised person in terms of section 125 of the National Water Act.

Offences

2.13. A person who contravenes any provision of this authorisation is guilty of an offence and is subject to the penalty set out in Section 151(2) of the National Water Act.

DISCHARGING OF DOMESTIC AND INDUSTRIAL WASTEWATER INTO WATER RESOURCES

3.7. (1) A person who-

(a) owns or lawfully occupies property registered in the Deeds Office as at the date of this notice; or

(b) lawfully occupies or uses land that is not registered or surveyed, or

(c) lawfully has access to land on which the use of water takes place, may on that property or land outside of the areas excluded in paragraph 3.4 above,

(i) discharge up to 2 000 cubic metres of wastewater on any given day into a water resource that is **not** a listed water resource set out in Table 3.43, provided the discharge-

> (a) complies with the General wastewater Limit Values set out in Table 3.12;
>
> (b) does not alter the natural ambient water temperature of the receiving water resource by more than 3 degrees Celsius; and
>
> (c) the discharge is not a complex Industrial wastewater.

(ii) discharge up to 2 000 cubic metres of wastewater on any given day into a listed water resource set out to in Table 3.43, provided the discharge -

> (a) complies with the special wastewater limit values set out in Table 3.21;
>
> (b) does not alter the natural ambient water temperature of the receiving water resource by more than 2 degrees Celsius; and
>
> (c) is not a complex Industrial wastewater,

if the discharging of wastewater-

(*a*) does not impact on a water resource or any other person's water use, property or land; and

(*b*) is not detrimental to the health and safety of the public communities in the vicinity of the activity.

(2) A person may not discharge stormwater runoff from any premises, not containing waste, or water containing wastewater emanating from industrial activities and premises, into a water resource.

Registration of discharges into water resources

3.8. (1) A person who discharges wastewater into a water resource in terms of this authorisation must submit a registration form for registration of the water use before commencement of the discharge.

(2) On written receipt of a registration certificate by the Department, the person will be regarded as a registered water user.

(3) All forms for registration of water use are obtainable from the Regional offices of the Department, as well as from the Departmental web-site at http://www.-dwaf. gov.za

TABLE 2.8.1: Wastewater limit values applicable to discharge of wastewater into a water resource

SUBSTANCE/PARAMETER	GENERAL LIMIT	SPECIAL LIMIT
Faecal Coliforms (per 100 mℓ)	1 000	0
Chemical Oxygen Demand (mg/ℓ)	75 (i)*	30(i)*
pH	5.5-9.5	5.5-7.5
Ammonia (ionised and un-ionised) as Nitrogen (mg/ℓ)	36	2
Nitrate/Nitrite as Nitrogen (mg/ℓ)	15	1.5
Chlorine as Free Chlorine (mg/ℓ)	0.25	0
Suspended Solids (mg/ℓ)	25	10
Electrical Conductivity (mS/m)	70 mS/m above intake to a maximum of 150 mS/m	50 mS/m above background receiving water, to a maximum of 100 mS/m
Ortho-Phosphate as phosphorus (mg/ℓ)	10	1 (median) and 2.5 (maximum)
Fluoride (mg/ℓ)	1	1
Soap, oil or grease (mg/ℓ)	2.5	0
Dissolved Arsenic (mg/ℓ)	0.02	0.01
Dissolved Cadmium (mg/ℓ)	0.005	0.001
Dissolved Chromium (VI) (mg/ℓ)	0.05	0.02
Dissolved Copper (mg/ℓ)	0.01	0.002
Dissolved Cyanide (mg/ℓ)	0.02	0.01
Dissolved Iron (mg/ℓ)	0.3	0.3
Dissolved Lead (mg/ℓ)	0.01	0.006
Dissolved Manganese (mg/ℓ)	0.1	0.1
Mercury and its compounds (mg/ℓ)	0.005	0.001
Dissolved Selenium (mg/ℓ)	0.02	0.02
Dissolved Zinc (mg/ℓ)	0.1	0.04
Boron (mg/ℓ)	1	0.5

* (i) After removal of algae

Record-keeping and disclosure of information

3.9. (1) The water registered user must ensure the establishment of monitoring programmes to monitor the quantity and quality of the discharge prior to the commencement of the discharge, as follows-

(a) the quantity of the discharge must be metered and the total recorded weekly; and

(b) the quality of domestic wastewater discharges must be monitored monthly by grab sampling and analysed for specific substances and parameters as required by the responsible authority. as set out in Table 3.32.

(c) the quality of industrial wastewater discharges must be monitored weekly by grab sampling-

(i) for all substances which have been added to the water through any industrial activity;
(ii) for all substances which have been concentrated in the water through any industrial activity;
(iii) for all substances which may be harmful or potentially harmful to human health or to the water resource quality; and
(iv) as set out in paragraph 3.9(1)(b) above, if the wastewater contains any domestic wastewater.

TABLE 2.8.2: Monitoring requirements for domestic wastewater discharges

DISCHARGE VOLUME ON ANY GIVEN DAY	MONITORING REQUIREMENTS
< 10 cubic metres	None
10 to 100 cubic metres	pH Electrical Conductivity (mS/m) Faecal Coliforms (per 100 mℓ)
100 to 1000 cubic metres	pH Electrical Conductivity (mS/m) Faecal Coliforms (per 100 mℓ) Chemical Oxygen Demand (mg/ℓ) Ammonia as Nitrogen (mg/ℓ) Suspended Solids (mg/ℓ)
1 000 to 2 000 cubic metres	pH Electrical Conductivity (mS/m) Faecal Coliforms (per 100 mℓ) Chemical Oxygen Demand (mg/ℓ) Ammonia as Nitrogen (mg/ℓ) Nitrate/Nitrite as Nitrogen (mg/ℓ) Free Chlorine (mg/ℓ) Suspended Solids (mg/ℓ) Ortho-Phosphate as Phosphorous (mg/ℓ)

(d) The methods for the measurement of specific substances and parameters in any wastewater must be carried out-
(i) by a laboratory that has been accredited under the South African National Accreditation System (SANAS) in terms of SABS Code 0259 for that method; or
(ii) as approved in writing by the responsible authority .

(2) Upon the written request of the responsible authority the registered user must-
(a) ensure the establishment of any additional monitoring programmes; and
(b) appoint a competent person to assess the water use measurements made in terms of this authorisation and submit the findings to the responsible authority for evaluation.

(3) Subject to paragraph 3.9. (2) above, the water user must, for at least five years, submit the following information on a monthly basis to the responsible authority keep a written record of the following wastewater discharge and related activities-
(a) the quantity of wastewater discharged;
(b) the quality of wastewater discharged;
(c) details of the monitoring programme/s;
(d) details of failures and malfunctions in the discharge system and details of measures taken, and such information must be made available upon written request to the responsible Authority.

(4) Any information on the occurrence of any incident that has or is likely to have a detrimental impact on the water resource quality must be reported to the Responsible Authority.

Precautionary practices
3.10. (1) The water user must follow acceptable construction, maintenance and operational practices to ensure the consistent, effective and safe performance of the discharge.
(2) All reasonable measures must be taken to provide for mechanical, electrical, operational, or process failures and malfunctions of the discharge system.

Inspections
3.11. Any property or land in respect of which a water use has been authorised in terms of this notice must be made available for inspection by an authorised person in terms of Section 125 of the National Water Act.

Offences
3.12. A person who contravenes any provision of this authorisation is guilty of an offence and is subject to the penalty set out in Section 151(2) of the National Water Act.

TABLE 2.8.3: Listed water resources

	WATER RESOURCE
1	Hout Bay River to tidal water
2	Palmiet River from Kogelberg Dam to its estuary: Eerste River to tidal water
3	Lourens River to tidal water
4	Steenbras River to tidal water
5	Berg and Dwars Rivers to their confluence
6	Little Berg River to Vogelvlei weir
7	Sonderend, Du Toits and Elandskloof Rivers upstream and inclusive of Theewaterskloof Dam
8	Witte River to confluence with Breede River
9	Dwars River to Ceres divisional boundary
10	Olifants River to the Ceres divisional boundary

	WATER RESOURCE
11	Hisloot and Smalblaar (or Molenaars) Rivers to their confluence with Breede River
12	Hex River to its confluence with Breede River
13	Van Stadens River to tidal water
14	Buffalo River from its source to where it enters the King Williams Town limits municipal area
15	Klipplaat River from its source to Waterdown Dam
16	Swart Kei River to its confluence with the Klipplaat River
17	Great Brak River
18	Bongola River to Bongola Dam
19	Kubusi River to the Stutterheim limits municipal boundary
20	Langkloof River from its source to Barkly East limits municipal boundary
21	Kraai River to its confluence with the Langkloof River
22	Little Tsomo River
23	Xuka River to the Elliot limits district boundary
24	Tsitsa and Inxu Rivers to their confluence
25	Mvenyane and Mzimvubu Rivers from sources to their confluence
26	Mzintlava River to its confluence with the Mvalweni River
27	Ingwangwana River to its confluence with Umzimkulu River
28	Umzimkulu and Polela Rivers to their confluence
29	Elands River to the Pietermaritzburg-Bulwer main road
30	Umtamvuma and Weza Rivers to their confluence
31	Umkomaas and Isinga Rivers to their confluence
32	Lurane River to its confluence with the Umkomaas River
33	Sitnundjwana Spruit to its confluence with the Umkomaas River
34	Inudwini River to the Polela district boundary
35	Inkonza River to the bridge on the Donnybrook-Creighton road
36	Umlaas to the bridge on District Road 334 on the farm Maybole
37	Umgeni and Lions River to their confluence
38	Mooi River to the road bridge at Rosetta
39	Little Mooi and Hlatikula Rivers to their confluence
40	Bushmans River to Wagendrift Dam
41	Little Tugela River and Sterkspruit to their confluence
42	M'Lambonjwa and Mhlawazeni Rivers to their confluence
43	Mnweni and Sandhlwana Rivers to their confluence
44	Tugela River to its confluence with the Kombe Spruit
45	Inyamvubu (or Mnyamvubu) River to Craigie Burn Dam
46	Umvoti River to the bridge on the Seven Oaks-Rietvlei road
47	Yarrow River to its confluence with the Karkloof River
48	Incandu and Ncibidwane Rivers to their confluence
49	Ingogo River to its confluence with the Harte River
50	Pivaan River to its confluence with Soetmelkspruit

	WATER RESOURCE		
51	Slang River and the Wakkerstroom to their confluence		
52	Elands and Swartkoppie Spruit to their confluence		
53	All tributaries of the Komati River between Nooitgedacht Dam and its confluence with and including Zevenfontein Spruit		
54	Seekoeispruit to its confluence with Buffelspruit		
55	Crocodile River and Buffelskloofspruit to their confluence		
56	All tributaries of the Steelpoort River down to its confluence with and including the Dwars River		
57	Potspruit to its confluence with the Waterval River		
58	Dorps River (or Spekboom River) to its confluence with the Marambanspruit		
59	Ohrigstad River to the Ohrigstad Dam		
60	Klein-Spekboom River to its confluence with the Spekboom River		
61	Blyde River to the Pilgrim's Rest municipal boundary		
62	Sabie River to the Sabie municipal boundary .		
63	Nels River to the Pilgrim's Rest district boundary		
64	Houtbosloop River to the Lydenburg district boundary		
65	Blinkwaterspruit to Longmere Dam		
66	Assegaai River upstream and inclusive of the Heyshope Dam		
67	Komati River upstream and inclusive of the Nooitgedacht Dam and the Vygeboom Dam		
68	Ngwempisi River upstream and inclusive of Jericho Dam and Morgenstond Dam		
69	Slang River upstream and inclusive of Zaaihoek Dam		
70	All streams flowing into the Olifants River upstream and inclusive of Loskop Dam, Witbank Dam and Middelburg Dam		
71	All streams flowing into Ebenezer Dam on the Great Letaba River		
72	Dokolewa River to its confluence with the Politzi River		
73	Ramadiepa River to the Merensky Dam on the farm Westfalia 223, Letaba		
4			
	LISTED WATER RESOURCES WHERE SPECIAL LIMIT FOR ORTHO-PHOSPHATE AS PHOSPHORUS IS APPLICABLE (Crocodile (west) Marico Water Management Area)		
74	Pienaars River and tributaries as far as Klipvoor Dam		
75	Crocodile River and tributaries as far as Roodekopjies Dam		
76	Elands and Hex River and trrbutaries as far as Vaalkop Dam		
77	Molopo River and Tributaries as far as Madimola Dam		
	RAMSAR LISTED WETLANDS:	**PROVINCE**	**LOCATION**
78	Barberspan	North-West	26°33' S 25°37' E
79	Blesbokspruit	Gauteng	26°17' S 28°30' E
80	De Hoop Vlei	Western Cape	34°27' S 20°20' E
81	De Mond (Heuningnes Estuary)	Western Cape	34°43' S 20°07' E
82	Kosi Bay	Kwazulu-Natal	27°01' S 32°48' E
83	Lake Sibaya	Kwazulu-Natal	27°20' S 32°38' E
84	Langebaan	Western Cape	33°06' S 18°01' E

	WATER RESOURCE		
85	Orange River Mouth	Northern Cape	$28^{o}40'$ S $16^{o}30'$ E
86	St Lucia System	Kwazulu-Natal	$28^{o}00'$ S $32^{o}28'$ E
87	Seekoeivlei Nature Reserve	Free State	$27^{o}34'$ S $29^{o}35'$ E
88	Verlorenvlei	Western Cape	$32^{o}24'$ S $18^{o}26'$ E
89	Verloren Valei	Mpumalanga	$25^{o}14'$ S $30^{o}4'$ E
90	Nylsvlei	Northern	$24^{o}39'$ S $28^{o}42'$ E
91	Wilderness Lakes	Western Cape	$33^{o}59'$ S $22^{o}39'$ E

2.9 SLUDGE UTILISATION AND DISPOSAL

2.9.1 INTRODUCTION

Wastewater sludges can be derived from a number of sources, for example:

3. Raw or primary sludge (from a primary settling tank)
4. Anaerobically digested sludge
5. Oxidation pond sludge
6. Septic tank sludge
7. Waste activated sludge (sludge wasted from an activated sludge plant)
8. Humus tank sludge
9. Composted sludge

The type of sludge needs to be taken into consideration when choosing a management option for the disposal of the sludge.

New, comprehensive guidelines for the disposal of wastewater sludges were produced as part of a Water Research Commission project (Snyman and Herselman, 2006). The previous guidelines (WRC Report TT85/97,1997) dealt predominantly with the use of wastewater sludges as soil conditioners, but failed to clarify the requirements for other options, such as disposal to landfill sites. In 2002 an addendum to the 1997 Guidelines was produced (WRC Report TT 154/02, 2002) which clearly defined the different management options available for sludge disposal. The most recent Guidelines (Snyman and Herselman, 2006) clarify these options still further and provide a comprehensive sludge classification system. The previous Guidelines were based mainly on international findings, but the new Guidelines incorporate the findings of a WRC initiated research programme devised to increase the knowledge base of sludge management suitable to South African conditions.

The new Guidelines adopt the concept of sustainability, meaning that management options are chosen which do not harm the environment, either by using non-renewable resources or by resulting in a build-up of constituents, beyond that which the environment can safely assimilate. The three options available for sustainable disposal of sludge include:

1. Utilising the calorific energy value of the sludge (e.g. heat generation)
2. Using the useful components of the sludge (e.g. as a soil conditioner, compost)
3. Extracting useful constituents from the sludge

The option of using the useful components, such as for agricultural use, is the most viable option for South African conditions, but not all sludges produced in this country are suitable for this application. Factors which prevent agricultural use of wastewater sludge include:

1. Unacceptably high concentrations of heavy metals or organic contaminants

2. Land availability
3. Community resistance to agricultural use of sludge in their area (odours, flies etc.)

The new Guidelines therefore developed four different management options for the disposal of wastewater sludges, each option developed as a separate guideline volume. These Guidelines now provide the regulatory requirements for the disposal of wastewater sludges.

The new Guidelines require that a sludge be classified in terms of three criteria, namely:
1. Microbial
2. Stability
3. Pollution

Each of the three classes, listed above, then consists of three subclasses. In the case of the microbial class, this is based on the concentrations of faecal coliforms and helminth ova present in the sludge (Microbial Classes A, B and C). The sub-classes making up the Stability Class depend on the treatment process that the sludge has undergone prior to disposal, the more stable the sludge, the higher the Stability Sub-Class into which it falls (Stability Class 1, 2 and 3). The Pollutant Class depends on the results of the analysis of the sludge in terms of arsenic, cadmium, chromium, copper, lead, mercury, nickel and zinc (Pollutant Class a, b and c). A good-quality sludge would therefore be classified as an A1a sludge.

The sludge classification determines the management options available for the sludge. For example, for an A1a sludge, there are no restrictions for the agricultural use of this sludge, while a sludge classified as Microbial Class C must attain Stability Class 1 or 2 in order to qualify for agricultural use (e.g. C1a and C2a would be suitable for agricultural use, but not C3a).

The Guidelines also specify monitoring frequency, sampling procedures, analytical tests, record-keeping requirements and the application rates of sludge to land or waste sites.

The sludge disposal options are an important factor in any wastewater plant design and the various options available should be carefully considered, since the cost implications can be significant. It therefore strongly advised that the Guidelines for the Utilisation and Disposal of Wastewater Sludge be consulted when designing a wastewater works.

The Guidelines can be obtained from:
Water Research Commission
Private Bag X03
Gezina 0031
or

The Department of Water Affairs and Forestry
Resource Protection and Waste
Private Bag X313
Pretoria 0001

CHAPTER 3

3. PRELIMINARY TREATMENT

3.1 FLOW MEASUREMENT

3.1.1 INTRODUCTION

Flow measurement, either of the raw sewage entering the works, or the final effluent leaving a plant, is important if operation of the plant is to be optimised or for the loading on the works to be monitored.
There are five reasons to measure plant flows:

1. To assist in process control and operation of the treatment facility.
2. To help minimise the cost of operation and maintenance.
3. To provide a historical record of wastewater flows and process performance on which to base future plant expansions and modifications.
4. To meet the monitoring requirements of regulatory agencies. These requirements are usually contained in the treatment plant discharge permit.
5. To check for excessive levels of infiltration during wet weather periods or sudden drops in flow indicating a sewer break or pump-station failure

3.1.2 FLOW MEASUREMENT

3.1.2.1 General

All plants, regardless of size, should provide facilities for measurement of flow, even if a flow meter is not provided. Metering devices within a sewage works should be located so that recycle flow streams do not inadvertently affect the flow measurement.

3.1.2.2 Flow Meter Selection

Factors to be considered in selecting the method of flow measurement are as follows:

1. Probable flow range
2. Acceptable head loss
3. Required accuracy
4. Fouling tendency from wastewater.

3.1.3 MISCELLANEOUS DESIGN CONSIDERATIONS

3.1.3.1 Measuring Flumes

Venturi flumes are the most commonly used type of flume which basically consists of a smooth-walled constriction in the flow channel with a sensor to measure the height of flow through the flume on the upstream side. This is related to the flow in the channel by an equation containing the flume dimensions and the flow height, and can be obtained from standard textbooks on hydraulics. Parshall flumes differ from Venturi flumes in that the former has a downward slope in the bottom of the channel, downstream of the throat of the flume whereas venture flumes have a level bottom.

The flow from the calibration equation can be read off a gauge plate set up the wall of the channel as an instantaneous reading, or can be integrated from a float sensor in a side chamber, or from an ultrasonic sensor mounted vertically above the flume to read the water level. Previously float chambers were the norm, now it is usual to specify ultrasonic sensors.

FIGURE 3.1.1: Example of a flume

Flumes can be considered when measuring raw sewage or primary effluent, because of their freedom from clogging problems. Requirements to be observed when designing a flume installation are as follows:

1. The crest shall have a smooth, definite edge. If a liner is used, all screws and bolts shall be countersunk.
2. Downstream elevations should be low enough to maintain free-flow discharge conditions and prevent excessive “backing up” in the diverging section, or provisions must be made to correct the measurement for submergence.

3. Proper location of the flume is very important for accuracy. The flume should not be installed too close to turbulent flow, surging or unbalanced flow, or a poorly distributed velocity pattern. It should be located in a straight section of a channel, without bends immediately upstream of the flume. The flume should be readily accessible for both installation and maintenance purposes. The ultrasonic sensor should be rigidly mounted over the channel in the upstream part of the flume, with sufficient signal damping to provide a relatively noise-free reading and should be connected to the integrator and/or flow indication by suitable compatible electronic means.

1.3.1.1 Flow through Flumes

A simplified formula for flow through flumes is as follows

$Q = k.b.h^{1,5}$ (Approximately)

Where Q - flow rate (m^3/s)

b - width of throat at narrowest point (m)

h - height of liquid above invert upstream of throat (m)

k - a constant that varies, but is typically about 1.75

3.1.4 OTHER FLUMES

Other types of flumes are also available for measuring plant flows. Manufacturers' instructions should be followed.

3.1.5 MEASURING WEIRS

Weirs are appropriate for measuring effluent flows where there is limited likelihood of solids deposition and build-up behind the weir. (Weirs included in these guidelines are V-notch, rectangular with end contractions, and Cipolletti.)

, The following criteria should be met for installation of weirs:

1. The upstream face of the bulkhead should be smooth and in a vertical plane, perpendicular to the axis of the channel
2. The entire crest of a horizontal weir should be a level, plane surface that forms a sharp, right-angled edge where it intersects with the upstream face
3. The upstream corners of the notch must be sharp. They should be machined or filed perpendicular to the upstream face, free of burrs or scratches.
4. The distance of the crest from the bottom of the approach channel (weir pool) should be not less than twice the depth of water above the crest
5. The water overflowing the weir should touch only the upstream edges of the crest and sides

6. The measurement of head on the weir should be taken as the difference in elevation between the crest and the water surface, at a point upstream from the weir a distance of four times the maximum head on the crest
7. The cross-sectional area of the approach channel should be at least six times that of the crest for a distance upstream from 15 to 20 times the upstream head on the weir
8. The head on the weir should have at least 75 mm of free fall at the maximum downstream water surface to ensure free fall and aeration of the nape.

3.1.5.1 Other Flumes

Other types of flumes are also available for measuring plant flows. Manufacturers' instructions should be followed.

3.1.5.2 Measuring Weirs

Weirs are appropriate for measuring effluent flows. (Weirs included in these guidelines are V-notch, rectangular with end contractions, and Cipolletti.)

FIGURE 3.1.2: Example of a rectangular weir

The following criteria should be met for installation of weirs,:

1. The upstream face of the bulkhead should be smooth and in a vertical plane, perpendicular to the axis of the channel

2. The entire crest of a horizontal weir should be a level, plane surface which forms a sharp, right-angled edge where it intersects with the upstream face
3. The upstream corners of the notch must be sharp. They should be machined or filed perpendicular to the upstream face, free of burrs or scratches
4. The distance of the crest from the bottom of the approach channel (weir pool) should be not less than twice the depth of water above the crest
5. The water overflowing the weir should touch only the upstream edges of the crest and sides
6. `The measurement of head on the weir should be taken as the difference in elevation between the crest and the water surface, at a point upstream from the weir a distance of four times the maximum head on the crest
7. The cross-sectional area of the approach channel should be at least six times that of the crest for a distance upstream from 15 to 20 times the upstream head on the weir
8. The head on the weir should have at least 75 mm of free fall at the maximum downstream water surface to ensure free fall and aeration of the nape.

3.1.5.2.1 Flow over V-notch weirs

The equation for flow over a V-notch weir is:

$Q\ (m^3/s) = 1{,}4\ h^{2.5}$

where h – Head of water above weir (m)

3.1.5.2.2 Flow over Rectangular weirs

For a weir of width L (m) and a height of flow of h (m) the flow Q (m^3/s) is given by:

$Q = 2.0\ L.h^{1.5}$ for sharp-crested weirs (Steel plates etc)
And
$Q = 1.7\ L.h^{1.5}$ for broad-crested weirs (Concrete etc)

3.1.5.3 Venturi and Orifice Meters

Venturi and orifice meters can be used for measuring liquid or gas flows in pipes. Requirements to be observed for application of Venturi and orifice meters are as follows:

1. The range of flows, hydraulic gradient, and space available for installation must be suitable for a Venturi or orifice meter and are very important in selecting the mode of transmission to the indicator, recorder, or integrator.
2. Venturi or orifice meters should not be used where the range of flows is too great, or where the liquid may not be under a positive head at all times.

3. Orifice meters are not suitable for raw sewage or sludge. Cleanouts or hand holes are desirable for such duties on Venturi meters.
4. Units used to measure air delivered by positive-displacement blowers should be located as far as possible from the blowers, or means should be provided to dampen blower pulsations.
5. The velocity and direction of the flow in the pipe ahead of the meter can have a detrimental effect on accuracy. There should be no bends or other fittings for five pipe diameters upstream of the Venturi or orifice meter, unless compensated for by the manufacturer's calibration.

3.1.5.4 Magnetic Flow Meters

Magnetic flow meters are appropriate for measuring influent, effluent, and process flows, and are frequently specified for sludge flow measurement. They must be installed in a straight run of pipe at least four pipe diameters away from the nearest bend or pipe appurtenance. They should also be installed away from pump vibration and according to manufacturers' instructions. The pipe should flow full at all times. It is beneficial to have a means of checking the calibration of such meters by displacement tests. If possible this should be allowed for in the works design

3.1.5.5 Other Flow Metering Devices

Flow meters, such as propeller meters, orifice meters, Pitot tubes, and other devices should only be used in accordance with the manufacturers' recommendations and design guidelines. If possible the plant design should include a section of open channel flow or tank filling application where electronic flow meters can be verified.

3.2 SCREENING

3.2.1 INTRODUCTION

Domestic and Municipal wastewaters contain large solids, plastics, rags, and grit that can interfere with treatment processes or cause undue mechanical wear and increased maintenance on wastewater treatment equipment. To minimise potential problems, these materials require removal and/or separate handling. Preliminary treatment at the Head of Works removes these constituents from the influent wastewater. Preliminary treatment normally consists of screening, grit removal, and usually flow measurement.

3.2.2 APPLICABILITY

Because various types of screening and grit removal devices are available, it is important that the proper design be selected for each situation. Though similarities exist between different types of equipment for a given process, an improperly selected design may result in an inefficient treatment process.

3.2.3 SCREENS

Screening is the first unit operation used at wastewater treatment plants (WWTPs). Screening removes objects such as rags, paper, plastics, and metals to prevent damage and clogging of downstream equipment, piping, and appurtenances. Some modern wastewater treatment plants use both coarse screens and fine screens.

3.2.3.1 Coarse Screens

Coarse screens remove large solids, rags, and debris from wastewater, and typically have openings of 10 mm or larger. Types of coarse screens include mechanically and manually cleaned bar screens, including trash racks. **Table 3.2.1** describes the various types of coarse screens.

3.2.3.2 Fine Screens

Fine screens are typically used to remove material that may create operation and maintenance problems in downstream processes, particularly in systems that lack primary treatment. Typical opening sizes for fine screens are 1.5 to 6 mm. Very fine screens with openings of 0.2 to 1.5 mm placed after coarse or fine screens can reduce suspended solids to levels near those achieved by primary clarification and are sometimes used instead of primary sedimentation tanks.

FIGURE 3.2.1: Typical coarse screens

TABLE 3.2.1: Description of coarse screens

Trash Rack

Designed to prevent logs, timbers and other large debris from entering treatment processes.

Opening size: 38 to 150 mm

Manually Cleaned Bar Screen

Designed to remove large solids, rags, and debris.

Opening size: 30 to 50 mm

Bars set at 30 to 45 degrees from vertical to facilitate cleaning.

Primarily used in older or smaller treatment facilities, or in bypass channels.

Mechanically Cleaned Bar Screen

Designed to remove medium to large solids, rags, and debris.

Opening size: 6 to 38 mm

Bars set at 0 to 30 degrees from vertical.

Almost always used in new installations because of large number of advantages relative to other screens.

3.2.4 EQUIPMENT SELECTION

Most large facilities use mechanically-cleaned screening systems to remove larger materials because they reduce labour costs and they improve flow conditions and screening capture. It may be possible to run a works on a one- or two-shift basis with mechanical screening whereas three shifts may be required if hand-raked screens are provided. The savings in labour costs soon make up for the additional cost of a mechanical screen.

Typically, only older or smaller treatment facilities use a manually cleaned screen as the primary or only screening device. A screening compactor is usually situated close to the mechanically cleaned screen and compacted screenings are usually bagged and conveyed to a dumpster or disposal area.

Plants utilising mechanically cleaned screens should have a standby hand-raked screen to put in operation when the primary screening device is out of service. This is standard design practice for most newly designed plants.

The use of fine screens in preliminary treatment has experienced a resurgence in the last 20 years. Such screens were a common feature before 1930 but their use diminished because of difficulty in cleaning oils and grease from the screens. In the early 1980s, fine screens regained popularity because of improved materials.

3.2.5 ADVANTAGES AND DISADVANTAGES

3.2.5.1 Advantages

Manually cleaned screens require little or no equipment maintenance and provide a good alternative for smaller plants with few screenings. Mechanically cleaned screens tend to have lower labour costs than manually cleaned screens and offer the advantages of improved flow conditions and screening capture over manually cleaned screens.

3.2.5.2 Disadvantages

Manually cleaned screens require frequent raking to avoid clogging and high backwater levels that cause build-up of a solids mat on the screen. The increased raking frequency increases labour costs. Removal of this mat during cleaning may also cause flow surges that can reduce the solids-capture efficiency of downstream units. Mechanically cleaned screens are not subject to this problem, but they can have high equipment maintenance costs.

FIGURE 3.2.2: Manual cleaning of screens

FIGURE 3.2.3: Mechanically raked screens (eWISA)

3.2.6 CLASSIFICATION

Screening devices are classified based on the size of the material they remove (the screenings). The "size" of screening material refers to its diameter. **Table 3.2.2** lists the correlation between screening sizes and screening device classification.

TABLE 3.2.2: Screening device classification

Screening Device Classification	Size Classification	Size Range of Screen Opening
Bar screen		
Manually cleaned	Coarse	25 –50 mm
Mechanically cleaned	Coarse	15 – 75 mm
Fine bar or perforated coarse screen (mechanically cleaned)		
Fine bar	Fine to coarse	3 – 12.5 mm
Perforated plate	Fine to coarse	3 –9.5 mm
Rotary drum	Fine to coarse	3 –12.5 mm
Fine Screen (mechanically cleaned)		
Fixed parabolic	Fine	0.25 – 3.2 mm
Rotary drum	Fine	0.25 – 3.2 mm
Rotary disc	Very fine	0.15 – 0.38 mm

In addition to screening size, other design considerations include the depth, width, and approach velocity of the channel; the discharge height, the screen angle; wind and aesthetic considerations; redundancy; and head loss. **Table 3.2.3** lists typical design criteria for mechanically cleaned bar rack type screens.

TABLE 3.2.3: Design criteria for mechanically cleaned bar screens

Design Criteria	Metric Units
Bar Width	5 – 15 mm
Bar Depth	25 – 40 mm
Clear spacing between bars	15 – 75 mm
Slope from vertical	0 – 30°
Approach velocity	0.6 – 1.0 m/s
Allowable headloss	150 mm

The use of fine screens produces removal characteristics similar to primary sludge removal in primary sedimentation. Fine screens are capable of removing 20 to 35 percent suspended solids and BOD_5. Fine screens may be either fixed or movable, but are permanently set in a vertical, inclined, or horizontal position and must be cleaned by rakes, teeth, or brushes.

3.2.7 PERFORMANCE

The use of screening and grit removal systems is well documented. The performance of bar screens varies depending on the spacing of the bars. **Table 3.2.4** lists typical screening quantities for various screen sizes and **Table 3.2.5** lists grit and screenings removal quantities for various plants in the USA.

The quantity of screenings depends on the length and slope of the collection system and the presence of pumping stations. When the collection system is long and steep or when pumping stations exist, fewer screenings are produced because of disintegration of solids. Other factors that affect screening quantities are related to flow, as quantities generally increase greatly during storm flows. Peak daily removals may vary by a 20:1 ratio on an hourly basis from average flow conditions. Combined collection systems may produce several times the coarse screenings produced by separate collection systems.

TABLE 3.2.4: Screening removal quantities

Screen Size (mm)	Screenings Quantity ($m^3/10^6\ m^3$)
13	60.8
38	11.2

TABLE 3.2.5: Grit and screenings removals at various plants in the USA

Plant Location	Flow m^3/d	Grit $m^3/10^3\ m^3$	Screenings $m^3/10^3 m^3$
Uniontown, Pennsylvania	11 400	0.074	0.006
East Hartford, Connecticut	15 100	0.017	0.009
Duluth, Minnesota	45 400	0.006	0.004
Lamberts Point, Norfolk, Va	75 700	0.034	0.009
Village Creek, Ft Worth	170 000	0.009	0.005
Milwaukee, Wisconsin	454 000	0.003	0.004
Twin Cities, Minnesota	825 000	0.034	0.008
Chicago, Illinois (Northside)	1 260 000	0.003	0.006

Local data for screening and grit removal have been obtained for a large South African Municipality. Based on annual average figures, the amount of screenings and grit removed averaged about 20 kg/Mℓ with a range of variation from 6 kg/Mℓ to about 60 kg/Mℓ over ten wastewater works with a combined treatment capacity of about 500 Mℓ/d.

Screenings and grit removed per megalitre tended to be highest for wastewater from poorer residential areas, intermediate for wealthier areas and lower for areas with mixed or predominantly industrial catchments.

The ratio between grit and screening removed ranged from roughly equal (1:1) to about 4.0 to 1 for detritus to screenings. The weighted average is about 2 to 1 for grit to screenings.

3.2.8 OPERATION AND MAINTENANCE

Manually cleaned screens require frequent raking to prevent clogging. Cleaning frequency depends on the characteristics of the wastewater entering a plant. Some plants have incorporated screening devices, such as basket-type trash racks, that are manually hoisted and cleaned.

Mechanically cleaned screens usually require less labour or operation than manually cleaned screens because screenings are raked with a mechanical device rather than by facility personnel. However, the rake teeth on mechanically cleaned screens must be routinely inspected because of their susceptibility to breakage and bending. Drive mechanisms must also be frequently inspected to prevent fouling due to grit and rags. Grit removed from screens must be disposed of regularly.

3.3 DEGRITTING (DETRITUS REMOVAL)

3.3.1 INTRODUCTION

Grit (detritus) includes sand, gravel, cinder, or other heavy solid materials that are “heavier” (higher specific gravity) than the organic biodegradable solids in the wastewater. The grit normally removed on a works also includes eggshells, bone chips, seeds, coffee grounds, and large organic particles, such as food waste. Removal of grit prevents unnecessary abrasion and wear of mechanical equipment, grit deposition in pipelines and channels, and accumulation of grit in anaerobic digesters and aeration basins.

Grit removal facilities typically precede primary clarification, and follow screening and comminution. This prevents large solids from interfering with grit handling equipment. In secondary treatment plants without primary clarification, grit removal should precede aeration (Metcalf & Eddy, 2003).

Many types of grit removal systems exist, including aerated grit chambers, vortex-type (paddle or jet-induced vortex) grit removal systems, detritus tanks (short-term sedimentation basins), horizontal flow grit chambers (velocity-controlled channels), and hydrocyclones (cyclonic inertial separation). Various factors must be taken into consideration when selecting a grit removal process, including the quantity and characteristics of grit, potential adverse effects on downstream processes, head loss requirements, space requirements, removal efficiency, organic content, and cost. The type of grit removal system chosen for a specific facility should be the one that best balances these different considerations. Specifics on the different types of grit removal systems are provided below.

3.3.2 TYPES OF GRIT REMOVAL EQUIPMENT

3.3.2.1 Aerated Grit Chamber

In aerated grit chambers, grit is removed by causing the wastewater to flow in a spiral pattern. Air is introduced in the grit chamber along one side, causing a perpendicular spiral velocity pattern to flow through the tank. Heavier particles are accelerated and diverge from the streamlines, dropping to the bottom of the tank, while lighter organic particles are suspended and eventually carried out of the tank.

3.3.2.2 Vortex-Type Grit Chamber

FIGURE 3.3.1: Typical vortex-type grit chamber

The vortex-type grit chamber consists of a cylindrical or conical tank in which the flow enters tangentially, creating a vortex flow pattern. Grit settles by gravity into the bottom of the tank (in a grit hopper) while effluent exits at the top of the tank. The grit that settles into the grit hopper may be removed by a grit pump or an air lift pump.

3.3.2.3 Detritus Tank

A detritus tank (or square tank degritter) is a constant-level, short-detention settling tank. These tanks require a grit-washing step to remove organic material. One design option includes a grit auger and a rake that removes and classifies grit from the grit sump.

3.3.2.4 Horizontal Flow Grit Chamber or Channel

The horizontal flow grit chamber is the oldest type of grit removal system. Grit is removed by maintaining a constant stream velocity of 0.3 m/s. The velocity is controlled by proportional weirs or rectangular control sections, such as Parshall flumes. In this system, heavier grit particles settle to the bottom of the channel, while lighter organic particles remain suspended or are resuspended and transported out of the channel. The detritus or grit is often removed manually but can be removed by a conveyor with scrapers, buckets, or ploughs. Screw-conveyors or bucket elevators are used to elevate the grit for washing or disposal. In smaller plants, grit chambers are usually cleaned manually.

3.3.2.5 Hydrocyclone

Hydrocyclone systems are typically used to separate grit from organics in grit slurries or to remove grit from primary sludge. Hydrocyclones are sometimes used to remove grit and suspended solids directly from wastewater flow by pumping at a head ranging from 3.7 to 9 m. Heavier grit and suspended solids collect on the sides and bottom of the cyclone due to induced centrifugal forces, while scum and lighter solids are removed from the centre through the top of the cyclone.

3.3.2.6 Selection

When selecting a grit removal process, the quantity and characteristics of grit and its potential to adversely affect downstream processes are important considerations. Other parameters to consider may include head-loss requirements, space requirements, removal efficiency, organic content, and economics.

3.3.3 ADVANTAGES

3.3.3.1 Aerated Grit Chamber

Some advantages of aerated grit chambers include:

1. Consistent removal efficiency over a wide flow range
2. A relatively low putrescible organic content may be removed with a well controlled rate of aeration
3. Performance of downstream units may be improved by using pre-aeration to reduce septic conditions in incoming wastewater
4. Aerated grit chambers are versatile, allowing for chemical addition, mixing, re-aeration and flocculation.

3.3.3.2 Vortex-Type Grit Chamber

These systems remove a high percentage of fine grit, up to 73 percent of 140-mesh (0.11 mm diameter) size. The advantages of such a system are as follows:

1. Vortex grit removal systems have a consistent removal efficiency over a wide flow range.
2. There are no submerged bearings or parts that require maintenance.
3. The “footprint” (horizontal dimension) of a vortex grit removal system is small relative to other grit removal systems, making it advantageous when space is an issue.
4. Head-loss through a vortex system is minimal, typically 6 mm. These systems are also energy efficient.

3.3.3.3 Detritus Tank

Detritus tanks do not require flow control because all bearings and moving mechanical parts are above the water line. There is minimal head-loss in this type of unit.

3.3.3.4 Horizontal Flow Grit Channels and Chambers

Horizontal flow grit chambers are flexible because they allow performance to be altered by adjusting the outlet flow control device. Construction is not complicated. Grit that does not require further classification may be removed with effective flow control.

3.3.3.5 Hydrocyclone

Hydrocyclones can remove both grit and suspended solids from wastewater. An hydrocyclone can potentially remove as much solids as a primary clarifier.

3.3.4 DISADVANTAGES

Grit removal systems increase the head-loss through a wastewater treatment plant, which could be problematic if head-loss is an issue. This could require additional pumping to compensate for the head-loss. The following paragraphs describe the specific disadvantages of different types of grit removal systems.

3.3.4.1 Aerated Grit Chamber

Potentially harmful volatile organics and odours may be released from the aerated grit chamber. Aerated grit chambers also require more power than other grit removal processes, and maintenance and control of the aeration system requires additional labour.

3.3.4.2 Vortex-Type Grit Chamber

1. Vortex grit removal systems are usually of a proprietary design, which makes modifications difficult
2. Paddles tend to collect rags
3. Vortex units usually require deep excavation due to their depth, increasing construction costs, especially if unrippable rock is present
4. The grit sump tends to clog and occasionally requires high-pressure agitation using water or air to loosen grit compacted in the sump.

3.3.4.3 Detritus Tank

1. Detritus tanks have difficulty achieving uniform flow distribution over a wide range of flows because the inlet baffles cannot be adjusted
2. This type of removal system removes large quantities of organic material, especially at low flows, and thus requires grit washing and classifying
3. Grit may be lost in shallow installations (less than 0.9 m) due to the agitation created by the rake arm associated with this system.

3.3.4.4 Horizontal Flow Grit Chamber

1. It is difficult to maintain a 0.3 m/s velocity over a wide range of flows
2. The submerged chain, flight equipment, and bearings for mechanical removal systems undergo excessive wear

3. Channels without effective flow control will remove excessive amounts of organic material that require grit washing and classifying
4. Head loss is high (typically 30 to 40% of flow depth)
5. High velocities may be generated at the channel bottom with the use of proportional weirs, leading to bottom scour.

3.3.4.5 Hydrocyclone

Hydrocyclones require energy because they use a pump to remove grit and suspended solids. Coarse screening is required ahead of these units to remove sticks, rags, and plastics.

3.3.5 DESIGN CRITERIA

With respect to grit removal systems, grit is traditionally defined as particles larger than 0.21 mm (65 mesh) and with a specific gravity of greater than 2.65 (U.S. EPA, 1987). Equipment design was traditionally based on removal of 95%of these particles. However, with the recent recognition that smaller particles must be removed to avoid damaging downstream processes, many modern grit removal designs are capable of removing up to 75% of 0.15 mm (100 mesh) material.

3.3.5.1 Aerated Grit Chamber

Aerated grit chambers are typically designed to remove particles of 70 mesh (0.21 mm) or larger, with a detention period of two to five minutes at peak hourly flow. When wastewater flows into the grit chamber, particles settle to the bottom according to their size, specific gravity, and the velocity of roll in the tank. A velocity that is too high will result in lower grit removal efficiencies, while a velocity that is too low will result in increased removal of organic materials. Proper adjustment of air velocity will result in nearly 100% removal of the desired particle size and a well-washed grit.

Design considerations for aerated grit chambers include the following (WEF 1998):

1. Air rates typically range from 0.3 to 0.7m^3/min.m of tank length
2. A typical minimum hydraulic detention time at maximum instantaneous flow is two minutes
3. Typical length-to-width ratio is 2.5:1 to 5:1
4. Tank inlet and outlet are positioned so the flow is perpendicular to the spiral roll pattern
5. Baffles are used to dissipate energy and minimise short-circuiting.

3.3.5.2 Vortex-Type Grit Chamber

Two designs of vortex grit units exist: chambers with flat bottoms and a small opening to collect grit; and chambers with a sloping bottom and a large opening into the grit hopper. Flow into a vortex-type grit system should be straight, smooth, and streamlined. The straight inlet channel length is typically seven times the width of the inlet channel, or 4.6 m, whichever is greater. The ideal velocity range in the influent is typically 0.6 to 0.9 m/s at 40 to 80% of peak flow. A minimum velocity of 0.15 m/s should be maintained at all times, because lower velocities will not carry grit into the grit chamber (WEF, 1998).

3.3.5.3 Detritus Tank

Detritus tanks are designed to keep horizontal velocity and turbulence at a minimum while maintaining a detention time of less than one minute. Proper operation of a detritus tank depends on well-distributed flow into the settling basin. Allowances are made for inlet and outlet turbulence as well as short circuiting by applying a safety factor of 2.0 to the calculated overflow rate.

3.3.5.4 Horizontal Flow Grit Chamber (Channels)

Horizontal flow grit chambers use proportional weirs or rectangular control sections to vary the depth of flow and keep the velocity of the flow stream at a constant 0.3 m/s. The length of the grit chamber is governed by the settling velocity of the target grit particles and the flow control section-depth relationship. An allowance for inlet and outlet turbulence is added. The cross sectional area of the channel is determined by the rate of flow and the number of channels. Allowances are made for grit storage and grit removal equipment. **Table 3.3.1** lists design criteria for horizontal flow grit chambers.

TABLE 3.3.1: Horizontal flow grit chamber design criteria

Design Criteria	Range	Typical
Detention time	40 – 90 s	60 s
Horizontal velocity	0.24 – 0.4 m/s	0.3 m/s
Settling velocity		
50-mesh	2.8 – 3.1 m/min	2.9 m/min
100-mesh	0.6 –0.9 m/min	0.8 m/min
Headloss (% of channel depth)	30 – 40%	36%
Inlet and outlet length allowance	25 – 50%	30%

If the specific gravity of the grit is significantly less than 2.65, lower velocities should be used.

3.3.6 PERFORMANCE

Given the complexity of collection systems and types of materials that may be considered “grit,” the quantity and characteristics of grit removed from wastewater will vary. Grit quantity is influenced by the type and condition of the collection system, the characteristics of the drainage area, waste disposal methods, the slope of the collection system, and the efficiency of the grit removal system. The quantity of grit may vary from 0.004 to 0.21 $m^3/10^3\ m^3$ (Crites and Tchobanoglous, 1998). The performance of a grit removal system may be enhanced if actual plant data is used when designing a new grit removal system.

Section 3.2 on Screening should be consulted for additional information on quantities of grit requiring removal.

3.3.7 OPERATION AND MAINTENANCE

Collected grit must be removed from the chamber, dewatered, washed, and conveyed to a disposal site. Some smaller plants use manual methods to remove grit, but grit removal is usually accomplished by an automatic method. The four methods of automatic grit removal include inclined screw or tubular conveyors, chain and bucket elevators, clamshell buckets, and air-lift or other means of pumping. A two-step grit removal method is sometimes used, where grit is conveyed horizontally in a trough or channel to a hopper, where it is then elevated from the hopper to another location.

Aerated grit chambers use a sloped tank bottom in which the air roll pattern sweeps grit along the bottom to the low side of the chamber. A horizontal screw conveyor is typically used to convey settled grit to a hopper at the head of the tank. Another method to remove grit from the chamber floor is a chain and flight mechanism. Once removed from the chamber, grit is usually washed with a hydrocyclone or grit classifier to ease handling and remove organic material. The grit is then conveyed directly to a truck, dumpster, or storage hopper. From there, the grit is taken to a landfill or other disposal facility.

CHAPTER 4

4. ANAEROBIC TREATMENT

4.1 SEPTIC TANKS

4.1.1 INTRODUCTION

The process taking place in a septic tank is an anaerobic one and treatment is only partial. As the septic tank provides for partial treatment of sewage only, it does not produce an effluent complying with the General Authorisation limits for discharge to land or watercourse. Limited irrigation use of effluent is permissible under certain circumstances, but generally septic tanks are used before soakaway systems for small applications or as a preliminary or first stage before secondary aerobic treatment. In the latter case the septic tank serves the purpose of largely removing the settleable solids in the sewage and effects a partial reduction in the COD or BOD load entering the aerobic stage. No ammonia reduction takes place through a septic tank and it is not infrequent for the ammonia to increase somewhat due to breakdown and conversion of proteinaceous matter to free and ionised ammonia. The most important parameters for design of septic tanks are the retention available for the settlement of solid matter and the capacity provided for storage and partial degradation of the sludge.

In its most basic form a septic tank can be a single-compartment watertight tank with an inlet pipe discharging below the surface and outlet pipe at the top water level baffled to prevent discharge of scum. Normally however, even for single housing units, a two or even three compartment tank is provided. In a two or three compartment tank, the compartments are separated by vertical walls with openings above mid-water level so as to retain primary sludge or scum in the first compartment as far as possible.

The sizing of septic tanks is usually based on the frequency of de-sludging and the daily per-capita contribution. For a small combined system the size of the tank will normally give a retention of 48 hours. A sizing formula is used which is given below for larger developments.. However a minimum retention period of 24 hours at ADWF should always be allowed for in the septic tank excluding the accumulated sludge and scum.

4.1.2 COMBINED AND SEPARATE DRAINAGE SYSTEMS

The two types of house drainage systems in general use are namely:

1. The separate or two-pipe system, whereby the kitchen and bathroom wastes bypass the septic tank and are diverted to a separate soil percolating system

2. The combined system, whereby all the liquid household wastes are discharged to the septic tank and then to a percolation system.

In either case the kitchen waste should pass through a grease trap before entering the drain. This grease trap requires regular cleaning and maintenance that should not be neglected.

The separate system has the advantage that it offers a safety factor should failure of the kitchen/bath waste percolation system occur. The volume of faecally polluted effluent is considerably less than the combined volumes of waste water.

The combined system has the following advantages over the separate system:

1. Kitchen waste is pre-treated in the septic tank before discharge to the percolation system. The deleterious effect of raw kitchen waste on the absorptive capacity of the soil is thereby prevented.
2. If the grease trap should be neglected, the combined system will deal with the fats more efficiently than the percolation system of the separate system.

The combined system is invariably used on large installations. The recommendation for the fitting and maintenance of grease traps stands, however, particularly for hotels, kitchens and restaurants. A two or three compartment tank is preferable where the combined system is used.

4.1.3 CAPACITY OF SEPTIC TANK

The capacity should be sufficient to provide for storage of sludge and scum between desludging.

4.1.3.1 Single households and small communities up to 30 persons:

For combined systems a retention of 48 hours based on the estimated domestic waste water volume, or 0.3 m^3/person, whichever gives the larger tank size. The minimum recommended size for one household is 2 m^3.

For separate systems a retention of 0.15 m^3/person is the minimum recommended.

4.1.3.2 Larger communities

It is recommended that multiple tank systems be used. A minimum retention period of 24 hours at ADWF should be allowed for in the septic tank excluding the sludge and scum accumulated.

The septic tank volume is calculated as follows:

$V = P(Q + 0.1 \times \sqrt{S})$ ……………………………………………………………(4.1)

Where

V = volume in m^3 excluding freeboard,

Q = flow per capita per day in m^3 (usually 130 or 150 ℓ/hd.d),

S = years between desludging with 0.1 m^3 being sludge accumulated in one year per person and a maximum of 10 years being provided and

P = contributing population.

The formula given above is based on recommendations given in the CSIR Publication K86-1985 "A Guide to the Use of Septic Tank Systems in South Africa".

Sludge and scum accumulation is a function of the desludging interval and could vary between 0.09 to 0.15 m^3/hd.yr. (Refer the 0.1 m^3 in the above formula). However, the volume of accumulated sludge and scum reduces with time due to progressive digestion and compaction. It is thus roughly proportional to the square root of the years of operation as indicated in the above formula.

Desludging of septic tanks should be carried out at the frequency given in **Table 4.1.1** below:

TABLE 4.1.1: Desludging frequency for septic tanks.

Population Served	Desludging Interval
Single Household	5 to 8 years
10 to 30 persons	2 years
50 to 200 persons	1 year
200 to 500 persons	6 months

However it may be appropriate in remote areas to install somewhat larger tanks requiring less frequent desludging.

4.1.4 CONSTRUCTION OF SEPTIC TANKS

Septic tanks for single house units and small communities should have a depth of 1 to 2 m, but otherwise the shape of the tank is relatively unimportant. Usually a length to breadth ratio of 3:1 to 4:1 is recommended. The minimum dimension in any direction should be 0.7 m.

For larger installations the septic tank comprising not less than two compartments should have a length to breadth ratio of 3:1 to 4:1 with a minimum water depth of 2 m. The second compartment should be designed as a sedimentation tank with upflow velocities not exceeding 1.5 m/h at PDWF. In two-compartment tanks the first compartment should be about two-thirds of the total capacity. In a three compartment tank, the compartments should be roughly half, quarter, and quarter the total respectively.

The vertical leg of the inlet pipe, if a square junction pipe is used, should discharge 150 to 300 mm below the surface to prevent blockage by the scum layer. Exit openings or pipes from the first chamber to the second, and from the second to subsequent drains, should also be 150 to 300 mm below the surface, with facilities for cleaning.

A septic tank requires periodic desludging and provision must therefore be made for this by incorporating an adequately sized manhole to each compartment. Good access to all compartments should be provided for scum and sludge removal. The septic tank should be situated where it is accessible to vacuum tankers for sludge and scum removal.

When properly designed, the COD reduction through a septic tank improves with time. Accumulated sludge should thereafter never be completely removed during de-sludging. At least 100 mm should always be left in the tank as an inocculum. The first and final compartments are usually the only ones which need regular desludging

For large tanks, particularly for communities in excess of 200 persons, two or three additional, but smaller compartments than the first, may be included and the following appurtenances provided:

1. Multiple inlets in the first compartment.
2. Removable covers or manholes to permit easy access for cleaning or for mixing the tank contents.
3. Land or drying beds should be available for drying and disposal of the sludge.
4. Depending on topography hopper bottoms fitted with draw-off pipes valves, etc. should be provided for regular sludge removal.

FIGURE 4.1.1: Septic tank before installation

FIGURE 4.1.2: Partially installed septic tank

4.1.5 DISPOSAL OF SEPTIC TANK EFFLUENT

Septic tank effluent may not be discharged to a water course or irrigated on land where it is accessible to humans or animals. Further treatment or other means of disposal are required.

The effluent may be further treated in stabilisation ponds, on biological filters, by the activated sludge process, by a rotating biological contactor (RBC), or it can be disposed of by percolation into the soil. For relatively small or spacious developments the latter is by far the most common.

4.1.6 LOCATION OF SEPTIC TANKS

Septic tanks should be located in firm soil, 2 to 5 m from buildings and boundaries, in accordance with by-laws, but so as to facilitate:

1. Sludge removal by tanker
2. Possible connection of house drainage to a reticulation system at a later date.

4.1.7 LOCATION OF SOIL PERCOLATION SYSTEMS

Percolation systems for septic tank effluent should generally be located such that pollution of surface or ground water is unlikely to occur. Spread of bacteriological pollution above the water table is normally very limited (less than 3 m); but should the percolation system extend into the water table, widespread contamination of ground water supplies can occur. Investigation into the geology of the site is therefore strongly advised. Since percolation systems are subject to progressive failure, they should preferably be located in such a way that they may be extended without disruption of the operating system.

4.1.8 SUITABILITY OF THE SOIL

There are no simple tests by means of which the suitability of a soil to absorb septic tank effluent can be determined accurately. Indications can be obtained by visual inspection of the soil, i.e. whether it has a sandy or clayey nature, or a percolation test may be carried out. The latter entails firstly the excavation of a test hole 300 mm diameter at the depth of the proposed trenches. The ground should then be soaked by adding water to the hole for at least four hours, after which the hole should be allowed to drain and then be refilled with 150 mm of water. The time taken for the water to seep away should then be noted, from which the average time for the water to drop 10 mm can be calculated. From this time, known as the percolation time, an indication of the sidewall area to be provided in a percolation trench can be obtained from the **Table 4.1.2** below:

TABLE 4.1.2: Septic tank sidewall area required for different percolation times.

Percolation Time minutes/10 mm	**Rate of septic tank effluent application to sidewall area of percolation trench ℓ/m^2.d**
1	170
2	110
4	75
6	50
10	40
18	30
24	25
More than 24	Soil unsuitable for percolation

Note: Trench bottom area should be neglected since it gets clogged very rapidly.

4.1.9 PERCOLATION TRENCH CONSTRUCTION

Trenches are normally 1 to 2 m deep and 600 mm wide. After excavation the trench is filled with broken stone (12 mm to 75 mm) up to about 150 mm from the top and covered with galvanised iron or fibre cement sheets and backfilled with soil. Distribution of effluent may be effected by placing open jointed agricultural pipes in the top of the broken stone. Where trenches are constructed adjacent to each other, the distance between them should be at least twice the depth of the deepest trench.

In very sandy soil, honeycomb walls may be built to support the trench sides. It is beneficial for percolation trenches to have a resting period. This can be achieved by providing the required area in two or more trenches and by using them in rotation.

4.2 ANAEROBIC PONDS

4.2.1 INTRODUCTION

Waste stabilisation ponds have been used extensively for the treatment of municipal and industrial wastewaters. Anaerobic ponds are single-stage, continuous-flow, anaerobic reactors operating at ambient temperatures and low volumetric organic loading as a pretreatment method. On a wastewater management scheme the zero-energy demand of a waste stabilisation pond series for the effective removal of organic and microbiological loading remains a valuable tool for sustainable development.

4.2.2 MECHANISM OF TREATMENT AND ADVANTAGES

Anaerobic bacteria degrade organic materials in the absence of oxygen and produce methane and carbon dioxide. Benefits include a reduction of total bio-solids volume of up to 50-80% and a final waste sludge that is biologically stable can serve as rich humus for agriculture when the pond requires de-sludging after a period of several years.

Advantages of the process include:

1. No, or very low, energy demand
2. Low investment costs
3. Low space requirements
4. Is applicable at small as well as large scale
5. Low production of excess sludge, which is well stabilised
6. Low nitrogen and phosphorus requirements
7. A high loading capacity (5 to 10 times that of aerobic treatment)

4.2.3 POND DESIGN

Anaerobic ponds are deep treatment ponds that exclude oxygen and encourage the growth of bacteria, which break down the effluent. The anaerobic pond acts like an uncovered septic tank. Sludge is deposited on the bottom and a crust forms on the surface. Anaerobic ponds are commonly 2 to 5 m deep and receive a high organic loading (usually > 100 g BOD/m^3.d equivalent to > 3000 kg/ha.d for a depth of 3 m).

Anaerobic ponds can be satisfactorily designed, and without risk of odour nuisance, on the basis of volumetric BOD loading, B_v (g/m^3.d), which is given by Equation (4.2):

$$B_v = L_i Q / V_p \qquad \ldots\ldots(4.2)$$

Where:

Li = influent BOD, mg/ℓ (g/m^3)

Q = flow (m^3/d)

V_p = anaerobic pond volume (m^3)

Loadings of 100 g BOD/m^3.d up to 400 g BOD/m^3.d are practiced, but odour generation can occur above 200 g BOD/m^3.d. It is therefore beneficial to provide for recycle of final aerobic effluent back to the head of a highly loaded anaerobic pond at about 0.5:1 when treating strong wastes.

These loadings are high relative to the amount of oxygen entering the pond, which maintains anaerobic conditions at the pond surface. Anaerobic ponds do not contain algae, although occasionally a thin film of mainly *Chlamydomonas* can be seen at the surface. They work well in warm climates and can attain 60-85% BOD removal even at relatively short retention times. Retention times of up to 4 days may be required for strong wastes but usually 1 to 2 days is sufficient (for a BOD of up to 300 mg/ℓ, one day is sufficient at temperatures >20°C).

4.2.4 PERFORMANCE

Anaerobic ponds reduce N, P, K and pathogenic microorganisms by sludge formation and the release of ammonia into the air. As a complete process, the anaerobic pond serves to:

1. Separate out solid from dissolved material allowing the solids to settle as a bottom sludge
2. Dissolve organic material further
3. Break down biodegradable organic material
4. Store undigested material and non-degradable solids as bottom sludge.

They therefore serve as a good first stage for aerobic treatment either in the form of oxidation ponds or as a mechanical plant. Incorporation of an anaerobic pond at the head of a pond system reduces the size of the Primary Aerobic Pond (which is the capacity limiting unit) by about 50% or more. Normally, a single anaerobic pond in each treatment train is sufficient if the strength of the influent wastewater is less than 1000 mg/ℓ BOD. However it may often be preferable to split the anaerobic pond volume in two so that two units can work in parallel, giving the flexibility to take one unit off line at a time for sludge removal purposes. Careful selection of loadings will avoid overload conditions with one pond on line, and under loaded conditions with two on line.

In the past designers have been cautious about incorporating anaerobic ponds into a design, in case they create odour. However, recent work suggests that maximum design volumetric loadings may be increased to 300 g BOD/m^3.d at 20°C without odour nuisance. The option of recycle of aerobic effluent is also available. Furthermore, Mara and Pearson (1986) propose an alternative limit of a maximum sulphate volumetric loading rate of 500 g SO_4 /m^3.d (equivalent to 170 g S/m^3.d) in order to avoid odour nuisance which will favour areas with low conductivities in their water supply.

Strong ammonia inhibition in anaerobic ponds can occur at concentrations >80 mg NH_3-N/ℓ and may impact significantly on COD and BOD reduction, reducing this to as low as 10% in primary anaerobic ponds

Table 4.2.1 below shows the expected BOD removal at a typical loading for various retention times, but is presented as a rough guide. Performance tends to be site-specific in many cases.

TABLE 4.2.1: BOD removals in anaerobic ponds loaded at 250 g BOD/m^3.d (Mara, 1976).

Retention Time (Days)	BOD Removal %
1	50
2.5	60
5	70

4.3 ANAEROBIC CONTACTORS

4.3.1 INTRODUCTION

Anaerobic biological treatment of wastewaters has gained considerable recognition in recent years. In some cases, the anaerobic processes currently being used have been shown to be more cost effective treatment alternatives than the more commonly used aerobic treatment processes, particularly where highly concentrated organic wastes are concerned. Anaerobic treatment is being repeatedly proposed as an alternative to aerobic treatment for high-strength effluents with COD concentrations greater than or equal to 1000 mg/ℓ. Use of an anaerobic treatment alternative offers many advantages, including reduced energy requirements. Also, because methane is produced during this process, anaerobic treatment can often be a net energy producer.

Biomass production is generally between 10 and 20% of that of activated sludge. Anaerobic treatment also greatly reduces the requirements for nitrogen and phosphorus, imposes minimum constraints of food to microorganism (F/M) control, and generally has a lower sensitivity toward heavy metal poisoning.

4.3.2 ENVIRONMENTAL FACTORS

During the methane fermentation process, the methane bacteria are limited in the quantity of energy they can obtain from substrate fermentation, because the majority of the substrate energy is lost in the methane gas produced. As a result, the rate of growth is restricted. In addition, the rate of substrate use per unit of organism is relatively low. The combination of these two factors tends to restrict the overall rate of substrate use by methane bacteria. Therefore optimum environmental conditions are necessary for satisfactory rates of methane fermentation to occur.

Optimum methane fermentation has generally been found to occur between the pH range of 6.0 to 8.5 but drops rapidly outside this range. Values of pH below 6 or above 8.5 have been found to be restrictive and somewhat toxic to methane bacteria. For proper pH control, sufficient alkalinity is essential because it acts as a buffer to the system. Alkalinity is produced from the breakdown of organic material and, at a typical fermentation pH around 7.0, is present primarily in the form of bicarbonates.

The optimum temperature for methane fermentation is in the range of 30 to 35°C. As the temperature drops below this range for a given hydraulic retention time (HRT), the quantity of substrate removed from the system decreases. Unless HRT is increased at temperatures below 2O°C, anaerobic treatment is usually not feasible because biomass will be removed from the system faster than it is being produced and eventually all the biomass will be lost.

4.3.3 PROCESS DESCRIPTION

An anaerobic contact process consists of a heated digestion tank followed by a settling tank which removes the active biological suspended solids from the effluent flow and returns them to the digestion tank to increase the solids contact. The process involves two parts:

1. A contact portion where the raw waste is intimately mixed with a previously developed anaerobic sludge culture
2. A separation portion where the activated sludge particles are separated from the treated liquor and recycled to the contact unit.

Typical industrial wastes which have been treated with an anaerobic contact process are:

- Waste maize starch
- Whisky distillery
- Cotton kiering
- Citrus
- Brewery
- Starch gluten
- Wine
- Yeast
- Molasses
- Meat packing
- Potato starch

Wastewater enters an anaerobic reactor that is normally maintained at a temperature range of 35 to 37°C, although ambient temperatures are acceptable in warm climates. The readily biodegradable BOD in the influent is metabolised and sludge, methane, and carbon dioxide are produced. After leaving the anaerobic reactor, the treated wastewater enters a clarifier or sludge separator, where most of the solids are separated and recycled to the anaerobic reactor. The effluent from the reactor tends to be saturated with carbon dioxide and methane, which releases into the atmosphere and can cause flotation of the solids in the clarifier. For this reason a vacuum degasser is often used preceding the clarifier. Recycle of the clarifier underflow stream allows a high microorganism concentration to be maintained in the reactor, thereby reducing process retention time.

A commonly used second stage to the anaerobic contact process is a conventional activated sludge plant where further degradation of the waste occurs. Excess sludge from the aerobic treatment process is digested in the anaerobic reactor which reduces the total excess sludge production and results in a higher yield of methane.

4.3.4 PERFORMANCE

Typically, BOD treatment efficiencies of 80% to 90% are achieved in the anaerobic stage. The remaining organic material is then degraded a further 80% to 90% in the second aerobic stage, resulting in a total BOD reduction between 95% and 99% percent. **Table 4.3.1** below represents performance data from various applications of the anaerobic contact process.

TABLE 4.3.1: Anaerobic contact process performance data from various applications.

Type of Waste	Hydraulic Retention Days	Digestion Temperature °C	BOD of Raw Waste mg/ℓ	BOD Load kg/m^3.d	BOD Removal %
Maize Starch	3.3	23	6 280	1.77	88
Whisky Distillery	6.2	33	25 000	4.01	95
Cotton Kiering	1.3	30	1 600	1.19	67
Citrus	1.3	33	4 600	3.43	87
Brewery	2.3		3 900	2.04	96
Starch Gluten	3.8	35	14 000	1.61	80
Wine	2.0	33	23 400	11.72	85
Yeast	2.0	33	11 900	5.97	65
Molasses	3.8	33	32 800	8.76	69
Meat Packing	1.3	33	2 000	1.77	95
Meat Packing	0.5	33	1 380	2.50	91
Meat Packing	0.5	35	1 430	2.63	95
Meat Packing	0.5	29	1 310	2.44	94
Meat Packing	0.5	24	1 110	2.10	91
Potato Starch	2.2	23		0.77	52
Potato Starch	1.3	24		1.93	43

4.3.5 DESIGN

From **Table 4.3.1** above it can be inferred that the performance of anaerobic contact is highly dependent on the waste being treated. As a rule of thumb, hydraulic retentions of about 1.5 days or longer are reasonably safe with solids retention times of 40 to 60 days. With occasional exceptions, BOD loadings of about 2 kg/m^3.d would be acceptable. The process is temperature dependent and unheated reactors should be somewhat more conservatively sized.

The success of the anaerobic contact process rests on the separation of the anaerobic solids in the clarifier after the reactor. Unless effective vacuum degassing is provided the solids will tend to float over the discharge weir of the clarifier and be lost from the system. As the anaerobic sludge is slow-growing this can soon lead to process failure. Very low surface loadings should also be used when designing the clarifier. Typical rise rates are 0.3 to 0.4 m/h.

4.4 ANAEROBIC BAFFLED CONTACTORS

4.4.1 INTRODUCTION

An Anaerobic Baffled Reactor, or ABR, in common with most anaerobic processes, is a design approach which can significantly reduce primary sludge production. The ABR design is based around the creation of a simple compartmentalised reactor design which can be free standing or in some cases has been installed inside an existing primary basin. In the latter case the compartmental design encourages anaerobic biological degradation of the primary sludge in situ, thereby reducing the amount of sludge activity within the primary tank.

As with the anaerobic contact process the key to the successful operation of an ABR is to separate the solids retention times (SRT) from the hydraulic retention times (HRT), thus making it possible to anaerobically treat wastewaters at relatively short retention times (4 to 10 hours). ABRs can thus be viewed as an attractive alternative to aerobic treatment.

The benefits associated with ABR technology include:

1. Treatment of moderate to low strength wastes in a primary stage tank: Previous work indicates that COD removal efficiencies of the order of 50 - 80% can be expected. This will provide significant benefits in terms of reducing the loadings on the secondary aerobic treatment plant.
2. Reduced sludge production: The long solids residence time in the ABR will produce significant in situ degradation of the primary sludge solids. Furthermore, due to the significantly decreased load on the secondary treatment plant, minimal secondary sludge would be expected. Results to date indicate a reduction in sludge production of between 30 - 50% can be realised.
3. Low operating cost: The operating costs of an ABR are expected to be of a similar order to those of operating a primary settlement tank. The ABR can be regarded as a low cost treatment option, especially when compared to the high energy costs associated with conventional activated sludge secondary treatment.
4. Robust to shock loads: As outlined below, previous work has indicated that the ABR is a stable process which is robust to both organic and hydraulic shock loads.

4.4.2 LAYOUT

Anaerobic baffled reactors are basically tanks with dividing internal baffles to create a series of compartments to approximate to plug flow. The tanks are usually rectangular and have vertical baffles to create successive up and down flow patterns. Solids formed during the process settle to the bottom of the tank and the water passes through what eventually is essentially an anaerobic sludge blanket at alternate passes. The process is therefore suspended growth rather than attached growth as in an anaerobic submerged filter.

4.4.3 DESIGN

Although many benefits are claimed for the anaerobic baffled reactor, its design criteria are not well established and what information is available suggests similar loadings to be applicable as for other anaerobic processes such as UASB reactors, and anaerobic contactors. Hydraulic retention times vary from about 6 hours to 2 days and COD loadings range between 3 and 10 kg COD/m^3.d. The crux of successful high rate anaerobic treatment is the retention of the anaerobic organisms in the system, whether on fixed film or in a suspension or in a granular bed. Solids retentions of 40 to 60 days or even longer are required for good COD removals.

As for submerged anaerobic filters, facilities for gas (methane) draw-off should be provided as well as drains for excess solids at suitable points.

CHAPTER 5

5. PONDS AND WETLANDS

5.1 INTRODUCTION

Waste stabilisation ponds are large shallow basins enclosed by earthen embankments in which raw sewage is treated by natural processes involving both algae and bacteria. Because of the use of natural processes, the rate of oxidation is slow and as a result, long hydraulic retention times are employed, retentions of 30 to 50 days being normal.

Ponds have considerable advantages, particularly regarding costs and maintenance requirements and the adequate removal of faecal bacteria, over other methods of treating the sewage from communities of more than about 100 people. Ponds are the most important method of sewage treatment in hot climates where sufficient land is normally available and where the temperature is most favourable for their operation.

There are three major types of pond relying on natural processes:

1. Facultative
2. Maturation
3. Anaerobic ponds

In addition to this, aerated lagoons are ponds fitted with mechanical aeration devices that enable smaller units to be constructed.

A variation on maturation ponds is the construction of a maturation river as the last maturation unit in the treatment sequence. This is essentially a maturation pond with dividing walls to ensure plug flow and eliminate short-circuiting. Apart from assisting bacterial die-off, this also greatly reduces the concentration of phyto-plankton (algae) in the final effluent.

5.1.1 TREATMENT SEQUENCES

It is not normal, and is also not good engineering practice, to install a single pond for treatment of effluent. Ponds are normally constructed in series and follow certain sequences. The simplest sequence is a facultative pond followed by a secondary pond and two or more maturation ponds. The facultative pond is sized on the BOD load entering the pond and is usually 30 to 50 days retention. The secondary pond allows biological purification reactions to go further to completion and is often about 10 to 15 days retention. The maturation ponds are smaller ponds of about 5 days retention connected in series to assist bacterial (*E. coli*) die-off.

If the wastewater is strong, one may have one or two anaerobic ponds at the head of the system followed usually by a somewhat smaller facultative pond, a secondary pond and two or more maturation ponds.

If land is scarce or expensive one may have an optional anaerobic pond sequence followed by an aerated lagoon which is usually much smaller than a normal facultative pond, followed in turn by a secondary pond and maturation ponds. The inclusion of a maturation river is an option as the final unit in any of these treatment sequences.

5.1.2 EFFLUENT QUALITY

Pond systems tend to produce very stable effluents that do not change much in quality from day to day, due to the long retention in the system. The effluent quality is usually very good, although ammonia reduction is a problem in winter in colder areas. The suspended solids content of the effluent tends to be on the high side unless a maturation river is specified as the final treatment unit, and the effluent may not meet discharge standards in respect of suspended solids and ammonia at certain times.

The effluent from pond systems is often irrigated, which is a highly suitable disposal route. If the system is carefully designed, a pond system effluent would be preferable in many cases to an effluent from a mechanical plant that is not well operated.

5.2 FACULTATIVE PONDS

5.2.1 INTRODUCTION

Facultative ponds are the most commonly constructed type of pond. They normally receive raw sewage or sewage that which has received only preliminary treatment. They are, however, becoming increasingly used to treat the settled effluent from septic tanks and anaerobic pre-treatment ponds.

The term 'facultative' refers to a mixture of aerobic and anaerobic conditions, and in a facultative pond aerobic conditions are maintained in the upper layers while anaerobic conditions exist towards the bottom. Although some of the oxygen required to keep the upper layers aerobic comes from re-aeration through the surface, most of it is supplied by the photosynthetic activity of the algae that grow naturally in the pond where considerable quantities of both nutrients and incident light energy are available. Indeed, so profuse is the growth of algae that the pond content is often green in colour. The pond bacteria utilise this 'algal' oxygen to oxidise the organic waste matter. One of the major end-products of bacterial metabolism is carbon dioxide which is readily utilised by the algae during photosynthesis as their demand for it exceeds its supply from the atmosphere. Thus there is an association of mutual benefit ('symbiosis') between the algae and bacteria in the pond. Since photosynthesis is a light-dependent activity there is a diurnal variation in the amount of dissolved oxygen present in the pond, and a similar fluctuation in the level of the 'oxypause' (the point below the surface at which the dissolved oxygen concentration becomes zero) occurs.

The pH of the pond content also follows a daily cycle, increasing with photosynthesis to a maximum which may be as high as 10. This happens because at peak demand, algae remove CO_2 from solution more rapidly than it is replaced by bacterial respiration. As a result the bicarbonate ions present dissociate to provide more CO_2, increasing the pH, through their consumption

5.2.2 MIXING

Wind and heat are the two factors of major importance which influence the degree of mixing that occurs within a pond. Mixing fulfils a number of vital functions in a pond: it minimises hydraulic short-circuiting and the formation of stagnant regions and it ensures a reasonably uniform vertical distribution of BOD, algae and oxygen. Mixing is the only means by which the large numbers of non-motile algae can be carried up into the zone of effective light penetration (the 'photic' zone). Since the photic zone comprises only the top 150 to 300 mm of the pond, much of the pond contents would remain in permanent darkness if mixing did not occur. Mixing is also responsible for the transportation of the oxygen produced in the photic zone to the bottom layers of the pond. Good mixing thus increases the safe BOD load that can be applied to a pond.

Wind is the most important factor in mixing. The depth to which wind-induced mixing is effective is largely determined by the distance the wind is in contact with the water (the 'fetch'). An unobstructed contact length of about 100 m is required for maximum mixing by wind action. The importance of wind action has been demonstrated by an experiment in Zambia where a well-operating pond was totally fenced off by a wind-proof enclosure. Within three weeks the pond had gone anaerobic.

5.2.3 SLUDGE LAYER

As the sewage enters the pond most of the solids settle to the bottom to form a sludge layer. At temperatures greater than 15°C intense anaerobic digestion of the sludge solids occurs; as a result, the thickness of the sludge layer depth is rarely more than about 250 mm and often much less. Desludging is only rarely required, possibly once every 10 to 15 years, and many ponds operate much longer without desludging being necessary.

At temperatures greater than 22°C the evolution of methane gas is often sufficiently rapid to buoy sludge particles up to the surface where drifting sludge mats are formed. These must be removed (together with any other floating debris or scum) so that they do not prevent the penetration of light into the photic zone.

The soluble products of fermentation diffuse into the bulk liquid of the pond where they are oxidised further. The seasonal variation of the rate of fermentation (which increases approximately sevenfold with each 5°C rise in temperature) explains why the COD in the pond often remains ostensibly constant throughout the year in spite of the changes in temperature. During summer the degradation rate is high and, from the theory, a low equilibrium COD in the pond is established. However, the COD load received from the sludge is high. During winter the degradation rate is low, establishing a relatively high equilibrium COD in the pond, but a low COD load is received from the sludge. The two processes, operating simultaneously, tend to cancel out, and decrease the cyclic variation of pond COD.

5.2.4 DEPTH

The recommended depth for a facultative pond is 1.0 to 1.5 m. Pond depths less than 1.0 m do not prevent the emergence of vegetation. This must be avoided as otherwise the pond becomes an ideal breeding ground for mosquitoes and midges. With depths greater than 1.5 m the oxypause is too near the surface, with the result that the pond is predominantly anaerobic rather than predominantly aerobic. This is undesirable, as the pond would have an unacceptably low factor of safety in normal operation and so be less able to cope with a fluctuating load or a sudden slug of heavy pollution.

In arid climates, evaporation rates are high and water losses should be minimised by increasing the depth to about 2 m and thus reducing the surface area. In cold climates (e.g. at high altitude) similar depths are used so as to preserve as much of the thermal energy of the influent sewage as possible. These considerations are usually more important in these extremes of climate than those concerned with the position of the oxypause.

5.2.5 TEMPERATURE

A hot climate is ideal for pond operation. Solar radiation is intense and as a result, pond temperatures are high and there is more than an adequate intensity of light. The long daylight hours enable algal photosynthesis to occur for extended periods and so provide a reserve of dissolved oxygen for use during the night. In order to ensure that the pond works satisfactorily at all times, it must be designed for the worst (i.e. coldest) conditions. The mean temperature of the coldest month is commonly used as the design temperature.

5.2.6 ADVANTAGES OF PONDS.

The major disadvantage of ponds is that they require much larger areas of land than other forms of sewage treatment. However, in many countries, especially tropical developing countries, this is rarely a disadvantage of any importance since sufficient land is normally available at relatively low cost.

Some advantages of ponds are:

1. **They can achieve any required degree of purification at the lowest cost and with the minimum of maintenance by unskilled operators.**

This has been demonstrated in many studies but obviously the lowest cost aspect would not necessarily hold in highly developed countries where land may be scarce and expensive.

2. **The removal of pathogens is considerably greater than that in other methods of sewage treatment,**

The effluent from a series of three ponds usually contains less than 5 000 *E. coli*/100 mℓ whereas the final effluent from a conventional biofilter works (humus tank effluent) typically contains about 5 000 000 *E. coli*/100 mℓ and from an activated sludge plant about 240 000 *E. coli*/100 mℓ. Cysts and ova of intestinal parasites, which are commonly present in conventional effluents, are not found in maturation pond effluent. The pond habitat is fortunately also unsuitable for the growth of the snail hosts of parasitic nematode worms such as *Schistosoma* spp. and *Clonorchra sinertsis.*

3. **They are well able to withstand both organic and hydraulic shock loads.**

The long retention times (20-30 d in facultative ponds receiving raw sewage) ensure that there is always sufficient dilution available for short shock overloads.

4. They can effectively treat a wide variety of industrial and agricultural wastes.

Wastes that are readily biodegradable (such as those from dairies. slaughter houses and food-processing factories) have been successfully treated with domestic sewage in facultative ponds. Anaerobic ponds are particularly advantageous for strong wastes that are to be treated alone.

5. They are tolerant of heavy metals

The high pH of the pond causes toxic heavy metals to precipitate as hvdroxides and are so removed into the sludge layer. Experiments conducted in Israel showed that a heavy metal concentration of 30 mg/ℓ (6 mg/ℓ each of cadmium, hexavalent chrome, copper, nickel and zinc) did not affect pond performance, but that 60 mg/ℓ did. Thus a facultative pond normally operating above pH 8 should be able to tolerate the heavy metals in municipal sewage for a considerable period of time before their accumulation in the sludge laver would affect its performance.

6. They can easily be designed so that the degree of treatment is readily altered.

By designing the pond outlet structure so that the top water level can be varied, the retention time and hence the degree of treatment can be altered. This can be a useful device when a pond receives a seasonal waste (e.g. a food-processing waste) in addition to its normal sewage flow.

7. The method of construction is such that, should at some future date the land be required for some other purpose, it can easily be reclaimed.

All that is required is the removal of the pond inlet and outlet structures and of the paving slabs at top water level. The ground should then be levelled. The sale of former pond land will usually yield a substantial fixed property profit to the municipality.

8. The algae produced in the pond are a potential source of high-protein food.

Fish have been successfully grown in maturation ponds. The sale of fish can bring in substantial revenue and so reduce the running costs of the treatment works. Ducks may also be reared on maturation ponds.

5.2.7 DESIGN PRELIMINARIES

The design of waste stabilisation ponds is part rational and part empirical. The depth is selected with due regard to the site and to the considerations given above. The range of depths most commonly used for facultative ponds is 1 to 1.5 m. The length and breadth are usually in the ratio 2 to 3 to 1

5.2.8 DESIGN OF FACULTATIVE PONDS.

5.2.8.1 South African Experience

Early work, largely by the National Institute of Water Research (NIWR), an Institute of the CSIR, produced design criteria which related the BOD load on a primary pond to the surface area required to produce sufficient natural oxygenation to avoid anaerobic conditions. On this basis ponds can be designed on criteria of kg BOD/ha.d, with the safe operating range generally lying between 120 and 180 kg BOD/ha.d. The lower loadings are applicable to cooler areas of South Africa, and the higher loadings to the warmer sub-tropical parts. These loading criteria are based on winter temperatures and can safely be used when a quick estimate of a primary (facultative) pond area is required.

Work carried out by G. V. R. Marais and co-workers of the University of Cape Town and published in the 60's and 70's developed a more comprehensive model of pond behaviour.

Based on actual operating experience of two ponds, one in a tropical climate in Zambia, and one in a cold climate in Canada, and assuming solids deposition in winter, and organic breakdown of accumulated solids in summer, with good mixing of pond contents, it was found that the effluent BOD concentration changed very little throughout the year. The rate constant in the developed model was relatively independent of temperature in the range 10ºC to 32ºC and also over a fairly wide range of retention times.

In this model the effluent BOD becomes limiting as too high an effluent BOD will increase the probability of anaerobic conditions occurring in the pond. This is typically in the range 50 to 70 mg/ℓ, and a figure of 55 to 60 mg/ℓ is often assumed.

The equation for pond retention (typically for a 1.5 m deep pond) is

$$R = (P_i / P_o - 1) / K_e \qquad \text{……………………………………………………..(5.1)}$$

where R - pond retention (days)

P_i - influent BOD (mg/ℓ)

P_o - effluent BOD (mg/ℓ)

K_e - equivalence rate constant = 0.11 (for variation in the range 10 – 32ºC)

5.2.8.2 Worked Example

Task - Size a primary facultative pond for a population of 2 000 people living on the Highveld

<u>Sewage load</u>

Assume a sewage volume contribution of 150 l/cap.d (Section 2.3. Table 2.3.1)

Design volume = 2 000 x 150/1000 = 300 kℓ/d

Assume a per capita BOD load of 65 g/cap.d (Section 2.5. Table 2.5.1)

Design BOD load = 2 000 x 65/1000 = 130 kg/d

BOD concentration = 130/300 x 1000 = 433 mg/ℓ

Pond Size based on NIWR Loadings

Select a BOD loading of 120 kg/ha.d

Select a pond depth of 1.5 m

Pond area = 130/120 = 1.08 Ha = 10 833 m^2

Pond volume = 10 833 x 1.5 = 16 250 m^3

Retention = 16 250 /300 = 54 days

Pond Size based on Marais Design sequence

Select an effluent BOD of 60 mg/ℓ

Retention = (433/60-1) /0.11 = 66.7 days

Pond volume = 300 x 66.7 = 20 015 m^3

Assume depth of 1.5 m

Area = 20015/1.5 = 13 343 m^2

The Marais design sequence gives a somewhat larger pond size. As this is the more conservative, and is also the more rigorous approach it would probably be advisable to go with the larger unit.

5.2.9 OTHER CRITERIA

5.2.9.1 First order kinetics

The simplest approach to the rational design of facultative ponds is to assume that they are completely mixed reactors in which BOD removal follows first order kinetics.

$$\frac{L_{\partial}}{Li} = \frac{1}{1 + k_1 t^*} \qquad \text{……………………………………………………….(5.2)}$$

Rearranging:

$$t^* = \left(\frac{L\alpha}{Li} - 1\right)\frac{1}{k_1} \qquad \text{……………………………………………………….(5.3)}$$

substituting for *t** gives:

$$A = \frac{Q\,(L_I/L\alpha - 1)}{D\,k_1} \qquad \text{………………………………………………………. (5.4)}$$

Work in South Africa has suggested that in order to maintain the pond contents predominantly aerobic (rather than predominantly anaerobic) L_α should be in the range 50 to 70 mg/ℓ for pond depths of 1 to 1,5 m. The value of k_1 is about 0.3 d^{-1} at 20 °C and its variation with temperature is described by the following equation:

$$k_1 = 0.3(1.05)^{T-20} \quad \text{……(5.5)}$$

Substitution of this equation into Equation (5.4) and selection of L_e as 60 mg/ℓ gives the following design equation for A:

$$A = \frac{Q(L_i - 60)}{18\,D(1.05)^{T-20}} \quad \text{……(5.6)}$$

Where:

L - BOD (i = incoming; e = exit)

T - Temperature (taken as the mean temperature of the coldest month)

A - Pond area (ha)

D - Pond Depth (m)

Q - Flow (m^3/d)

5.2.9.2 McGarry and Pescod empirical procedure

An analysis of operational data from many facultative ponds all over the world (**Figure 5.2.1**) showed that the maximum BOD surface loading that could be applied to a facultative pond before it failed (i.e. became completely anaerobic) was related to the mean monthly ambient air temperature (this being taken as the most convenient measure of climate for which records exist) as follows:

$$\lambda = 11.2\,(1.054)^T \quad \text{……(5.7)}$$

where:

λ = maximum BOD loading, (kg/ha.d)

T = temperature, °C.

However, ponds are not normally designed to operate just at their point of failure. Therefore the introduction of a safety factor for design purposes would be required. For example Equation (5.7) could be modified to:

$$\lambda_d = 7.5\,(1.054)^T \quad \text{……(5.8)}$$

where λ_d is the design loading, kg/ha.d.

An alternative design equation for λ_d is the straight line relationship shown in **Figure 5.2.1:**

$\lambda_d = 20\,T - 120$ ……………………………………………………(5.9)

where T is in °C.

The design equation for A is then simply obtained from equation (5.4) as:

$$A = \frac{L_i Q}{2T - 12} \qquad \text{……………………………………………………(5.10)}$$

The degree of BOD removed in facultative ponds was found to be related
to the BOD_s loading as follows:

$\lambda_r = 0.725\,\lambda_s + 10.75$ ……………………………………………………(5.11)

Where:

λ_r = BOD removal (kg/ha.d)

λ_s = BOD load (kg/ha.d)

Equation (5.9), if restricted to temperatures between 5°C and 30°C is probably as realistic a design equation as any other and it has the advantage of simplicity of form.

5.2.9.3 Mara Empirical Equation

In a more recent publication (1987) Mara gives the following global design equation

$\lambda_s = 350(1.107 - 0.002T)^{T-25}$ ……………………………………………………(5.12)

5.2.9.4 WORKED EXAMPLE

Task - Size a primary facultative pond for a population of 2 000 people living on the Highveld

Sewage load

Assume a sewage volume contribution of 150 ℓ/cap.d (Section 2.3. Table 2.3.1)

Design volume = 2 000 x 150/1000 = 300 kℓ/d

Assume a per capita BOD load of 65 g/cap.d (Section 2.5. Table 2.5.1)

Design BOD load = 2 000 x 65/1000 = 130 kg/d

BOD concentration = 130/300 x 1000 = 433 mg/ℓ

Pond Size from First order kinetics

Assume depth of 1.5 m

Select minimum temperature of 10°C

From equation (ii) A = $[300(433/60-1)]/[1.5 \times 0.3(1.05)^{T-20}]$ = 7 970 m^2

Pond volume = 7970 x 1.5 = 11 954 m^3

Retention = 11 954/300 = 40 days

Pond size from Empirical data equations

Temperature in °F = 10 x 1.8 +32 = 50

Loading from equation (v) = $7.5\ (1{,}054)^{50}$ = 104 kg/Ha.d

Pond area (A) = 130/104 x 10 000 = 12 500 m^2

Pond volume = 12 500 x 1.5 = 18 750 m^3

Retention = 18 750 /300 = 63 days

Loading from equation (vi) = 20 x 10 – 120 = 80 kg/Ha.d

Pond area (A) = 130/80 x 10 000 = 16 250 m^2

Pond volume = 16 250 x 1.5 = 24 375 m^3

Retention = 24 375 /300 = 81 days

Loading from equation (vii) = $350(1.107 - 0.002 \times 10)^{10-25}$ = 100 kg/ha.d

Pond area (A) = 130/100 x 10000 = 13 000 m^2

Pond volume = 13 000 x 1.5 = 19 500 m^3

Retention = 19 500/300 = 65 days

The Marais sequence, the McGarry and Pescod curve [Equation (v)], and the Mara global design empirical equation [equation (vii)], all give very similar results and would probably give the best design sizing.

5.2.10 2.8.3 RECIRCULATION.

The design loadings given above can be exceeded by about 30% in areas with mild winters by recirculation of final pond effluent at about 0.5:1 to the head of the facultative pond. This has the effect of preventing or delaying the onset of anaerobic conditions predominating in winter on heavily loaded ponds and is a means of assisting systems which are becoming overloaded.

5.3 MATURATION PONDS

5.3.1 INTRODUCTION

Maturation ponds are used as a second stage to facultative ponds and secondary ponds which are used for COD reduction. Their main function is the destruction of pathogens. Faecal bacteria and viruses die off reasonably quickly owing to what is to them an inhospitable environment. The cysts and ova of intestinal parasites have a relative density of about 1.1 and as a result of the long retention times they settle to the bottom of the pond where they eventually die. The removal of BOD in maturation ponds is small: two ponds in series, each with a retention time of 7 d, are required to reduce the BOD from about 50-70 mg/ℓ to less than 25 mg/ℓ.

Maturation ponds are wholly aerobic and are able to maintain aerobic conditions at depths of up to 3 m. However, most often, the depth of a maturation pond is taken as the same as that of the associated facultative pond (1 to 1.5 m). This is advisable, as well as usually being convenient, since the destruction of viruses is better in shallow ponds than in deep ones.

FIGURE 5.3.1: Typical maturation pond

The effectiveness of maturation ponds in removing pathogens is conveniently assessed by the removal of *E. coli*. With proper design, removals greater than 99.99% can be achieved. In these circumstances no difficulty should be experienced in satisfying an effluent standard of less than 5000 *E. coli* /100 mℓ.

5.3.2 DESIGN

5.3.2.1 COD and BOD Removal

To produce an effluent with a BOD of less than 25 mg/ℓ it has been found that 2 maturation ponds in series, each with a retention time of 7 d, are required. This assumes that the BOD of the influent (i.e. of the facultative or secondary pond effluent) is not more than about 75 mg/ℓ. COD is not necessarily a good indicator for pond effluent quality as the prevalence of algae gives rise to spuriously high readings. If using COD it is advisable to measure COD on a filtered sample when assessing pond effluents.

5.3.2.2 Bacterial reduction

The reduction of faecal bacteria in a pond (anaerobic, facultative or maturation) has been found to follow first order kinetics. The appropriate version of equation:

$$N_e = \frac{N_i}{1 + K_b t^*}$$

…………………...(5.13)

Where N_e = number of *E. coli*/100 mℓ of effluent

N_i = number of *E. coli*/100 mℓ of influent

K_b = first order rate constant for FC removal, d⁻'.

For n ponds in series equation (5.13) becomes:

$$N_e = \frac{N_i}{(1 + K_b t_1^*)(1 + K_b t_2^*) \ldots .(1 + K_b t_n^*)}$$

…………………...(5.14)

where t* = retention time in the nth pond

The value of K_b is extremely temperature sensitive: it is given by the equation:

$$K_{b(t)=2{,}6(1{,}19)^{T-20}}$$

…………………... (15.15)

Where $K_{b(t)}$ = the value of K_b at T(ºC.)

A reasonable design value of N_i is 4 x 10^7 FC/100 mℓ; this is slightly higher than average values normally found in practice. The best design procedure is to calculate the retention time in the facultative pond and determine the value of N_e from Equation (15.3) which results from having two maturation ponds each with t* = 7 d. If this value of N_e is unacceptable, then three or more maturation ponds each with t* = 5 d are chosen and N_e recalculated on this basis.

Although *E. coli* are commonly used to indicate the removal of faecal organisms in a series of ponds, there is evidence that some pathogenic bacteria do not die off as quickly as do *E. coli*. For example, a species of salmonella was found to have a K_b value of 0.8 d^{-1} in the same pond as *E. coli* with a K_b value of 2.0 d^{-1}. Also drug-resistant coliforms are known to die off more slowly than those without resistance genes.

5.3.2.3 Worked Example

Task **-** Design a maturation pond system to produce a final effluent with an *E. coli* count of less than 1 000/100 mℓ for a population of 2 000 persons.

<u>Sewage volume and pond sizing</u>

Assume a sewage volume contribution of 150 ℓ/cap.d (Section 2.3. Table 2.3.1)

Design volume = 2 000 x 150/1000 = 300 kℓ/d

Select pond unit size of 5 day retention

Select a depth of 1.5 m

Pond volume for a retention of 5 days = 5 x 300 = 1 500 m^3

Pond area for a retention of 5 days = 1 500/1.5 = 1 000 m^2

<u>Calculation of bacterial die-off in winter</u>

Assume minimum temperature of 12°C

$K_b = 2.6\,(1.19)^{12-20} = 0.65\ d^{-1}$

E Coli after first pond = $4 \times 10^7/(1 + 0.65 \times 5) = 9.4 \times 10^6$ *E. coli*/100 mℓ

E Coli after second pond = $9.4 \times 10^6 / (1 + 0.65 \times 5) = 2.2 \times 10^6$ *E. coli*/100 mℓ

After third pond = 5.2×10^5 *E. coli*/100 mℓ

After fourth pond = 1.2×10^5 *E. coli*/100 mℓ

After fifth pond = 28 800 *E. coli*/100 mℓ

After sixth pond = 6 800 *E. coli*/100 mℓ

After seventh pond = 1 600 *E. coli*/100 mℓ

<u>Calculation of bacterial die-off in summer</u>

Assume temperature of 26°C

$K_b = 2.6(1.19)^{26-20} = 7.38\ d^{-1}$

E Coli after first pond = $4 \times 10^7/(1 + 7.38 \times 5) = 1.06 \times 10^6$ *E. coli*/100 mℓ

E Coli after second pond = $1.06 \times 10^6 / (1 + 0.65 \times 5) = 27\ 850$ *E. coli*/100 mℓ

After third pond = 734 *E. coli*/100 mℓ

<u>Comment</u>

As can be seen from the calculations the *E. coli* die-off is very strongly temperature-dependent. One would need 8 maturation ponds in winter but only three in summer to get below 1000 *E. coli*/100mℓ. In practice 8 ponds would be uneconomical to build and one would normally construct three or at most four ponds, allowing the *E. coli* count to increase somewhat in the few colder months.

5.4 MATURATION RIVERS

5.4.1 INTRODUCTION

Maturation rivers are essentially maturation ponds fitted with flow baffles to prevent short-circuiting and ensure plug flow. The pond is normally divided into a number of channels in multiple forward and return passes, to ensure a reasonable forward velocity and prevent back-mixing.

Although maturation rivers have been installed as tertiary treatment at a number of mechanical plants, their success has been variable due to accumulation of solids, particularly in the first pass. For this reason it is recommended that they only be used as the final pond in a series of two or more, whether preceded by a mechanical plant or by a pond system.

Maturation rivers are suitable for bacterial die-off, but their main advantage is that the absence of short-circuiting and back-mixing, and maintenance of plug flow conditions, eliminates the re-inoculation of incoming water with algae and the final effluent from the system therefore has a low algal count and a relatively low suspended solids.

5.4.2 DESIGN

For similar reasons as those pertaining to ponds, maturation rivers should not be less than 1 m deep and the normal recommendations of a 1 to 1.5 m depth applies. The shape of the pond should suit the land and a limited number of long passes is equally acceptable as a larger number of short passes. For cost reasons the width of the channel is important as narrow channels would imply a large number of dividing walls, and very wide channels would allow back mixing.

Design criteria are not that well established but most maturation rivers constructed to date have had approximately a 2 day (48 hour) retention (longer retentions involve higher cost and may give algae long enough to increase in numbers), with a channel forward velocity of 1 to 2 mm/s (3.6 to 7.2 m/h).

The dividing walls need not be watertight or load-bearing. Prefabricated concrete fencing has been used successfully.

5.5 AERATED LAGOONS

5.5.1 INTRODUCTION

Aerated lagoons are mechanically aerated wastewater treatment ponds. These are completely mixed wastewater treatment ponds utilising either surface-type aerators, submerged propeller, or turbine-type aerators. The principal source of oxygen is furnished by mechanical aeration rather than by photosynthesis. The purifying organisms which develop in an aerated lagoon are typical of an activated sludge process rather than a naturally oxygenated pond. For this reason some literature classes aerated lagoons as an activated sludge variant rather than a pond variant.

The solids carry-over from the aerated lagoon must be removed by a clarification process following its treatment in the aerated pond. This is usually achieved by passing the effluent into one or more maturation ponds. As the system approximates to a completely mixed system the concentration of suspended solids in the lagoon effluent is approximately equal to that of the bulk contents of the lagoon.

The advantage of using an aerated lagoon over a natural primary pond is that a much smaller primary pond unit will be required. A primary facultative pond in South Africa typically has an effective oxygenation capacity of 120 to 180 kg/d.ha. The same amount of oxygen could be supplied by a mechanical surface aerator of about 7 kW. The size of the primary pond would then be governed by COD or BOD removal kinetics rather than natural oxygen dissolution rates.

There are still many natural primary facultative ponds in existence as these have no power requirements and no need for mechanical maintenance. They are also aesthetically pleasing in that the ponds are an attraction for bird life. A pond system is an attractive treatment option in a semi-rural environment. However it may be worth considering an aerated lagoon as a primary pond particularly if sufficient suitable land is unavailable or too expensive for a naturally oxygenated facultative pond.

5.5.2 DESIGN PARAMETERS

5.5.2.1 Retention

Aerated lagoons can treat either raw wastewater [usually after preliminary treatment (Ch 3)], or settled wastewater e.g. anaerobic pond effluent. BOD removals above 90% are achievable at retention times of 2.5 to 7 days. Retention times of less than about 2.5 days do not allow sufficient time for full development of the purifying activated sludge type organisms.

The upper limitation on sizing these ponds will be the ability to maintain aerobic conditions and to maintain the effectiveness of the microorganisms by mixing. Large ponds would tend to require an excessive number of aerators. The aerated aerobic ponds should be designed to keep active biological solids in suspension and maintain a minimum dissolved oxygen (DO) concentration of 2.0 mg/ℓ.

5.5.2.2 Mixing

Mechanical aerators can be designed to provide either complete mixing of solids, including oxygen dispersion, or just to provide uniformly dispersed oxygen. In the latter case some solids deposition will occur in the basin.

The power level required to maintain solids in suspension is about 4 to 6 W/m^3, and to disperse oxygen uniformly throughout the basin is about 1.5 to 3 W/m^3 respectively. A mixing velocity (average velocity of any given particle in the pond) greater than 0.2 m/s should be maintained in the basin to prevent solids deposition.

Mixing energy input varies with the size of the aeration unit. **Table 5.5.1** below gives an approximation as to the mixing energy for oxygen dispersion from various sized aerators

TABLE 5.5.1: Estimation of mixing energy for oxygen dispersion from various sized aerators.

Size of Aerators (kW)	Mixing Energy (W/m^3)
75	3
37	3.6
15	4.2

It can be seen that mixing energy improves with decreasing size of aerator. From a process point of view one would therefore prefer a larger number of small aerators to a lesser number of large aerators. This would also enable the aerators to cover the lagoon area better. Large aerators also require deeper ponds. The final selection would be based on the economics of land area required versus increased cost of mechanical equipment.

5.5.2.3 Oxygenation Capacity

The total aeration capacity required will depend on the total COD or BOD load based either on population per capita contribution or on sewage volume and strength. Due to the large volume of the pond relative to daily flow no large peak oxygenation factors are required but the aeration efficiency of small floating aerators is lower than those used for activated sludge plants. An installed aerator power of 1.0 kW per kg COD.h is suggested (1 kW/24 kg COD.d or 2 kW/24 kg BOD.d). This should then be checked against the mixing energy criteria above and increased, if necessary.

5.5.3 BOD REMOVAL EFFICIENCY FOR SIZING OF POND

The BOD removal efficiency in an aerated pond is a function of temperature and detention time in the basin. **Table 5.5.2** below lists the calculated efficiencies for combinations of different temperatures and detention times (based on an empirical equation).

Table 5.5.2 can be used as a guideline and selection should be based on the average winter temperature for the coldest month and the number of maturation ponds planned to follow the aerated pond. In no case should there be less than two following cells and the more ponds the greater the bacterial die-off. The design sequence for bacterial die-off in maturation ponds is presented in Section 5.3. From BOD removal kinetics for a system of an aerated lagoon and two maturation ponds for a raw sewage BOD of 300 mg/ℓ, one would get to an effluent BOD of 15 mg/ℓ with a 95% overall removal which is probably about where purification would be adequate. Lower effluent standards may be applicable in the case of irrigation of certain crops.

TABLE 5.5.2: Calculated efficiencies at different temperatures and detention times

Temperature (°C)	Percent BOD Removal for Various Detention Times (Days)					
	2	**3**	**4**	**6**	**8**	**10**
4	12	16	21	28	35	40
6	15	20	25	34	40	46
8	18	25	30	40	47	52
10	22	30	36	46	53	59
12	27	35	42	52	59	64
14	32	41	48	58	65	70
16	38	47	55	64	71	75
18	44	54	61	70	76	80
20	50	60	67	75	80	85
22	53	63	69	77	83	85
24	55	65	71	79	83	86
26	58	67	73	79	85	87
28	61	70	75	82	86	88
30	63	72	77	84	87	90
32	66	74	79	85	88	90
34	68	76	81	86	89	91
36	70	78	82	88	90	92
38	72	80	84	89	91	93

5.6 ARTIFICIAL WETLANDS AND REEDBEDS

5.6.1 INTRODUCTION

The potential of artificial or constructed wetlands as a reliable and fundamental process for the secondary treatment of wastewater and for nutrient removal has received considerable attention during the past twenty years. The discharge of wastewaters into constructed wetlands may be considered a viable alternative treatment option, particularly suited to small and medium sized communities in sparsely populated and developing areas. It is generally accepted by researchers that wetland systems have considerable potential and may offer a number of advantages compared to conventional wastewater treatment options:

1. Low operating cost
2. Low energy requirements
3. Low maintenance requirements
4. Can be established close to the site of wastewater production
5. Can be established by relatively untrained personnel
6. Robust process able to withstand a wide range of operating conditions
7. Environmentally acceptable offering considerable wildlife conservation opportunity
8. Can readily be integrated into existing forms of effluent treatment

5.6.2 DESCRIPTION OF BASIC LAYOUT

A constructed wetland consists of a shallow, often lined excavation, (depending upon acceptance of seepage to the ground system) containing a bed of porous soil, gravel or ash, in which emergent aquatic vegetation is planted (commonly *Phragmites Australis*).

The depth of the bed is generally 0.3 to 1.0 m deep and is constructed with a peripheral embankment at least 0.5 m high above the bed to contain storm conditions and accumulation of vegetation and influent solids.

Earlier beds were usually constructed at an incline of 2 to 8% where a soil media was used to ensure adequate hydraulic gradient to encourage passage of the effluent through the bed. Gravel and ash beds may, however, be constructed essentially level as long as the length to width ratio is adequate, in relation to the influent flow.

The selection of the permeable media represents the dominant factor in ensuring the desired hydraulic path, surface or subsurface, and consequently the treatment efficiency and reliability. It also forms the dominant cost factor for the wetland, particularly if a gravel or ash is required and has to be imported to the site. However, it has been identified that it is rarely possible to compensate for poor permeability by incorporating an incline. An incline can promote surface ponding and short-circuiting thereby diminishing treatment efficiency.

Many units have been constructed on the 'root-zone' principles developed in West Germany whereby the wastewater passes horizontally through the media in which the plants are established. Other systems incorporate surface flow, comparable to natural wetlands, or vertical flow (upflow and downflow) through the media, or combinations of individual wetland units to optimise the removal of pollutants (modularisation of systems).

Practical application rates for the treatment of municipal wastewaters range from 40 to 200 m^3/ ha.d for hydraulic loading and 30 to 400 kg BOD/ha.d for organic load.

FIGURE 5.6.1: Artificial reedbed under construction

FIGURE 5.6.2: The same artificial reedbed two years later

5.6.3 TREATMENT MECHANISMS

There is still a great deal to be learnt and understood concerning the very complex reactions taking place within an artificial wetland system. Design criteria are at this stage very tentative and the required effluent quality objectives may not be met at the design loading rates. Experience overseas in this regard has been most erratic.

Despite the imponderables, artificial wetland systems nevertheless have a place in sewage treatment technology. They are not suitable for treating raw sewage, but can be used successfully to treat raw sewage that has either passed through an anaerobic pond system or some form of primary treatment i.e. with septic tanks. The construction should whenever possible be appropriate making maximum use of local materials and resources.

Factors not taken into account in the design of these pond and wetland systems are the water losses which will occur to the main soil body. These losses may be very significant where the ponds are constructed over semi-permeable materials of very low natural moisture content. Initial filling of the systems may take many months or even years in saturating the local soil body. In European practice use has been made of geomembranes to line the pond systems. In the local situation this would become prohibitively expensive and could not be justified unless the effluent has a high economic value.

Although the aquatic plants are the most obvious biological component of the wetland ecosystem, the actual purification is accomplished through a combination of biological, physical and chemical interactions between the plants, the media and the microbiological community. The primary role of the plants is to provide surfaces for bacterial growth, the filtration of solids, the translocation of oxygen to the rhizosphere and improvement of the soil's permeability.

In addition to biological processes, wetland systems are capable of removing significant amounts of pollutants by physico-chemical mechanisms. Ion exchange and precipitation processes within soil and ash media will result in a substantial reduction in certain ions. For example, co-precipitation of phosphate with iron, aluminium and calcium can dramatically reduce phosphate levels. Heavy metals may be precipitated with sulphide in zones where sulphate reduction occurs and the formation of water-insoluble organo-metallic compounds may also represent an active mechanism for pollutant removal.

5.6.4 COD (AND BOD) REMOVAL

The COD (BOD) associated with settleable solids in wastewater is removed by sedimentation. The colloidal and soluble COD remaining in solution is removed as a result of the metabolic activity of micro-organisms that are suspended in the water column, attached to the sediments and to the roots and stems of the aquatic plants. Conceptually, wetland systems are slow-rate, horizontal-flow trickling-filters with built-in secondary clarification in which plants supplement rocks or other media as bacterial support structure, as well as transmitting oxygen into the wastewater system.

The rate of biodegradation of various organic substances depends on temperature, oxygen concentration (gradients and profiles), pH, nutrient availability, substrate concentration and the presence of potential toxins. The character of any organic compound will also affect its biodegradation rate so that within the readily biodegradable fraction, rates of degradation vary widely.

The most readily-biodegradable substances such as protein, sugar and starch are metabolised very rapidly, while fat, wax, cellulose, lignins, phenols and cyanides decompose more slowly, and sometimes incompletely. As a result, the time taken for wastewater to flow through a wetland determines which organic compounds are present at their outflow. While the readily biodegradable portions of wastewaters are decomposed rapidly, less readily biodegradable fractions may be incorporated into the organic sediment to be anaerobically degraded over an extended period extending the overall retention time available for pollutant removal by microorganism degradation.

The removal of BOD, TOC and COD on both natural and artificial wetlands and on overland flow systems can most easily be described as a first order function of the form:

$$C_t = C_0 e^{-kt} \qquad \text{……………………………………………………………(5.16)}$$

Where:

C_t = concentration remaining at the time t, (mg/ℓ)

C_0 = concentration at time t = 0 (mg/ℓ)

k = specific removal rate constant for given constituent at 20 °C (d^{-1})

t = detention time in wetland, (d)

Such a relationship may also apply to the removal of pathogenic microorganisms, certain trace organics, and heavy metals.

If it is assumed that 95% of the BOD in primary wastewater is removed in 10 days, the value of k (which is temperature dependent) is of the order of 0.3 d^{-1}, according to North American conditions. This requires re-evaluation under long term Southern African conditions, but may be assumed, in general, to be lower as a consequence of a more conducive climatic environment.

The effect of temperature can be modelled with sufficient accuracy using the following expression.

$$k_t = k_{20} \theta^{(T-20)} \qquad \text{…………………………………………………………..(5.17)}$$

where

k_t = removal rate constant at temperature T (d^{-1})

k_{20} = removal rate constant at 20 °C (d^{-1})

θ = temperature coefficient, 1.05 to 1.08

T = temperature, (°C) of the water in the wetland

The significance of the above equation for cold regions is that the area of most wetlands must be increased by a factor of two or more during the winter to achieve the same level of treatment. As it is assumed that the bacteria attached to the plant stems and humus are responsible for treatment, the fact that the wetland plants may be dormant or die in the winter is of little concern with respect to BOD removal unless the physical plant support structure is lost.

5.6.5 SOLIDS REMOVAL

Wetland systems have long hydraulic residence times. Consequently, virtually all settleable and floatable solids of wastewater origins are removed. Colloidal solids are removed, at least either by bacterial growth (which results in the settling of some colloidal solids and the microbial decay of others) or collisions (inertial and Brownian) which result in adsorption to other solids (plants, pond bottom and suspended solids).

The annual build-up of solids within the system has been observed to be very low, reflecting efficient mineralisation and degradation mechanisms. The inlet areas, however, may require additional attention where high loadings per unit area are evident. In horizontal systems, solids removal occurs predominantly in the initial 12 to 20% of the wetland area. The remaining area may be important for the removal of solids which are produced by the system, including those precipitated as insoluble salts.

The root structure of plants such as *Phragmites* also provides infiltration pathways through the upper layers of the bed thereby ensuring that the surface of the filter bed does not clog up.

5.6.6 NITROGEN

There are two pathways by which wetlands can be used for nitrogen removal from wastewater:

1. Nitrogen can be stored (assimilated or adsorbed) in the system.
2. Nitrogen can be removed from the system through denitrification and ammonia volatilisation.

Leaching and runoff can also remove nitrogen from a wetland but these are not desirable pathways for wastewater treatment.

Storage of nitrogen in a wetland is a temporary measure in terms of wastewater treatment. Regardless of whether nitrogen is assimilated into biomass or absorbed onto the soil, the maximum capacity of the system will be reached sooner or later. When nitrogen is assimilated into biomass it is frequently returned to the sediments through biomass senescence and die-off.

Harvesting plants to remove nitrogen is inefficient. Herskowitz (1988) reported harvested material to account for less than 10% of nitrogen removed whilst Gersberg et al. (1985) reported harvesting to account for 12% to 16%. Assuming favourable conditions, the maximum nitrogen removal rate of this mechanism is approximately 6 kg/ha.d. To achieve this, approximately 2.25 t (dry weight) of plant biomass would have to be harvested per hectare per day, representing a significant solid waste handling and disposal problem (Stowell et al., 1981).

A neutral pH is usually maintained in artificial wetlands. Ammonia volatilisation is therefore not likely to be a significant pathway for nitrogen removal under normal circumstances Volatilisation of ammonia may occur.if the pH is raised to 8 or more as a result of CO_2 uptake by submerged plants during photosynthesis,. The nitrogen removal potential of this mechanism is seasonal and inconsistent and can best be estimated for a particular locale by determining the nitrogen loss from local stabilisation pond systems. Volatilisation of NH_3 can result in nitrogen removal rates as high as 22 kg/ha.d. A concomitant treatment benefit of removing nitrogen by NH_3 volatilisation is that insoluble metal phosphate precipitates are also formed at elevated pH.

Biological nitrification and denitrification are the key processes for long-term nitrogen removal from a wetland. These are two sequential processes which co-exist in many ecosystems where heterogeneous aerobic, anoxic and anaerobic microsites exist. Under conditions where dissolved organic carbon is not limiting, (required for denitrification) the factor most limiting nitrogen removal is the supply of oxygen which is necessary to sustain nitrification. In this regard, the ability of an aquatic plant to translocate O_2 from the shoot to the root, and thereby establish an oxidised rhizosphere is an important factor.

The nitrification potential is also limited by water temperature and BOD fluxes, which cause alternative demands to be made on that oxygen. A simplistic (and thus somewhat inaccurate) description of this interaction is that any oxygen flux in excess of the BOD flux will be used to nitrify ammonia if the water is sufficiently warm (nitrification virtually ceases at water temperatures below 10° C).

A guideline for summer nitrogen removal is 9 to 11 kg/ha.d, although figures as high as 44 and 54 kg/ha.d have been reported. Winter nitrogen removals (or releases) will be site-specific and a function of climate and plant species.

In a new wetland, optimum nitrification and ammonia reduction reactions will not occur until optimum oxygen transfer is occurring. This will be when the vegetation has 'matured' (i.e. growing densely above the surface and forming a dense root/rhizome system within the substrate). The vegetation usually takes about two to four years to reach this stage depending on the initial plant density. A new system cannot be expected to meet stringent ammonia or nitrogen limits until the vegetation has ‘matured’, unless alternative methods are employed to enhance the nitrification reaction. The addition of oxygen to the wastewater either by mechanical means or by changing the configuration of the system is such an alternative.

5.6.7 PHOSPHORUS REMOVAL

Phosphorus removal in a wetland is achieved through plant uptake and the biological and physico-chemical storage of phosphorus in the sediments. These latter mechanisms are the more significant in most wetland systems and must be properly considered in designing the wetland.

5.6.7.1 Plant Uptake

The initial removal of dissolved inorganic phosphorus from water under natural loading levels is due largely to microbial and aquatic uptake and the geochemical adsorption by aluminium and iron minerals in the soil. The microbial pool is small and quickly becomes saturated even where luxury phosphate can be accomplished under aerobic conditions by species such as *Acinetobacter*.

Aquatic plants supplied with sewage effluent tend to show increased growth, and usually have increased tissue P concentrations. As a result, they have the potential to remove significant quantities of P from wastewater under low-loaded conditions. Uptake values vary considerably between plant species, and the actual amount taken up in any wetland will depend on a number of factors. These include standing crop of the plants, age of the plants and nutrient status of the water. For the principal wetland system plants, P removal may be expected to be of the order of 60 to 180 kg/ha per annum as plant assimilation.

5.6.7.2 Biological and Physico-Chemical Storage

Algal communities associated with the surface waters may also remove considerable quantities of P from wastewater though the growth is erratic and removal of algae from the water phase represents a significant problem. P absorbed in rooted emergent vegetation and algae is released back into the water body and sediments after tissue death. Richardson (1985) suggests between 35 - 75% of the plant phosphorus is rapidly released on plant senescence. Thus, vegetation only serves as a short term sink for phosphorus unless the biomass is harvested.

5.6.7.3 Chemical Binding Reactions

In utilising soil-based wetland systems for effluent treatment it has been observed that soils have the capacity to regain, and in some cases enhance, their adsorptive capacities after apparently having reached saturation and subsequently been allowed to rest. This characteristic has an important bearing on the system design and operation for phosphate removal from wastewaters. The explanation behind the phenomenon is related to the slow dissolution of aluminium and iron compounds to create new sites for adsorption of phosphate with time.

Average soils can be expected to contain from 2 to 4% iron and from 5 to 7.5% aluminium. Thus if all iron and aluminium were dissolved and converted to iron and aluminium phosphates, nearly 1 000 tons of P could be precipitated per hectare (1 hectare area x 1 m deep). However, under normal conditions, i.e.

near neutral pH, and moderate temperatures, iron and aluminium are both slow to dissolve, and only a fraction of the possible capacity of the soil to precipitate P will be realised in the short term.

5.6.8 PATHOGEN REMOVAL

There is little information available on the long term fate of biological indicators of pollution such as total coliform bacteria in wetlands although it is recognised that wetlands offer a unique combination of physical, chemical and biological factors which contribute to inactivation and removal of both pathogenic viruses and bacteria. In addition to filtration through the media and the attached biofilm, physical removal factors include sedimentation, aggregation and inactivation by ultraviolet radiation (UV). Chemical factors include oxidation, exposure to biocides which may be excreted by plants, and adsorption to organic matter and the biofilm. Biological removal mechanisms include antibiosis, ingestion by nematodes or ciliates, attack by lytic bacteria (or viruses), and natural die-off.

5.6.9 DESIGN OF ARTIFICIAL WETLANDS

The criteria presented are for the application of primary or secondary effluent. (As discussed previously treatment of raw sewage is not advisable). The corresponding organic loading rates for a given wastewater can be derived from the hydraulic loading rates. Where primary effluent is applied it was assumed that the removal of suspended solids (SS) and BOD or COD are of principal concern. Where secondary effluent is applied it was assumed that nitrogen control is of prime concern, although some phosphorus would be removed for this particular data.

Kickuth (1983) has calculated that the wetland system design can simply be created from knowledge of the BOD loading where the area of a bed (A) is derived from the equation:

$$A = K.Q_d(\text{In}C_o - \text{In}C_t)$$

Where:

A = plan area of reed bed (m^2)

K = constant = 5.2

Q_d = the average flow rate of sewage (m^3/d)

C_o = the average BOD of the influent (mg/ℓ)

C_t = the average BOD of the effluent (mg/ℓ)

The value of K = 5.2 relates to the removal of BOD from sewage in a bed which is to be 0.6 m deep and operated at a minimum temperature of 8°C.

For wastewaters that are more difficult to biodegrade and for lower temperatures the value of K will be greater, up to 14 or 15.

For domestic sewage this tends to produce an area of approximately 2.2 m^2/p.e. This seems to be optimistic, however, and many European systems are designed between 3 to 5 m^2/p.e, which implies a K value of about 11, or are built in low-risk situations where the effluent quality is not critical. In practice, the wetland sizing should be determined as a consideration of both the effluent organic and volumetric loads, rather than a person equivalent basis. It is recommended that a COD loading of approximately 270 kg COD/ha.d should represent a conservative basis for evaluating the wetland basis of design.

5.6.10 PRETREATMENT

Pretreatment of wastewater before it enters the system is advantageous, but it does not necessarily mean that the reed-bed can be reduced in size according to COD removal in the pretreatment stage. Raw sewage should not be loaded directly onto a reed-bed where odour and blockage problems may result.

1. Pretreatment may involve any combination of the following:
2. Screening
3. Septic tanks
4. Localised digesters
5. Anaerobic lagoons
6. Facultative and aerated ponds
7. Conventional secondary treatment processes.

In the case of agricultural, industrial, and mine effluents specific attention should be paid to pretreatment requirements such as toxic material removal, pH correction (to an extent if required) and quality balancing (recycle), and ease of startup.

5.6.11 WETLAND APPURTENANCES AND OTHER PHYSICAL FEATURES

5.6.11.1 Slope

In practice systems have been constructed with gradients of up to 8% although experience in Europe has demonstrated that it is rarely possible to compensate for inadequate permeability by incorporating a slope into the design. Additionally a sloping surface causes further problems e.g. surface ponding of effluent, odour generation and erosion, uneven growth of reeds and most importantly poor effluent quality and difficulty in controlling competing weed growth by the preferred procedure of periodically flooding the bed.

Systems in Europe are being designed with the minimum slope needed to allow the water to pass through the bed (calculated from the D'Arcy's Law equation) and to use a level surface to permit complete flooding for weed control, with hydraulic gradient control via adjustment of the level at which the final effluent is drawn from the wetland profile

5.6.11.2 Bed Depth

The depth of the artificial wetland system is determined by the economics of media availability and construction rather than the expected long term depth penetration of the plant root system, which may be greater than 1.5 m. The bed depths are typically 0.6 m, although depths of between 0.3 and 1.5 m have been used creating a shallow, horizontal filter arrangement. It has been accepted in most of the UK and European systems that a depth of approximately 0.6 m is desirable. This is based on:

1. The statement that beyond 0.6 m root growth starts to weaken
2. Thinner beds may suffer from freezing during extended winter-periods.

5.6.11.3 Feed Distribution

The incoming wastewater needs to be distributed in the inlet zone into the main media throughout its depth. This is particularly important to reduce the potential for short-circuiting, ponding, stagnation in dead zones and surface flow during the initial establishment period, whilst reed rhizome structure is growing.
For the horizontal subsurface flow systems the feed should be introduced across the whole width of the influent area. In order to improve the distribution an inlet zone filled with 60 - 100 mm stones 1 to 2 m wide can be constructed.

Where horizontal flow systems are to be operated in series an efficient distribution system at the inlet of each cell must be constructed. Simple openings or weirs in the divider dykes are unlikely to achieve satisfactory flow regimes. In some of the more recently built systems in the UK and the USA, a simple pipe with a series of adjustable T-pieces has been used, to replace expensive castellated weirs.

Final effluent from subsurface flow systems can be collected into an outlet trench and flow out via a pipe, the discharge level of which should be capable of variation. During normal operation, the level of water would be maintained about 10 to 50 mm below the top of the bed to minimise potential for surface ponding and odour generation/release, and encourage mineralisation and composting of accumulating plant and bacterial humus. Periodically the level will be raised to flood the bed for weed control purposes.

In the case of vertical flow systems the influent can be introduced from central channels to the surface of the bed, using herring-bone agricultural piping over the surface, spray systems or contoured channels crisscrossing the surface to approximate even distribution across the whole area. The alternative is to create a single inlet zone similar in form to that of the horizontal flow systems and allow a layer of liquor to remain on the surface (5 - 10 cm). This will effectively form a combination of the horizontal and vertical flow systems, since a proportion of the inlet area will be performing the majority of the treatment (particularly filtering action) as effluent percolates through the system.

5.6.11.4 Baffles

Baffles may be required to control distribution of surface flow. Baffles may be created from stone/gravel, hay bales, loop or planks to improve flow distribution and penetration.

5.6.11.5 Embankments and Verges

The embankments should provide for at least 500 mm above the media level to accommodate sludge and straw accumulation before the top layer of composted sludge (peat) needs to be removed.

The slopes of embankments would be dictated by normal engineering practice for small dams. Depending on the construction material and compaction, embankment slopes should be at least 1:2 (usually 1:3). Fringes may be designed to prevent ingress of vegetation with stone pitching, soil-cement or grassing.

CHAPTER 6

6. SEDIMENTATION

6.1 INTRODUCTION

Sedimentation is fundamental to efficient sewage treatment practice. It is not confined to any one method of treatment but is common to all and, by the use of gravity, is the least expensive method of separating suspended matter from the liquid. In principle it consists of reducing the velocity of the flowing liquid sufficiently to allow suspended particles of higher specific gravity to separate from the main body of the liquid and settle on the floor of the tank as a sludge which may then be removed.

A distinction is made between the removal of heavy inorganic solids such as grit, which is accomplished at the head of works in grit channels and detritors, and the settling of lighter solids, mostly organic, as sludges in primary or secondary sedimentation tanks.

Primary sedimentation tanks remove the grosser suspended solids from the raw sewage, normally reducing the suspended solids (SS) by between 40% and 60%, and the oxygen demand (BOD or COD) by 30% to 35%.

Secondary sedimentation tanks (humus tanks) remove the excess bacterial slimes which are sloughed off the media in biological filters, rotating biological contactors (RBC's), and submerged media rectors.

Clarifiers settle the activated sludge out of the mixed liquor from an activated sludge aeration tank and return the sludge to the aeration tank.

The roles of humus tanks and clarifiers are quite different. The former operates at higher surface loadings and is associated with a low mass of sludge solids. The latter operates at relatively low surface loadings and handles high masses of sludge solids so that solids flux becomes relevant.

6.2 PROCESS CONSIDERATIONS

A sedimentation tank is defined as a tank in which sewage or treated effluent is retained long enough to bring about sedimentation of suspended matter but short enough to prevent anaerobic decomposition.

The fine organic material in sewage or treated effluent is susceptible to a degree of flocculation which results in the formation of flocs with settling velocities which permit their removal under quasi-quiescent conditions. The rate of flocculation is a function of both the suspended solids concentration and the action of complex physical, physicochemical and biological processes.

Other factors which have an influence on the efficiency of the sedimentation process are temperature, and accompanying thermal currents, the characteristics of the liquid and solids, and the physical characteristics of the sedimentation tank design.

It is essential that the physical characteristics of a sedimentation tank ensure that short-circuiting or dispersion (the phenomenon of unequal times of passage for different portions of the same stream entering continuous flow tanks) is reduced to a minimum. Short-circuiting in primary tanks results in poor settling efficiency. This in turn places a heavier burden on succeeding treatment processes, such as biological filters or activated sludge units, and thus can cause operational problems. In secondary tanks it leads to unacceptably high suspended solids contents in the effluents, and loss of solids from activated sludge processes.

Short-circuiting can be minimised by quiescent settling which in turn can be enhanced by the provision of a stilling box at the inlet to a tank. The latter, as the name implies, should provide for efficient dissipation of the kinetic energy of the incoming liquid.

6.3 GENERAL CONSIDERATIONS FOR DESIGN

6.3.1 BASIC GUIDE

Multiple units capable of independent operation should be provided for plants having an average design capacity greater than 1 Mℓ/d unless temporary removal of a single unit from service for repairs will not result in an adverse effect to the quality of the receiving stream. Clarifiers should be arranged to facilitate operating flexibility and maintenance, assure continuity of treatment, and ease of installation of future units.

Provision should be made for dewatering and bypassing each unit independently. The bypass should provide for redistribution of the wastewater to an appropriate point in the remaining process units. Due consideration should be given to the possible need for hydrostatic pressure relief devices to prevent structure flotation especially in areas of high water table.

Flow distribution arrangements should be provided such as to ensure that multiple tanks receive flow in proportion to their design capacities. The anticipated flow pattern should be considered in the selection of clarifier configurations and location and type of inlets and outlets. In some of the more difficult applications the use of computerised fluid dynamics (CFD) analysis may be beneficial.

6.3.2 DESIGN CONSIDERATIONS

6.3.2.1 Performance

Unless laboratory and/or pilot plant data are available, primary settling should be assumed to remove about one-third (30% to 35%) of the influent COD and between 50% and 60% of the influent suspended solids.

It is not recommended to return waste activated sludge (WAS) to the primary sedimentation tanks (PST's) as these are designed at more than double the surface loading of activated sludge clarifiers. The end result is that the WAS overflows the PST and goes back to the aeration tank, thus going around in circles. This sludge should be wasted to a separate thickener unit.

6.3.2.2 Inlets

Inlets should be designed to dissipate the inlet velocity, to distribute the flow uniformly and to prevent short-circuiting. Provisions should be made for removal of floating materials in inlet structures having submerged ports.

6.3.2.3 Dimensions

The minimum distance from the influent inlet to effluent weirs should be at least 3 m for all tank configurations unless special provisions are made to prevent short-circuiting. The side-water depth (SWD) of mechanically cleaned tanks should not be less than 2 m for primary clarifiers and those following fixed film reactors. Final activated sludge clarifiers should have side-water depths of at least 3 to 3.5 m. If depth limitations due to ground conditions are less than the minimum recommended, the overflow rate should be reduced by 0.15 m/h. for each 300 mm of SWD under that recommended for clarifiers. It is preferable that the sludge collector scraping mechanism be at least 2 m below the water level.

6.3.2.4 Channels

Inlet channels should be designed to maintain a velocity of at least 0.3 m/s at one-half design flow. Where minimum velocities are less, provisions should be made for the re-suspending of the solids. Corner fillets or channelling should be provided to eliminate corner pockets and dead ends. The width of effluent channels should be at least 300 mm. The bottom of the channels at the outlet structures should be at least equal to, or above, the water levels of the downstream treatment units. The effluent channel and discharge pipe-work should be sized to prevent weir submergence at the peak hourly flow. The bottom of effluent channels should be at least 300 mm below water levels maintained in the clarifiers, except for small package plants.

6.3.2.5 Baffling and Scum Removal

Scum baffles should be provided around all PST's with a flow capacity of greater than 1 Mℓ/d. The baffles should be located at the water surface to intercept all floating materials and scum. Baffles should extend at least 75 mm above the weir plate elevation and 300 mm below the water surface. Stilling chamber diameters should not be less than 20% of the tank diameter and should have a depth of 55 to 65% of the SWD. The maximum inlet velocity to a centre inlet well should not exceed 1 m/s. The outflow velocity should not exceed 4.5m per minute.

6.3.2.6 Submerged Surfaces

The topside of beams, troughs or similar construction components shall have a minimum slope of 1.4 vertical to 1 horizontal; the underside of such components shall have a slope of 1-to-1 to prevent the accumulation of scum and solids.

6.3.2.7 Weirs

Weir plates shall be adjustable for levelling and sealed against the effluent channel. Weirs should be located to optimise actual hydraulic detention time and minimise short circuiting. Circular tanks, with centre-feed inlets, should be provided with a full weir. Weirs should be of the saw-tooth or serpentine type to allow for better weir overflow and flow distribution.

6.3.2.8 Weir Loadings

Hydraulic loading criteria in tank design should be based on the sum of plant inflow and recycle rates

Weir loadings should not exceed 7.5 m^3/h per metre weir length for primary, intermediate or final sedimentation, at peak hourly flows on small plants (less than 500 kℓ/d) unless select design parameters are considered; (such as, depth, surface area, detention time, horizontal and/or vertical velocities, solids density) when higher weir loadings may be applied. Weir loadings at peak hourly flows in intermediate sized plants (with average design flows typically 0.5 to 5 Mℓ/d) should not exceed 10 m^3/h.m of weir length for primary and intermediate clarifiers, and on larger plants should not exceed 15 m^3/h.m of weir length. The weir loading on activated sludge clarifiers should not exceed 10 m^3/h.m of weir length.

6.3.3 DESIGN OF PRIMARY SEDIMENTATION TANKS

Sedimentation tanks are generally circular in plan unless this is precluded by site constraints. Rectangular tanks require special care in the arrangement of inlets, outlets and sludge removal, and weir loadings are often a limiting factor

FIGURE 6.3.1: Typical primary settling tank (courtesy eWISA)

The following criteria should be adhered to when designing primary sedimentation tanks:

Primary Tanks:

Dortmund type tanks	**Parameters**
Slop of cone	Not less than 60° to horizontal
Depth of cylindrical section	Usually 15% of tank diameter, but not less than 0.6 m
Diameter of stilling box	To give a downward velocity in the box at PDWF of 40 to 50 m/h
Level of stilling box below level of cylindrical section of tank	To be as steep as possible provided the upward velocity through the horizontal annulus at PDWF is not more than 5 m/h
Depth of scum board below TWL	300 to 400 mm
Distance between scum board and overflow weir	Maximum of 300 mm
Hydraulic head of sludge draw-off	Minimum of 2 m
Diameter of sludge lines	150 to 200 mm
Retention period of PDWF	Minimum of 1.5h
Upward velocity at ADWF	Maximum of 1.2 m/h*
Upward velocity at PDWF	Maximum of 2.4m/h*

** Use whichever criterion gives the largest diameter tank*

Note: Re-circulated effluent should preferably not pass through primary sedimentation tanks, but if provided for in the design, should be allowed for in calculating the upward velocity.

Scraped Rotating Bridge type tanks	**Parameters**
Depth of stilling box below TWL	Not less than 65% side wall depth
Depth of scum board below TWL	
i. without mechanical skimmer	Minimum of 500 mm
ii with mechanical skimmer	Minimum of 300 mm
Distance between scum board and overflow weir	Maximum of 300 mm
Overflow weir loading at PDWF	Typically 7.5 m^3/h per m (small plant)
Retention period of PDWF	Minimum of 1.5h
Upward velocity at ADWF	Maximum of 1.2 m/h*
Upward velocity at PDWF	Maximum of 2.4m/h*

** Use whichever criterion gives the largest diameter tank*

Note: Re-circulated effluent should preferably not pass through primary sedimentation tanks, but if provided for in the design, should be allowed for in calculating the upward velocity.

6.3.4 DESIGN OF SECONDARY SEDIMENTATION (HUMUS) TANKS

Sedimentation tanks are generally circular in plan unless this is precluded by site constraints. Rectangular tanks require special care in the arrangement of inlets, outlets and sludge removal.

FIGURE 6.3.2: Typical secondary sedimentation tanks

The following criteria should be adhered to when designing secondary sedimentation tanks:

Dortmund type tanks	**Parameters**
Slop of cone	Not less than 60° to horizontal
Depth of cylindrical section	Usually 15% of tank diameter, but not less than 0.6 m
Diameter of stilling box	To give a downward velocity in the box at PDWF of 40 to 50 m/h
Level of stilling box below level of cylindrical section of tank	To be as steep as possible provided the upward velocity through the horizontal annulus at PDWF is not more than 5 m/h
Depth of scum board below TWL	300 to 400 mm
Distance between scum board and overflow weir	Maximum of 300 mm
Hydraulic head of humus draw-off	Minimum of 1 m
Diameter of sludge lines	150 to 200 mm
Retention period of PDWF	Minimum of 1.5h
Upward velocity at ADWF	Maximum of 1.0 m/h*
Upward velocity at PWDF	Maximum of 1.5m/h*

** Use whichever criterion gives the largest diameter tank*

Note: Re-circulated effluent should preferably not pass through primary sedimentation tanks, but if provided for in the design, should be allowed for in calculating the upward velocity.

Scraped Rotating Bridge type tanks	**Parameters**
Depth of stilling box below TWL	Not less than 65% side wall depth
Depth of scum board below TWL	
i. without mechanical skimmer	Minimum of 500 mm
ii with mechanical skimmer	Minimum of 300 mm
Distance between scum board and overflow weir	Maximum of 300 mm
Overflow weir loading and PDWF	Typically 7.5 m^3/h per m (small plant)
Retention period of PDWF plus recirculated flow	Minimum of 1.5h
Upward velocity at ADWF	Maximum of 1.0 m/h*
Upward velocity at PDWF	Maximum of 1.5m/h*

** Use whichever criterion gives the largest diameter tank*

Note: Re-circulated effluent, if provided for in the design, should be allowed for in calculating the upward velocity.

6.3.5 DESIGN OF ACTIVATED SLUDGE CLARIFIERS.

The design of clarifiers follows the same principles as for Humus Tanks except that the limiting surface loadings are lower (maximum 1.0 m/h at peak flow), and the solids flux on the floor of the tank may become limiting (not more than about 7 $kg/m^2.d$). This is described in greater detail below.

6.3.5.1 Detention Time

Nominal detention periods should be in the 2 to 3 hour range at the average design flow rate including consideration for recirculation. Detention periods in clarifiers vary with surface-loading rates and side-water depths. The detention periods for final clarifiers following various activated sludge processes should be adjusted to between 2 and 4 hours dependent upon the type of process, design flow, recirculation rate, and surface-loading rate.

6.3.5.2 Surface-Loading Rates

Surface-loading rates for final clarifiers following activated sludge processes, such as: conventional, step aeration, contact stabilisation or the carbonaceous stage of separate-stage nitrification, shall not exceed 1.2 m/h Surface-loading rates for final clarifiers following an extended aeration process shall not exceed 1.0 m/h at peak hourly flow. Surface-loading rates for final clarifiers following separate-stage nitrification shall not exceed 1.0 m/h at peak hourly flow.

6.3.5.3 Solids Loading Rate (solids flux)

The solids loading rate (solids flux), excluding chemical additives applied to final clarifiers, shall not exceed 5 $kg/m^2.h$ at peak flow rate with 100% recirculation. Clarifiers following an extended aeration process or an oxidation ditch should not exceed this loading rate. The solids loading rate for final clarifiers following most activated sludge processes shall not exceed 8 $kg/m^2.h$ at peak hourly flow. The designer should assure that the clarifier area provided is equal to the greater of that required by surface-loading or by solids loading rates based on the maximum sludge volume index (SVI) anticipated and mixed-liquor suspended solids (MLSS).

The solids-loading rate for an activated-sludge clarifier may be computed by dividing the total solids applied by the surface area of the tank as follows:

Surface Loading Rate ($kg/m^2.h$) = MLSS (p+r) . Q/A ……………………………………..(6.1)

Where:

MLSS = kg/m^3

p = peak flow factor

r = recycle ratio (usually 1:1)

Q = average flow per hour (m^3/h)

A = surface area (horizontal floor area) of tank (m^2)

6.3.5.4 Sludge Storage

The floors of circular clarifiers shall be sloped at about (1:12) to form an inverted cone to a central sludge hopper. Simple mechanical collectors are recommended over the suction or siphon type for nutrient removal plants or where denitrification occurs in the aeration tank since rapid recirculation of solids is not required. Suction lift or siphon removal of sludge from flat-bottomed tanks is preferred where rapid removal of sludge is desirable (no denitrification and high temperatures).

The sludge or settled solids in clarifiers shall be scraped or drawn to a hopper or sump appropriately located for removal. The minimum slope of the side walls of sludge hoppers shall be 1.7 vertical to 1.0 horizontal. Hopper bottoms shall have a maximum dimension of 600 mm. Hopper walls shall be smooth with rounded corners. Extra depth sludge hoppers for sludge thickening are not advisable.

6.4 CONCLUSION

Besides ensuring that sedimentation tanks are sized in accordance with the loading criteria set out above, it is essential that such elements as the inlet, stilling box, scum board, weirs and launders and sludge draw-off lines are correctly designed in accordance with the criteria laid down. Adjustable weirs should be installed so as to obtain a uniform overflow of settled sewage at all points around the tank. Arrangements should be provided for cleaning sludge draw-off pipes. The sludge draw-off point should be easily accessible for sampling and observation during sludge draw-off periods.

6.5 WORKED EXAMPLES

Task - Design a primary sedimentation tank arrangement, and an activated sludge clarifier arrangement for a 10 Mℓ/d works

<u>Hydraulic Load</u>

Assume a peak flow factor of 2.5

Average hourly flow rate = 10 x 1000/24 = 417 m^3/h

Peak hourly flow rate = 10 x 1000 x 2.5/24 = 1042 m^3/h

Primary Sedimentation Tank (PST)

Select a maximum surface loading (Upflow rate) at peak flow of 2.2 m/h

Area of sedimentation tank(s) = 1042/2.2 = 474 m^2

The works is larger than 500 kℓ/d so at least two PST's should be provided to give a degree of standby.

Select two tanks for reason of economy

Area per tank = 474/2 = 237 m^2

Diameter of tank = $(237/\pi)^{0,5}$ x 2 = 17,37 (say) 17.5 m

Check weir loading = (1042/2)/(17.5 x π) = 9.5 m^3/m.h (which is OK)

Select Sidewater depth (SWD) of 2.5 m

Stilling chamber diameter @ 20% of diameter = 17.5 x 0.2 = 3.5 m

Stilling chamber depth @ 65% of SWD = 2.5 x 0.65 = 1.625 m.

Check outflow velocity from stilling chamber = [(1042/2)/60] / π x $(3.5/2)^2$ = 0.9 m/min (which is OK)

Weirs, baffles, and channels to be as specified above

Activated sludge clarifier

Select a maximum surface loading (upflow rate) of 1 m/h

Area of clarifiers = 1042/1 = 1042 m^2

The works is larger than 500 kℓ/d so at least two clarifiers should be provided to give a degree of standby.

Select two tanks for reason of economy

Area per tank = 1042/2 = 521 m^2

Diameter of tank = $(521/\pi)^{0,5}$ x 2 = 25.7 (say) 26 m

Check weir loading = (1042/2)/(26 x π) = 6.4 m^3/m.h (which is OK)

Select Sidewater depth (SWD) of 3.0 m

Stilling chamber diameter @ 20% of diameter = 26 x 0.2 = 5.2 m

Stilling chamber depth @ 65% of SWD = 3.0 x 0.65 = 1.95 m.

Check outflow velocity from stilling chamber = [(1042/2)/60] / π x $(5.2/2)^2$ = 0.4 m/min (which is OK)

Check solids flux - assume MLSS of 4 000 mg/ℓ and a return activated sludge ratio of 1:1 (on average flow)

Peak solids load on clarifier = (1042 + 417) x 4000/1000 = 5 835 kg/h

Solids flux = 5835/(521 x 2) = 5.6 kg/m^2.h (which is satisfactory)

Weirs, baffles, and channels to be as specified above

CHAPTER 7

7. ACTIVATED SLUDGE

7.1 ACTIVATED SLUDGE PROCESSES

7.1.1 INTRODUCTION

The activated sludge process effects oxidation of organic compounds similar to a biological filter but whereas sewage trickles over a packed bed in a biofilter, the activated sludge process necessitates that oxygen is introduced by mechanical means into the liquid. In the biological filter, the purifying micro-organisms are attached to the media as a relatively thin film, but in the activated sludge process these micro-organisms are suspended in the liquid as an 'active' sludge.

The process comprises a reactor basin in which micro-organisms are suspended in the sewage, oxygen is introduced by mechanical means and bio-chemical reactions take place by which organic pollutants are removed from the sewage. The suspended active sludge is separated from the effluent in a clarifier and is returned to the reactor. The mass of active sludge continually grows and sludge has to be wasted from the system from time to time to maintain the correct balance.

FIGURE 7.1.1: Typical activated sludge reactor basin (eWISA)

It should be clearly understood that the active sludge is a living microbial culture which develops in the process. This active sludge is not composed of the sludge which is introduced with the sewage, but grows as a result of the organic 'food' present in the influent. The composition and characteristics of this active sludge are determined by various factors such as the composition of the sewage, the retention time in the reactor, aeration rates and patterns and the average age of the sludge.

7.1.2 VARIANTS OF THE ACTIVATED SLUDGE PROCESS

The two basic types of activated sludge process are **Conventional Activated Sludge** where the process comprises treatment of settled sewage, and **Extended Aeration** where the feed is raw or unsettled sewage.

Conventional activated sludge is normally used for larger plants, usually 10 Mℓ/d and larger. This is because primary settlement of sewage necessitates the provision of primary sedimentation tanks and separate sludge treatment and disposal, and this only becomes economically justifiable on larger plants.

Extended aeration tends to be used on smaller plants usually less than 10 Mℓ/d. The extended aeration process is simple consisting of an aeration tank and a settling tank (clarifier) for the liquid treatment and on a sequencing batch reactor (SBR) plant only an aeration tank is required.

All activated sludge plants generate waste activated sludge, however, and the provision of lagoons, or drying beds, or mechanical de-watering equipment is also required.

There are several variants on the basic process, among which are plug flow and completely mixed aeration tanks, oxidation ditches, biological nutrient removal processes, membrane reactors, and sequencing batch reactor (SBR) systems which operate in a partially batch mode. There are also variants based on the mode of aeration such as coarse or fine bubble diffused air plants, high or low speed turbine aerators, horizontal shaft aeration paddles and discs, or submerged turbine aerators. The plants can also be distinguished by the loading regime applied where they may operate as highly loaded low sludge age plants without nitrification all the way to lightly loaded plants (usually of the extended aeration type) with very long sludge ages (over 60 days) where there is little waste sludge production.

7.1.3 PROCESS CONSIDERATIONS

In the past 20 years the understanding of the activated sludge process has been greatly advanced. Work done by many researchers in this country notably at the University of Cape Town, and by others overseas has led to the development of models as powerful aids to process design which can be loaded onto computers and which can size the reactors, the air requirements and the various aerobic, anoxic, and anaerobic compartments required.

Full presentation of these models is beyond the scope of this manual. Training and experience is needed to use the models and to select suitable values for the various coefficients used therein. As an aid to understanding, some of the concepts are discussed below. However the design basis presented in this manual will be the traditional empirical method as this is regarded as more appropriate for the non-specialist who does not carry out designs routinely. Use of the models is encouraged for specialists who have the necessary background training and facilities to carry out pilot studies where appropriate

7.1.3.1 Components of Wastewater

Wastewater strength is expressed as oxygen demand i.e. the amount of oxygen required per unit volume of wastewater to break down all the organic compounds present. Oxygen demand is usually measured as COD or BOD. Not all COD is biodegradable however, and therefore either the BOD or the biodegradable fraction of the COD is the significant parameter for design. This manual uses COD in its criteria, but if another source is used which expresses loadings in terms of BOD, the relationship between COD and BOD as set out in Chapter 2.5 may be applied for domestic sewage.

The organic component of sewage measured as COD may be divided into three parts:
(i) the biodegradable fraction, which can be utilised by the bacteria in the activated sludge process
(ii) the non-biodegradable particulate fraction which will form part of the sludge mass within the reactor being settled and returned from the clarifier and discharged with the waste activated sludge
(iii) the non-biodegradable soluble fraction which will be passed through the process unchanged.

7.1.3.2 Activated Sludge Mass

The mass of activated sludge in the reactor comprises four components:
(i) the mass of live organisms (biomass) which constitutes the reactive fraction of the sludge
(ii) the endogenous residue resulting from the decay of live biomass. This residue represents approximately 20% of the original live biomass.
(iii) the non-biodegradable organic mass fraction
(iv) the non-volatile or mineral mass.

(i), (ii) and (iii) together provide the total volatile mass in the reactor which is referred to as the mixed liquor volatile suspended solids (MLVSS). The addition of (d) provides the total mass of mixed liquor suspended solids (MLSS). Both the MLVSS and the MLSS are expressed as kg/m^3 or, in terms of concentration, as mg/ℓ. Normally the MLSS is only slightly greater than the MLVSS and they are often used interchangeably.

7.1.3.3 Waste Activated Sludge

The difference between the rate of growth of bacteria and the rate at which they die or decay is the surplus sludge generated from the biodegradable sewage fraction, to which must be added the mass of non-biodegradable material accumulated each day. In order to maintain steady state conditions this quantity, less the mass of sludge lost in the final effluent, must be wasted at regular intervals.

7.1.3.4 Nitrification / Denitrification

Nitrification is the biological process whereby nitrogenous compounds are converted to nitrates. The process takes place in two stages, firstly the conversion of ammonia to nitrites by the bacterial species *Nitrosomonas* and secondly the conversion of nitrites to nitrates by the bacterial species *Nitrobacter.* The growth rate of *Nitrosomonas* is far slower than that of *Nitrobacter* and the former reaction is therefore the rate-limiting step in the conversion process. As the DWAF effluent discharge regulations set a limit on the ammonia content, this generally means that activated sludge plants should be designed for nitrification.

The rate of reaction of nitrifying bacteria is highly temperature dependent and furthermore they require a dissolved oxygen concentration of at least 1 mg/ℓ in order to perform efficiently. This often means that nitrification is a critical factor in the design and operation of activated sludge plants. Nitrification also results in the formation of nitric acid, which reacts with, and reduces, the alkalinity in the wastewater. A low alkalinity causes a severe drop in pH and the addition of alkali (lime or soda ash) is often necessary in such cases.

Denitrification is the biological process where nitrates are reduced to nitrogen gas. This will only occur under conditions where no dissolved oxygen is present for microbial metabolism. Certain of the organisms then utilise the oxygen in the nitrate ion instead of free dissolved oxygen thereby producing nitrogen gas. This implies the existence of anoxic conditions (no dissolved oxygen present but nitrate present). This is distinguished from anaerobic conditions where there is no nitrate or dissolved oxygen present. The process of denitrification also recovers some of the alkalinity and it is often possible to avoid the need to add alkali to the process by encouraging adequate levels of denitrification.

Facilities for denitrification are often built in to the design of activated sludge plants by the provision of an anoxic zone or the use of timers to programme the aerators. This is particularly the case for biological nutrient removal (BNR) plants where the effluent has to comply with maximum nitrate and phosphorus standards. The design of BNR plants is effectively outside the scope of this manual as it generally makes use of the design models discussed above. However a mixed anoxic chamber with a retention of (say) 3 to 6 hours can be added to the head of the aeration tank to reduce nitrogen sufficiently in most cases for alkalinity conservation purposes.

7.1.3.5 Phosphorus

Nitrogen and particularly phosphorus (phosphate) are the nutrients involved in eutrophication of lakes, dams, and rivers and phosphorus, usually as inorganic phosphate is the limiting nutrient. For this reason DWAF has set fairly stringent phosphate limits on wastewater effluents (1.0 mg/ℓ) in designated sensitive catchments, and this in turn led to the interest in BNR plants discussed above.

Activated sludge is inherently better suited to BNR than fixed film processes such as biological filters, as the process is more easily modified. However BNR is not always successful in that the limits are often not complied with, and supplementary metal salt addition is then necessary. Metal salt addition is therefore normally provided as a matter of course.

Biological phosphorus removal is achieved by the provision of anaerobic zones in the aeration tank sequence and there are varying process configurations. Anaerobic zones are distinguished from anoxic zones in that the former have no oxygen (DO) or nitrate, whereas the latter have no DO but have nitrate present. In the anaerobic zone the activated sludge releases phosphate into the liquid. In the subsequent aerated zone the phosphate is taken up again together with the phosphate in the incoming wastewater thus removing it from the effluent.

The process is not stable in that subsequent anaerobic conditions can release phosphate again so that care is needed in the design of clarifiers and sludge dewatering to prevent excessive unaerated retention times. It is partly for this reason that the phosphate is often fixed in the sludge by the addition of metal salts.

The design of BNR plants is a specialised field and best left to experienced design practitioners. However the addition of metal salts is a relatively simple addition to the process and is covered in Chapter 11.

7.1.3.6 Return Activated Sludge

In order to retain the required level of activated sludge solids in the reactor the sludge which settles in the clarifier needs to be returned continuously to the aeration tank, with the rate being governed by the settleability of the sludge and the mean solids concentration required in the reactor.

7.1.3.7 Sludge Settleability

The sludge volume index (SVI) or the stirred specific volume index (SSVI) provides the information on both the settling and thickening properties of the sludge. The SSVI is defined as the volume occupied by 1 g of sludge after 30 minutes of settling in a gently stirred (1 rpm) settling column at a standard initial concentration of 3.5 g/ℓ MLSS. The SVI which is easier to use in the field is the volume occupied by 1 g of activated sludge after settling for 30 minutes in a 1 litre measuring cylinder.

In both cases the test is independent of settling column size, although for convenience SSVI data is normally measured in a 1000 ml graduated column or measuring cylinder, i.e.:

$$\text{SVI or SSVI} = V/X \quad (\text{m}\ell/\text{g}) \qquad \text{.......................................} (7.1)$$

Where

V = settled sludge volume in ml in a 1000 mℓ column

X = mixed liquor concentration (g/ℓ).

A good settling sludge will have an SVI of about 100 mℓ/g or less. SVI's in the 200 to 300 mℓ/g levels can cause settling problems in clarifiers. Adequate oxygen supply is one of the important factors in ensuring a good settling sludge.

7.1.3.8 Oxygen Demand

The quantity of oxygen required for synthesis of bacteria and for endogenous respiration is known as the carbonaceous oxygen demand and is dependent upon sludge age and COD loading. The oxygen demand for synthesis is proportional to the amount of biodegradable material present in the incoming sewage, while the endogenous demand is proportional to the mass of active sludge (i.e. sludge age). The oxygen requirement for nitrification is directly proportional to the quantity of ammonia converted to nitrate.

7.1.3.9 Dissolved Oxygen (DO)

As indicated above a minimum dissolved oxygen concentration of 1 mg/ℓ is required for nitrification. For efficient oxygen transfer a lower value is desirable but for good settling sludges, somewhat higher values are preferable. In practice plants are generally operated with a DO concentration of between 0.5 and 2 mg/ℓ.

7.1.3.10 Primary Settlement

The provision of primary sedimentation tanks is not essential for the operation of activated sludge plants. As mentioned previously this decision is normally based on the size of the works. The lower power requirement and reactor volume of a plant incorporating primary sedimentation should be compared with the additional cost of providing sedimentation tanks and digesters. Primary sedimentation is generally not justifiable for smaller works.

7.1.3.11 Plug Flow and Completely Mixed Reactors

A plug flow system is one in which the length to width ratio is in the order of 3:1 or greater. In this system there is theoretically no longitudinal mixing and it is assumed that the sewage progresses through the reactor as a plug. In a completely mixed reactor sewage entering the reactor is assumed to be uniformly dispersed throughout the basin within a short time scale. In a plug flow system the oxygen demand reduces along the length of the reactors and the rate of aeration may thus be reduced accordingly. In a completely mixed system the oxygen demand is assumed to be uniform throughout the reactor.

7.1.3.12 Oxidation Ditches

Oxidation ditches are examples of plug flow reactors with high recycle and a feature of oxidation ditches is the high occurrence of denitrification. The dissolved oxygen level is highest at the aerator and decreases as the distance from the aerator increases. If the dissolved oxygen level falls to near zero in before the mixed liquor reaches the next aerator, anoxic conditions develop and denitrification may occur.

While the design data used for fully mixed systems may be applied to oxidation ditches, the presence of an anoxic zone has a significant effect on the process kinetics. The minimum sludge age for nitrification refers only to the aerated fraction of the mixed liquor and therefore, if denitrification is to occur, the required anoxic capacity must be added to the aerobic volume to provide the total volume. Because the anoxic zone cannot be clearly defined the final design of oxidation ditches remains empirical. Oxidation ditches generally treat screened unsettled sewage.

7.1.4 DESIGN PARAMETERS

7.1.4.1 Sludge Age and Solids Loading Rate

This is the primary parameter in determining plant design as it affects all other variables. Activated sludge plants in South Africa generally operate at long sludge ages (approximately 15 days or greater), at which nitrification is generally assured and a relatively stable sludge generated. For minimal sludge production on small plants sludge ages of 30 days or even longer may be employed. Loading rates are inversely related to sludge age, i.e. a high sludge loading rate is associated with a low sludge age and vice versa. Permissible loading may thus be assessed in terms of Sludge Loading Rate (kg COD/day.kg MLSS), according to the values given in **Table 7.1.1** below.

TABLE 7.1.1: Approximate relationship between sludge age yield factor for new biomass produced per unit of COD loading and loading rate.

Sludge Age (Days)	Sludge Loading Rate (Kg COD/kg MLSS.d)	Sludge Growth Index (kg MLSS/kg COD)		Recommended Sludge Density (kg MLSS/m^3)
		Settled Sewage	Raw Sewage	
30	0.154	0.22	0.23	5.0
25	0.174	0.23	0.25	4.4
20	0.200	0.25	0.27	3.9
15	0.242	0.28	0.31	3.5
10	0.312	0.32	0.36	3.3
5	0.506	0.40	0.45	3.2

7.1.4.2 Activated Sludge Mass

The total mass of activated sludge required depends on the sludge age (or solids loading rate), the minimum reactor temperature and the biodegradable fraction of the incoming sewage. This mass of active sludge comprises not only that which is contained in the (aerobic) reactor, but also that in the final clarifier and the sludge return system. Design procedures result in a more conservative design, if the mass of the active sludge in the reactor only is taken into account.

Sludge age will determine the yield factor for new biomass produced per unit of COD loading, while at the same time limiting the loading rate. The approximate relationships between these three parameters are given in **Table 7.1.2** above:

The sludge mass in the system will then be equal to the product of the sludge age, the yield factor and loading in kg COD per day.

7.1.4.3 Reactor Volume

In present day designs the reactor volume is not a primary design parameter but is determined by the required sludge mass divided by the design MLSS. Thus if the total sludge mass is M_t (kg) and the total mixed liquor solids concentration is X_t (kg/m^3) the reactor volume (V_R) may be expressed as:

$V_R = M_t / X_t \quad (m^3)$ ……………………………………………………………(7.2)

Equally the reactor volume may be expressed in terms of volatile solids - i.e.

$V_R = M_v / X_v \quad (m^3)$ ……………………………………………………………(7.3)

Most models and the empirical procedure presented in this manual assume that reactor volume is a purely a function of the sludge mass (as determined from the COD load and the sludge loading rate which is in turn related to the sludge age) and the selected mixed liquor suspended solids concentration (sludge density). There are practical constraints which need to be borne in mind.

Firstly, there is a limit on the achievable MLSS. When using a clarifier this is often in the range of 5000 to 6000 mg/ℓ (5 to 6 kg/m^3). In practice some plants can operate at higher MLSS values if the sludge settles particularly well but often there will be settling problems in the clarifier due to high solids flux loadings. It is therefore not advisable to design on high MLSS values for the aeration tank.

Secondly, although most design models assume homogeneity of the substrate this is not the case for most wastewaters. Wastewater is a complex mixture of compounds which have a varying degree of biodegradability ranging from highly and rapidly biodegradable to compounds which are almost intractable and degrade very slowly. The models work for domestic sewage in the range of loadings normally applied but care is needed when extrapolating to industrial wastes in particular. It is not possible to fully treat some wastewaters in tanks with short aeration times. Many older design criteria therefore selected aeration tanks based on retention time, typically 12 to 18 hours for conventional activated sludge, and 18 to 24 hours for extended aeration. These criteria were purely empirical but were based on experience. In similar vein some older criteria set limits on volumetric loading (kg COD or BOD/m^3 of tank volume per day). These were typically 1.1 kg COD/m^3.d for conventional and 0.65 kg COD/m^3.d for extended aeration. These also have the effect of limiting the loading on the tank and ensuring adequate retention. There have been problems on some activated sludge plants with industrial effluents in the feed where the retention time has been designed as low as 6 hours

7.1.4.4 Waste Activated Sludge

The volume of sludge to be wasted each day (m^3/d) is dependent on sludge age and is calculated as follows:

(a) when wasting directly from the reactor:

Volume of reactor/Sludge age = Volume wasted (m^3/d) ……………………………………(7.4)

(b) when wasting from the returned activated sludge stream:

(Volume of reactor x MLSS)/ (Sludge age x RSS) = volume wasted (m^3/d) ……………….(7.5)

Where
RSS= Return Sludge Concentration (Clarifier underflow)

7.1.4.5 Nitrification

The relationship between the sludge age required for nitrification and the temperature is given by:

R_s = $3.05 \times (1.127)^{20-T}$ …………………………………………………………..(7.6)

Where
R_s = the minimum sludge age required for nitrification
T = temperature (ºC) of the mixed liquor in the reactor.

7.1.4.6 Return Activated Sludge

If p is the ratio of returned sludge to wastewater flow, RSS the concentration of the solids in the returned sludge and MLSS the mixed liquor concentration in the reactor then:

RSS = (1 + s) MLSS …………………………………………………………(7.7)

Values of 's' in excess of 1.5 are seldom encountered and are typically in the range 0.8 to 1.3 with a value of 1.0 being commonly selected. As indicated by the above equation increasing the recycle ratio would provide a diminishing return.

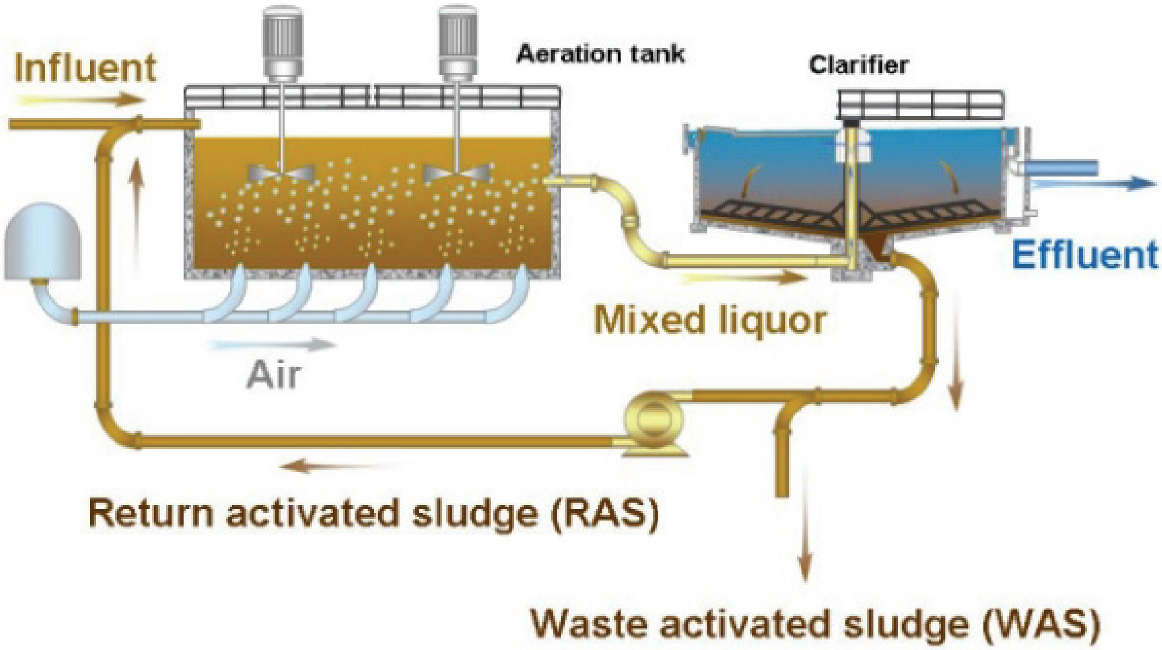

FIGURE 7.1.2: Typical activated sludge flow diagram (eWISA)

7.1.4.7 Oxygen Requirements

The carbonaceous oxygenation requirement is a function of the strength of the sewage, measured as either COD or BOD and sludge age (or sludge loading rate). Empirical values which may be used as an alternative to other more mathematical approaches in use are given in **Table 7.1.2** below:

TABLE 7.1.2: Empirical values to determine carbonaceous oxygen requirement

Sludge Loading Rate (kg COD applied per day/ kg MLSS)	Sludge Age (days)	Dissolved Oxygen required for carbonaceous Oxidation (kg O_2/kg COD removed	Ratio of maximum to mean Oxygenation Rate	Ratio of minimum to mean Oxygenation Rate
0.154	30	0.84	1.45	0.5
0.174	25	0.82	1.47	0.5
0.200	20	0.79	1.50	0.5
0.242	15	0.73	1.55	0.5
0.312	10	0.65	1.67	0.5
0.506	5	0.53	1.80	0.5

The oxygen demand for nitrification is directly proportional to the mass of nitrogeneous compounds which are to be nitrified, the relationship being approximately 4.5 g O_2 per 1 g TKN (Total Kjeldahl nitrogen) or ammonia as N converted.

7.1.4.8 Capacity of Aeration Equipment

There are several different types of aeration equipment available. The transfer efficiency of the different devices differs in accordance with such parameters as power intensity and reactor geometry but typical figures for each type are given in **Table 7.1.3** below. These figures represent the transfer efficiency into clean water of low TDS at 20°C and at sea level, assuming zero residual DO and that the water contains no surfactants or other ingredients that could influence the rate of oxygen transfer.

TABLE 7.1.3: Typical figures for oxygen transfer efficiency into clean water for different types of aeration equipment

Type of Aeration	**Transfer Efficiency (**kg oxygen/kWh)
Fine bubble diffused aeration	2.4 to 3.2
Low speed surface aeration	1.7 to 2.5
Submerged turbine aerators	1.4 to 2.0
High speed surface aeration	1.2 to 1.7
Coarse bubble aeration	0.7 to 1.4 (depending on tank depth)

Note: In Table 7.1.3 the power is 'shaft' and not 'motor' power.

The transfer efficiency of aerators operating in wastewater will be significantly lower than the above figures due to the presence of surfactants and various other impurities. It is therefore necessary to apply an appropriate factor to the specified "oxygenation capacity" in order to arrive at "oxygen input". This factor is known as "α" (the alpha factor) and can range from 0.4 up to about 0.9. α for diffused air is lower than for mechanical aerators and can be in the range 0.4 up to about 0.7. For mechanical aerators it is usually in the range 0.65 to 0.9 and is often assumed at 0.85. Oxygen input is also influenced by altitude, temperature, residual DO and the positioning of aerators in the basin. The oxygen input is the oxygenation capacity divided by the α factor.

Further corrections for oxygen solubility on site compared to standard conditions by a formula known as Henry's Law (or from tables) is also necessary, as well as correction to the oxygen solution driving force for the minimum specified DO concentration in the tank. If the solubility of oxygen on site is 7.5 mg/ℓ and a minimum DO of 1.5 mg/ℓ is specified, then the correction factors to the oxygen input will be roughly 9.0 / 7.5 (increase) for solubility and 7.5 / (7.5-1.5) (another increase) for driving force.

In addition to this, factors need to be put in for mechanical efficiency of motor and gearbox as well as power factor effects. The net effect of all the correction factors together can be an increase of 50% to even a doubling of the originally calculated oxygen requirement.

7.1.5 WORKED EXAMPLE

Task Design an extended aeration activated sludge plant for a 10 Mℓ/d works treating predominately domestic sewage at an altitude of 600 m above sea level.

<u>Wastewater Load</u>

Assume a wastewater COD of 650 mg/ℓ (= 0.65 kg/kℓ)

Assume ammonia (TKN) of 0.12 x COD = 0.078 kg/kℓ)

COD load = 10 000 x 0.65 = 6 500 kg/d

Ammonia load = 10 000 x 0.078 = 780 kg/d

<u>Aeration Tank Sizing</u>

Select a sludge age of 20 days

From Table 8.1 sludge growth coefficient = 0.27 kg MLSS/kg COD

Sludge produced = 6 500 x 0.27 = 1 755 kg/d

Total sludge in system = 1 755 x 20 = 35 100 kg

Select a MLSS of 4 000 mg/ℓ (4.0 kg/m^3)

Reactor volume = 35 100/4 = 8 775 (say) 9 000 m^3

Select two parallel aeration lines with each tank containing three slow speed aerators in series. (This gives more process flexibility than a single tank with six aerators. The choice of number of aerators is a compromise between the increased cost of a larger number of aerators and the process inflexibility of too few aerators. Very small plants will only have one or two aerators – large plants can have 15 or more).

Volume per reactor = 4 500 m^3

Select tank depth of 4,0 m - area = 4 500/4 = 1 125 m^2

Dimensions of each aerator compartment = $(1\,125/3)^{0,5}$ = 19.36 (say) 19.5 m

Aeration reactors dimensions = 58.5 x 19.5 x 4.0 m (two tanks each with three aeration compartments).

<u>Oxygen Requirements (aerator sizing)</u>

From Table 8.2 - Oxygen requirement for COD load for a 20 day sludge age

= 6 500 x 0.79 x 1.50 = 7 703 kg/d

Oxygen required for ammonia oxidation = 780 x 4.5 = 3 510 kg/d

Total oxygen required = 7 703 + 3 510 = 11 213 kg/d = 467,2 kg/h

Oxygen required per aerator = 467.2/6 = 77.9 kg/h

Specify a minimum dissolved oxygen (DO) in aeration tank of 1.5 mg/ℓ

Barometric pressure at sea level = 1 017 mBar

Barometric pressure at 600m elevation (or measured on site) = 950 mBar

Oxygen solubility at sea level at 20º C = 9.0 mg/ℓ (from tables)

Oxygen solubility at 600 m = 9.0 x 950/1 017 = 8.4 mg/ℓ

Correction factor for site solubility = 9.0/8.4 = 1.071

Correction factor for minimum DO in aeration tank = 8.4/(8.4 – 1.5) = 1.217

Assume transfer efficiency of 2.0 kg oxygen/kWh for slow speed aerators into clean water under standard conditions and an alpha factor of 0.85 for transfer into mixed liquor

Aerator power required = 77.9/(2.0 x 0.85) x 1.071 x 1.217 = 59.7 (say) 60 kW per aerator and 60 x 6 = 360 kW for all six aerators.

In specifying aerators it should be borne in mind that allowance should be made for motor and gearbox efficiency. It is common practice to specify a service factor of 2.0 for the gearboxes and to allow 10 to 15% additional motor capacity (say 67 kW motors in the present case).

7.2 SEQUENCING BATCH REACTORS (SBR)

7.2.1 INTRODUCTION

The sequencing batch reactor (SBR) is a fill-and-draw activated sludge system for wastewater treatment. In this system, wastewater is added to a single 'batch' reactor, treated to remove undesirable components, and then discharged. Equalisation, aeration, and clarification can all be achieved using a single batch reactor. SBR systems have been successfully used to treat both municipal and industrial wastewater. They are suited to wastewater treatment applications characterised by low or intermittent flow conditions.

The unit processes of the SBR and conventional activated sludge systems are the same. A 1986 USEPA report summarised this by stating that "the SBR is no more than an activated sludge system which operates in time rather than in space". The difference between the two technologies is that the SBR performs equalisation, biological treatment, and secondary clarification in a single tank using a timed control sequence. In a conventional activated sludge system, these unit processes would be accomplished by using separate tanks.

7.2.2 PROCESS DESCRIPTION

Influent wastewater generally passes through screens and grit removal prior to the SBR. It is possible to have primary sedimentation before the SBR but most SBR plants are of relatively low capacity where PST's and separate sludge treatment facilities are not warranted. In any event one of the attractions of the SBR plant is its operational simplicity and this is counteracted by the installation of primary sedimentation and sludge treatment.

The wastewater then enters a partially filled reactor, containing biomass, which is acclimated to the wastewater constituents during preceding cycles. Once the reactor is full, it behaves like a conventional activated sludge system. The aerator may operate while the tank is filling but once full, there is no inflow or outflow until the decant cycle starts.

The aeration and mixing is then discontinued after the biological reactions are complete, the biomass settles, and the treated supernatant is decanted. Excess biomass can be wasted at any time during the cycle. In the absence of a clarifier there is no need for return activated sludge (RAS) pumps.

An SBR serves as an equalisation basin when the vessel is filling with wastewater, enabling the system to tolerate peak flows or peak loads in the influent and to equalise them in the batch reactor. However unless there is more than one SBR tank at the installation, some form of wastewater holding tank is preferable to hold the wastewater during the settlement and decant parts of the cycle.

7.2.3 APPLICABILITY

SBRs are typically used at flow rates of 10 Mℓ/d or less and most are significantly smaller than this being typically of large package plant size. The more sophisticated operation required at larger SBR plants tends to discourage the use of these plants for large flow rates.

As these systems have a relatively small footprint, they are useful for areas where the available land is limited. In addition, cycles within the system can be easily modified for accommodating nutrient removal if necessary. This makes SBR's flexible in operation.

7.2.4 ADVANTAGES AND DISADVANTAGES.

Some advantages and disadvantages of SBR's are listed below:

7.2.4.1 Advantages

1. Equalisation, biological treatment, and secondary clarification can be achieved in a single reactor vessel
2. Operating flexibility and control
3. Minimal footprint
4. Potential capital cost savings by eliminating clarifiers and other equipment.

7.2.4.2 Disadvantages

1. A higher level of mechanical sophistication is required (compared to conventional systems), especially for larger systems, in terms of timing units and controls
2. Higher level of maintenance (compared to conventional systems) associated with more sophisticated controls, automated switches, and automated valves
3. Potential of discharging floating or settled sludge during the decant phase with some SBR configurations
4. Potential plugging of aeration devices during some operating cycles, depending on the aeration system used by the manufacturer
5. Potential requirement for equalisation before and after the SBR, depending on the upstream and downstream processes.

7.2.5 DESIGN

The process is essentially an activated sludge plant and the design guidelines particularly for extended aeration are relevant. However as a batch type process the loadings and retentions apply somewhat differently. **Table 7.2.1** below gives figures adapted from typical American practice as a cross-check.

Once the key design parameters are determined, the number of cycles per day, number of basins, decant volume, reactor size, and detention times can be calculated. Additionally, the aeration equipment, decanting, and associated piping can then be sized. Other site specific information is needed to size the aeration equipment, such as site elevation above mean sea level, wastewater temperature, and total dissolved solids concentration.

TABLE 7.2.1: Key design parameters for a conventionally loaded SBR

	Municipal	**Industrial**
Sludge loading rate (F:M) (kg COD/kg MLSS.d)	0.15 – 0.4	0.15 – 0.6
Treatment cycle duration (hours)	6.0 – 18	4.0 - 24
Typical low water level MLSS (mg/ℓ)	2 000 – 2 500	2 000 – 4 000
Hydraulic Retention Time (hours)	10 – 30	Varies

The operation of an SBR is based on the fill-and-draw principle, which consists of the following five basic steps: Idle, Fill, React, Settle, and Draw. More than one operating strategy is possible during most of these steps. For industrial wastewater applications, treatability studies are typically required to determine the optimum operating sequence. For most municipal wastewater treatment plants, treatability studies are not required to determine the operating sequence because municipal wastewater flow rates and characteristic variations are usually predictable and most municipal designers will follow conservative design approaches.

Construction of SBR systems can typically require a smaller footprint than conventional activated sludge systems. The SBR never requires secondary clarifiers. The size of the SBR tanks themselves will be site specific. Normally, however, the SBR system is advantageous if space is limited at the proposed site.

A few case studies are presented in the **Table 7.2.2** below to provide general sizing estimates at different flow rates. Sizing of these systems is site specific and these case studies do not reflect every system at that size. Also these reflect American practice where wastewater strengths are low. If directly applied to local conditions an increase in tank size of 50% to 100% would be needed in most cases to accommodate the limitation on biological loading rates.

TABLE 7.2.2: Case studies for several SBR installations.

Flow (m^3/d)	No. of Reactors	Volume (m^3)
45	1	80
375	2	260
4 500	2	3 400
3 750	2	1 780
5 200	2	2 550
5 450	2	3 560
15 800	4	5 800
19 300	4	5 100

SBR manufacturers should be consulted for recommendations on tanks and equipment. It is typical to use a complete SBR system recommended and supplied by a single SBR manufacturer. It is possible, however, for an engineer to design an SBR system, as all required tanks, equipment, and controls are available through different manufacturers. However this approach is not typical of SBR installations, because of the level of sophistication of the instrumentation and controls associated with these systems.

The SBR tank is typically constructed with steel or concrete. For industrial applications, steel tanks coated for corrosion control are most common, while concrete tanks are the most common for treatment of domestic wastewater.

Significant operating flexibility is associated with SBR systems. For a BNR plant, the aerated mode and the mixed mode (anoxic conditions) can be alternated to achieve nitrification and denitrification. In addition, these modes can ultimately be used to achieve an anaerobic condition where phosphorus removal can occur.

7.3 MEMBRANE BIOREACTORS (MBR)

7.3.1 INTRODUCTION

Membrane bioreactors are a recent innovation in wastewater treatment and make use of the activated sludge process in a modified form. In this type of plant the clarifier is replaced by semi-porous membranes arranged in sheets or tubes in the aeration reactor. The treated effluent passes through the membrane whilst the activated sludge is left behind. This avoids the need for a clarifier and sludge return pumps but is not always significantly cheaper as the membranes can be expensive. However it is a beneficial process for upgrading plants which are overloaded as it can operate at very high MLSS concentrations and short liquid retention times. It is also useful where compactness is an important consideration.

The MBR process utilises microporous membranes for solid/liquid separation in lieu of secondary clarifiers. This very compact arrangement produces a microfiltration/ultrafiltration (MF/UF) quality effluent suitable for reuse applications or as a high quality feed water source for reverse osmosis treatment. Indicative output quality of MF/UF systems include SS <1mg/ℓ, turbidity <0.2 NTU and up to 4 log removal of virus (depending on the membrane nominal pore size). In addition, it provides a barrier to certain chlorine resistant pathogens such as *Cryptosporidium* and *Giardia*.

The MBR process is an emerging advanced wastewater treatment technology that has been successfully applied at an ever increasing number of locations around the world. In addition to their steady increase in number, MBR installations are also increasing in terms of scale. A number of plants with a treatment capacity of around 5 to 10 Mℓ/d have been in operation for several years now whilst the next generation (presently undergoing commissioning or under contract) have design capacities up to 45 Mℓ/d.

7.3.2 PROCESS DESCRIPTION

The MBR process has an aeration tank containing mixed liquor as is the case for other activated sludge variants. A typical arrangement will include submerged membranes in the aerated portion of the bioreactor, and an anoxic zone and internal mixed liquor recycle can be incorporated if required. As an alternative some plants have used pressure membranes (rather than submerged membranes) external to the bioreactor.

The wastewater enters the aeration tank where it is aerated as in a conventional process. The differences are that the aeration time is usually much shorter than conventional activated sludge (typically 3 to 4 hours), and the MLSS (sludge solids concentration) much higher (typically 12 000 to 16 000 mg/ℓ compared to about one quarter of this for a conventional plant). A high rate of diffused air is required through the membranes to scour them and to aerate the high concentration of activated sludge in the tank.

The treated and clear effluent passes through the membranes usually by gravity, although some designs use suction pumps, and this comprises the effluent from the process. The membranes are in the MF/UF range with a typical pore size of 0.4 μm and are of the low pressure type. Most designs use flat sheet membranes but there are hollow fibre and tubular designs available. Sludge is wasted from the reactor as in a conventional process and the MBR system has the advantage that pre-thickening of the sludge for dewatering is often not necessary

7.3.3 ADVANTAGES OF THE MBR SYSTEM

1. Secondary clarifiers and tertiary filtration processes are eliminated, thereby reducing plant footprint. In certain instances, footprint can be further reduced because other process units such as digesters or UV disinfection can also be eliminated/minimised (dependent upon governing regulations)
2. Unlike secondary clarifiers, the quality of solids separation is not dependent on the mixed liquor suspended solids concentration or characteristics. Since elevated mixed liquor concentrations are possible, the aeration basin volume can be reduced, further reducing the plant footprint.
3. No reliance upon achieving good sludge settleability is necessary, hence quite amenable to remote operation
4. Can be designed with long sludge age, hence low sludge production
5. Produces a MF/UF quality effluent suitable for reuse applications or as a high quality feed water source for Reverse Osmosis treatment. Indicative output quality of MF/UF systems include SS <1mg/ℓ, turbidity <0.2 NTU and up to 4 log removal of virus (depending on the membrane nominal pore size). In addition, MF/UF provides a barrier to certain chlorine resistant pathogens such as Cryptosporidium and Giardia.
6. The resultant small footprint can be a feature used to address issues of visual amenity, noise and odour. Example MBR plants exist where the entire process is housed in a building designed to blend in with its surrounding land use. This can reduce the buffer distance required between the plant and the nearest neighbour and can increase the surrounding land values.

7.3.4 DISADVANTAGES OF MBR SYSTEMS

1. Membranes are prone to fouling and good design is necessary to prevent this as far as possible. Also operation needs to be such as to minimise this.
2. Membrane life continues to improve but it can be expected that membrane replacement will be needed at about 5 year intervals. This will entail significant cost.
3. Membranes operate best within a limited flux range and flow balancing is necessary on many plants.

7.3.5 COST COMPARISON – MBR VERSUS ALTERNATIVE PROCESS TRAINS

A detailed holistic cost comparison may reveal reasonably comparable results between the cost of the MBR option versus other advanced treatment options, especially if land value is considered. Furthermore, whilst the costs for conventional technologies are slowly rising with labour costs and inflationary pressures, the costs for all membrane equipment (both for direct filtration and MBR) has been falling steadily during each of the last 10 years. Hence on a capital cost basis for any given project, the likelihood of MBR becoming a favoured option is increasing with time. Designers are therefore advised to continuously review and to explore the options as the improvement in membranes, their lower relative cost, and the increasing cost of land may make the MBR option a preferred one on more occasions in the future

7.3.6 DESIGN

The design of membrane plants is a specialised process and suitably experienced practitioners in the field should be employed if a plant of this type is required. Any design received can then be assessed according to the norms presented in this section.

7.4 DIFFUSED AIR

7.4.1 INTRODUCTION

In wastewater treatment processes, aeration introduces air into a liquid, providing an aerobic environment for microbial degradation of organic matter. The purpose of aeration is two-fold:

1. To supply the required oxygen to the metabolising microorganisms; and
2. To provide mixing so that the microorganisms come into intimate contact with the dissolved and suspended organic matter.

The two most common aeration systems are subsurface and mechanical. In a subsurface system, air is introduced by diffusers or other devices submerged in the wastewater. A mechanical system agitates the wastewater by various means (e.g. propellers, blades, or brushes) to introduce air from the atmosphere.

Fine pore diffusion (fine bubble aeration) is a subsurface form of aeration in which air is introduced in the form of very small bubbles. Since the energy crisis in the early 1970’s, there has been increased interest in fine pore diffusion of air as a competitive system due to its high oxygen transfer efficiency. Smaller bubbles result in more bubble surface area per unit volume and greater oxygen transfer efficiency.

Coarse bubble diffusion is simpler than fine bubble aeration in that air filtration to prevent blockages of diffusers is not of serious concern. Fairly rudimentary filters can be used as the method of diffusion is through orifices which may be simply holes in a lateral pipe or pipes on the tank floor. Coarse bubble aeration is normally used on small plants where low capital cost is desired and the additional power cost penalty is small

Oxygen transfer studies that were performed on fine pore ceramic dome diffusers in order to compare results with the coarse bubble diffusers showed that the coarse bubble diffuser had an average standard oxygen transfer efficiency under field conditions of 4.8% with an average alpha (α) of 0.55. In contrast, the fine pore system has an average standard oxygen transfer efficiency under field conditions of about 9.5% and an average α of 0.4 during normal daytime high-load operation. Alpha is defined as the ratio of K_{La} (volumetric mass transfer coefficient) of a clean diffuser in process water, to the K_{La} of the same diffuser in clean water.

The amount of oxygen transferred is proportional to the depth of immersion, i.e. a 4 m deep tank will transfer nearly twice as much oxygen from the same amount of air as would be achieved in a 2 m deep tank which is why fine bubble diffusion is usually favoured for large plants where deep tanks are not out of proportion. Even for coarse bubble diffusion the aeration tanks should be made as deep as is feasible to optimise energy costs.

7.4.2 ADVANTAGES AND DISADVANTAGES OF FINE BUBBLE DIFFUSION

Some advantages and disadvantages of various fine pore diffusers are listed below:

7.4.2.1 Advantages

1. Exhibit high oxygen transfer efficiencies
2. Exhibit high aeration efficiencies (mass oxygen transferred per unit power per unit time)
3. Can satisfy high oxygen demands
4. Are easily adaptable to existing basins for plant upgrades
5. Result in lower volatile organic compound emissions than nonporous diffusers or mechanical aeration devices.

7.4.2.2 Disadvantages

1. Fine pore diffusers are susceptible to chemical or biological fouling, which may impair transfer efficiency and generate high head loss. As a result they require occasional cleaning which is a reasonably costly and labour-intensive procedure.
2. Fine pore diffusers may be susceptible to chemical attack (especially perforated membranes). Therefore, care must be exercised in the proper selection of material for a given wastewater.
3. Because of the high efficiencies of fine pore diffusers at low airflow rates, airflow distribution is critical to their performance and selection of proper airflow control orifices is important
4. Because of the high efficiencies of fine pore diffusers, the required airflow in an aeration basin (normally at the effluent end) may be dictated by mixing, not oxygen transfer
5. Aeration basin design must incorporate a means to easily dewater the tank for cleaning. In small systems, where no redundancy of aeration tanks exists, an in-situ, non process-interruptive method of cleaning must be considered.
6. Fine pore diffusers require sophisticated two or even three stage air filtration systems to avoid clogging.

7.5

7.5.1 ADVANTAGES AND DISADVANTAGES OF COARSE BUBBLE DIFFUSION

7.5.1.1 Advantages

1. Coarse bubble systems tend to be mechanically simple

2. Fairly basic air-filtration can be used
3. The diffusers can be simple orifices or holes in pipes
4. They are effectively clog-free
5. Can be modified or upgraded relatively easily
6. Mixing is usually effective.

7.5.1.2 Disadvantages

1. Only really suited to small plants because of their low transfer efficiencies
2. The types of blowers usually used are not as reliable as fine bubble systems or mechanical diffusers
3. Single orifices are less flexible for air flow variation if operation in the turbulent regime (high Reynolds numbers) is desired.

7.5.2 DIFFUSERS

In the past, various diffusion devices have been classified based on their oxygen transfer efficiencies as either fine bubble or coarse bubble. Since it is difficult to clearly demarcate or define between fine and coarse bubbles, diffused aeration systems have recently been classified based on the physical characteristics of the equipment. Diffused aeration systems can be classified into three categories:

1. Porous (fine bubble) diffusers: Fine pore diffusers are mounted or screwed into the diffuser header pipe (air manifold) that may run along the length or width of the tank or on a short manifold mounted on a movable pipe (lift pipe). These diffusers come in various shapes and sizes, such as discs, tubes, domes, and plates.
2. Nonporous (coarse bubble) diffusers: The common types of nonporous diffusers are fixed orifices (perforated piping, spargers, and slotted tubes); valved orifices; and static tubes. The bubble size from these diffusers is larger than the porous diffusers, thus reducing the oxygen transfer efficiency.
3. Other diffusion devices: These include jet aerators (which discharge a mixture of air and liquid through a nozzle near the tank bottom); aspirators (mounted at the basin surface to supply a mixture of air and water); and U tubes (where compressed air is discharged into the down leg of a deep vertical shaft).

7.5.3 TYPES OF FINE PORE DIFFUSERS

Fine pore diffusers (discs, tubes, domes, and plates) are usually made from ceramics, plastics, or flexible perforated membranes. Although many materials can be used to make fine pore diffusers, only the few mentioned above are commonly used due to cost considerations, specific characteristics, market size, and other factors.

Ceramic media diffusers have been in use for many years and have essentially become the standard for comparison since, in the past, they were the primary media in the fine pore aeration market. Ceramic, plastic, and flexible materials are resistant to the chemicals used in wastewater treatment.

Discussed below are common types of fine bubble diffusers. However, recent advances in technology have resulted in some modifications to these types.

7.5.3.1 Disc diffusers

Disc diffusers are relatively flat and range from approximately 180 to 250 mm in diameter with thicknesses of 13 to 19 mm. Materials for discs include ceramics, porous plastics, and perforated membranes. Therefore, thicknesses vary, as do construction features. Currently, manufacturers provide plenums or base plates that will accept all materials.

The disc is mounted on a plastic saddle-type base-plate, and either a centre bolt or a peripheral clamping ring is used to secure the media and the holder together. More commonly, the disc is attached to the holder via a screw-on retainer ring.

Disc diffusers are designed to have an airflow range of 0.25 to 1.5 l/s per diffuser.

7.5.3.2 Tube / flexible sheath diffusers

A typical tube diffuser is either a rigid ceramic or plastic hollow cylinder (tube) or a flexible membrane secured by end plates in the shape of a tube. A tube diffuser has a media portion up to 2 m long. The thickness of the diffuser varies, but the outside diameter is approximately 60 to 75 mm. The various components of a tube diffuser are made of stainless steel or durable plastic. Threaded rods are used with ceramic or porous plastic. The rod is threaded into the feed end of the holder with a hexagonal nut secured on the rod to hold the assembly in place.

Air flows through tube diffusers are in the range of 1 to 5 l/s.

7.5.3.3 Dome diffusers

Made from ceramics or porous plastics, dome diffusers are typically circular, 180 mm in diameter, and 40 mm high. The media is about 15 mm thick on the edges and 19 mm on the horizontal or flat surface. The dome diffuser is mounted on either a polyvinyl chloride or a steel saddle-type base plate.

The airflow rate for dome diffusers is usually 0.5 l/s with a range of 0.25 to 1 l/s.

7.5.3.4 Plate diffusers

Plate diffusers are flat and rectangular, and typically approximately 300 mm square, and 25 to 40 mm thick. They are normally made from ceramic or membrane materials. Installation involves either grouting the plates into recesses in the floor, cementing them into prefabricated holders, or clamping them into metal holders. Air plenums run under the plates and supply air from headers. Plate diffusers have largely been replaced by porous discs, domes, and tubes in new installations.

7.5.4 PERFORMANCE

The performance of diffused aeration systems under normal operating conditions is directly related to the following parameters:

1. Fouling
2. Wastewater characteristics
3. Process type and flow regime
4. Loading conditions
5. Basin geometry
6. Diffuser type, size, shape, density, and airflow rate
7. Mixed liquor dissolved oxygen (DO) control and air supply flexibility
8. Mechanical integrity of the system
9. Operator expertise
10. The quality of the preventive operation and maintenance (O&M) program.

The wastewater characteristics that establish the oxygen demand placed on a fine pore aeration system are the influent flow rate, chemical oxygen demand (COD) load, and ammonia-nitrogen (NH_3-N) load.

The presence of constituents such as surfactants, dissolved solids, and suspended solids can affect bubble shape and size and result in diminished oxygen transfer capability. In general, ceramic domes and discs yield slightly higher clean water transfer efficiencies than typical porous plastic tubes or flexible sheath tubes in a grid placement.

Good ceramic fine bubble systems can give an oxygen transfer efficiency of 20 - 22% into clean water. However they are prone to clogging and in addition diffused air systems have low α factors (typically 0.4 to 0.6) which means proportionately lower efficiencies into mixed liquor (8 – 12%).

Porous plastic or membrane diffusers have lower transfer efficiencies into water in the range 12-18% and are more resistant to clogging but the same caution regarding α factors applies.

Simple orifices typically have a transfer efficiency of about 8 – 10% into clean water and about 4% into mixed liquor.

7.5.5 AIR FILTRATION

Good air filtration down to sub-micron particle-size is essential for fine bubble diffusers if clogging is to be avoided and long term acceptable aeration efficiency is to be maintained. It is essential that diffusers be kept clean through two or three stages of air filtration and that the system be maintained. Good preventive maintenance can virtually eliminate air-side (blower filtration system) particulate fouling of fixed fine pore diffusers.

7.5.6 COSTS

The aeration system consumes approximately 50 to 70% of the net power demand for a typical activated sludge wastewater treatment plant. Therefore, the designer is responsible for selecting a system that will meet the mixing and oxygen requirements for the process at the lowest cost possible. Once the requirements for aeration are determined, comparative costs for different types of aeration systems can be estimated and the final equipment configuration selected to best match the requirements of the job.

Construction cost items mainly consist of aeration basins, air piping and headers as appropriate, aeration devices and their supports, air cleaning equipment, blowers, and buildings to house these items.

O&M costs are primarily for power, cleaning, and replacement. Operational costs are determined in part by the oxygen transfer efficiency of the fine bubble aeration system being used, as well as the characteristics of the influent wastewater.

7.6 MECHANICAL AERATION

7.6.1 INTRODUCTION

Surface aerators can be grouped into four general categories:

1. Radial flow turbine aerators, low speed
2. Axial flow turbine aerators, high speed
3. Aspirating devices
4. Horizontal rotors.

Each is used widely and has distinct advantages and disadvantages, depending on the application.

FIGURE 7.5.1: Typical bridge-mounted surface aerator

Surface aerators are float-, bridge, or platform-mounted. Platform and bridge designs reduce torque and vibration problems. Bridges should be designed for at least four times the maximum moment (torque and impeller side load) anticipated. The aerator manufacturer can provide the magnitude of this moment.

The efficiency and power consumption of platform- and bridge-mounted aerators are sensitive to changes in the depth of impeller submergence. An increase in submergence results in increased load on the impeller and an increase in power consumption, and can decrease gearbox life expectancy.

High-speed (radial and axial) surface aerators are most often float-mounted, providing flexibility in spacing arrangements, the ability to increase oxygen input easily, and low cost.

Some surface aerators, usually for relatively deep tank applications, are equipped with submerged draught tubes that tend to improve mixing by bringing liquid from the bottom of the tank up through the tube and to the impeller. Draught tubes are used where tanks are deeper than 4 m, where mechanical aerators alone might not provide enough mixing throughout the entire tank. This feature adds to the cost of the system and will require additional power.

All the mixed liquor pumped through a draft tube is dispersed into the air. Without draft tubes, a portion of the pumped fluid flows beneath the liquid surface and is not aerated (doughnut aeration pattern). Mixed liquor pumped in either way creates liquid momentum that tends to circulate the mixed liquor directly around the aerator.

Mechanical aerators provide point-source oxygen input and the pumped mixed liquor flows radially outward from the aerator with decreasing velocity. Dissolved oxygen reaches its maximum near the impeller blade where surface turbulence is greatest and decreases as fluid flows back below the surface of the aeration tank toward the aerator.

The action of surface aeration devices, particularly splashing from high-speed units, can generate mists with attendant health concerns and nuisance odours. Odours can result from insufficient oxygen supplied by the aerator or influent wastewater containing sulphides or other volatiles. Splashing effects can be minimised with proper geometric design of the aeration tank and use of deflector plates.

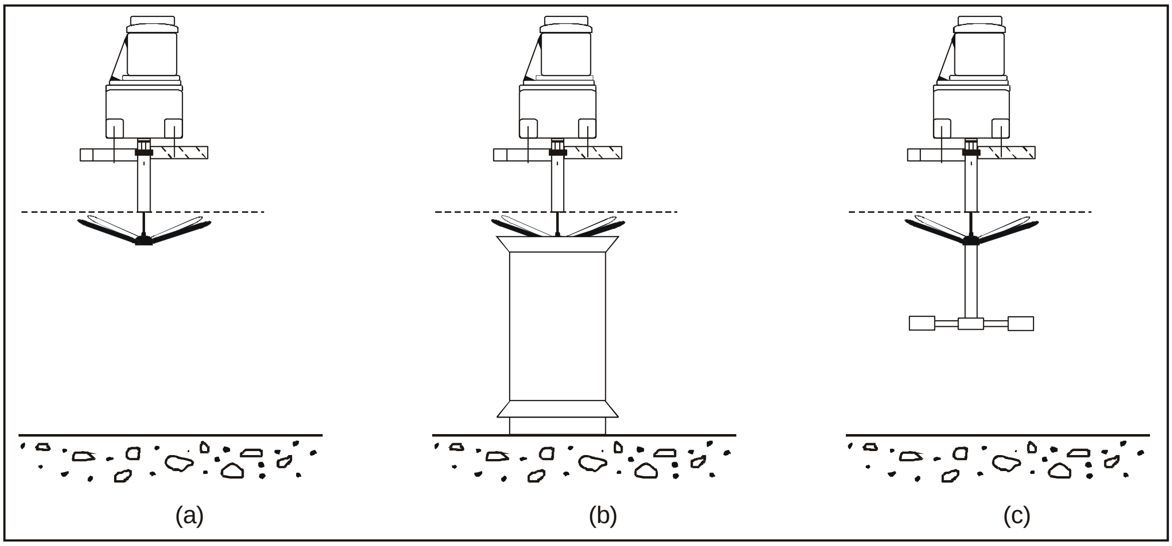

FIGURE 7.5.2: Some types of aerators: (a) standard surface aerator, (b) surface aerator with draft tube, and (c) surface aerator with impellor

7.6.2 SLOW SPEED TURBINE SURFACE AERATORS (RADIAL FLOW)

Slow speed surface aerators are probably the most common type of aeration in RSA. They are almost invariably used on small to fairly large plants from 1 Mℓ/d upwards to about 80 Mℓ/d where diffused air has not been selected as the (much less common) alternative. Diffused air becomes more common for really large plants of 100 Mℓ/d capacity and upwards, where the lower specific power consumption and lesser degree of mechanical complexity becomes significant.

Slow speed aerators are almost invariably bridge or platform mounted although there are designs for floating units of this type. The aerators consist of a motor, a two or three stage reduction gearbox, and an impeller which has a number of straight or curved axial vanes. The motor is usually a 1 450 rpm unit, and the gearbox reduces the shaft speed of the impellor down to about 45 to 80 rpm, with larger units tending to operate at slower speeds.

The power dissipation of this type of aerator can range from as low as about 2 kW to in excess of 100 kW (wire to water). Efficiencies vary with size, rotational speed, altitude and impeller design but are of the order of 1.7 to 2.2 kg O_2/kWh into clean water, and 1.2 to 1.4 kg O_2/kWh into mixed liquor under specified site conditions. A well-designed and maintained slow speed aerator with a gearbox service factor of at least 2.0 can have a service life of 15 to 30 years.

7.6.3 HIGH SPEED TURBINE SURFACE AERATORS (AXIAL FLOW)

High speed surface aerators are fairly common on small plants and are often used in aerated lagoons. The aerators are simpler in construction than slow speed aerators in that they usually do not have gearboxes and have impellers which are driven at motor speed. The motors are often six pole motors which rotate at a nominal 900 rpm.

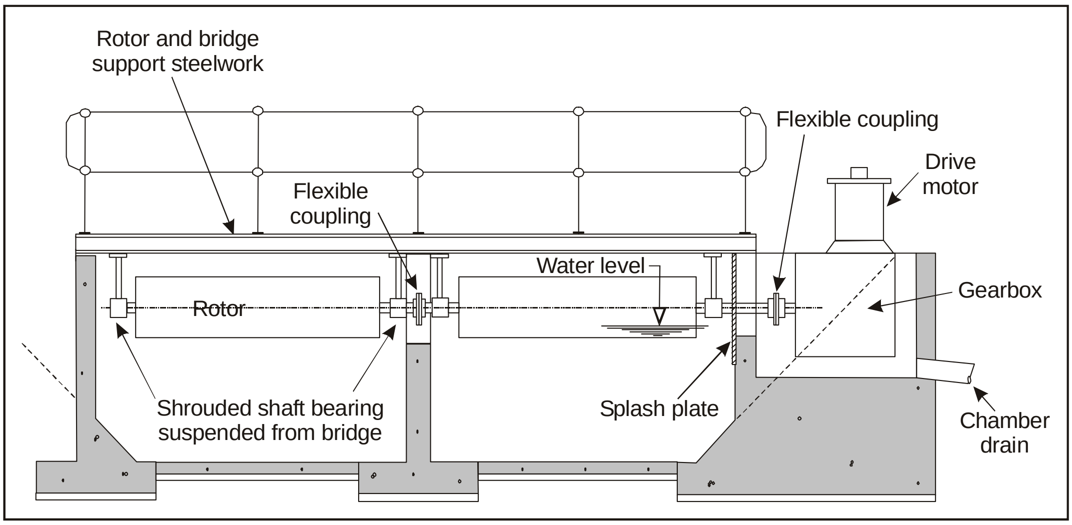

FIGURE 7.5.3: The layout of an horizontal shaft mechanical surface aerator

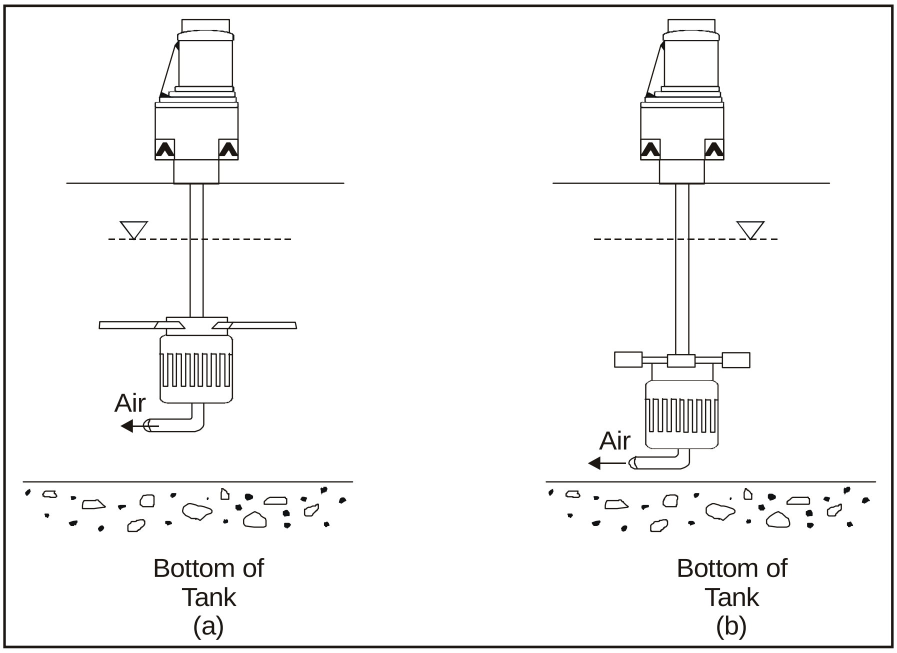

FIGURE 7.5.4: Submerged turbine aerators: (a) axial flow and (b) radial flow.

Most high speed aerators are float-mounted as they tend to be installed in plants where low cost and simplicity are important. They are not as robust as slow speed aerators and tend to have a shorter service life, typically 5 to 10 years, although there are units which have given better service.

These units range in size from around 1 kW to about 20 kW with most units being in the mid range. The transfer efficiencies are lower than for low speed units, being typically 1.4 to 1.7 kg O_2/kWh into clean water and 0.8 to 1.2 kg O_2/kWh into mixed liquor.

7.6.4 HORIZONTAL ROTORS

Horizontal rotors are normally installed on activated sludge plants of the oxidation ditch type. They can have a span of up to 3 or 4 m and on large ditches it is common to have two rotors side by side with a pedestal carrying the bearings of the non-drive ends of the rotors. The number of rotors installed on a ditch usually ranges for 2 or 3 up to 8 or 9.

The rotors consist of a motor (normally 1 450 rpm) and a gearbox which drives the rotor. The rotor usually consists of a rotating cage carrying several rows of paddles which both aerate the mixed liquor and also impart a forward motion. Excessive forward motion reduces the power and hence oxygen input to the water. For this reason many ditches have adjustable baffles to control the circulation rate in the ditch. As an alternative to paddles, the rotor shafts may also carry a large number of vertical discs with holes through them which also creates aeration and forward motion.

Horizontal rotors have motors ranging from a few kW up to about 20 kW. It is important to specify a high service factor for the gearboxes as the paddles impart a cyclic stress on the units due to the beating action on the water. The shafts are also prone to fatigue and need to be robustly designed of tubing rather than solid construction to minimise bending between the bearings. Well-designed units can have a satisfactory service life. The transfer efficiency is similar to high speed turbine aerators.

7.6.5 SUBMERGED AERATORS

Submerged aerators have characteristics of both mechanical aerators and diffused air in that the air is introduced by mechanical means as bubbles usually at the bottom of the aeration tank or pond. The design uses the higher solubility of oxygen at depth but requires more power to achieve the pressure head at such depths, so that there is no major advantage to the designs from a transfer efficiency point of view. The designs have the advantage of simplicity in some cases as they do not need floats or bridges for mounting and are therefore well suited to ponds, small, and temporary plants.

7.6.5.1 Submerged Turbine Aerators

A submerged turbine aerator is similar to a submersible pump on the floor of the tank with a pipe extending vertically to above the surface of the water through which air is drawn into the throat of the impeller. The air is discharged as a bubble cloud into the tank with the carrier water and aerates the tank contents. The efficiency into mixed liquor is of the order of 1kg O_2/kWh.

7.6.5.2 Venturi Aerators

The venturi aerator is similar in concept, but in this case the recirculating pump delivery passes through a venturi with air from the surface being drawn into the throat and again being discharged as a bubble cloud into the tank. Transfer efficiency is similar.

7.6.5.3 Jet systems

Jet systems, which are placed on the floor of aeration tanks, combine liquid pumping with air diffusion. The pumping system circulates mixed liquor in the aeration tank, ejecting it through a nozzle assembly. Air, supplied from a blower, is introduced to the mixed liquor before its discharge through the system’s nozzles.

7.7 MIXERS

7.7.1 INTRODUCTION

Mixers are used in a number of applications in wastewater treatment but most commonly in the activated sludge process where the mixed liquor solids in the anoxic or anaerobic compartments of the aeration tank need to be kept in suspension.

The choice of mixer should be made such that units which set up mixing patterns through relatively large volumes should be selected rather than mixers which dissipate their energy in localised areas. An ideal mixer would set up mixing currents which would keep all the mixed liquor solids in suspension throughout the compartment. This is not easily achieved and many designs use a number of small mixers rather than one large unit in order to achieve uniform energy dissipation throughout the tank.

There are a number of types of mixers which can be used. Turbine or paddle mixers were the typical choice for the application originally, and are still widely used and selected. An alternative which has become more attractive in recent years as mechanical reliability has improved is the use of submersible mixers. As a third possibility, the use of pumped mixing either by submersible pumps or external pumps is possible, and has been specified on a number of occasions due to experiences of mechanical unreliability of many mixer designs.

7.7.2 DESIGN

The objective of mixing is to circulate the entire contents of the compartment. If the tank is square or circular this can possibly be achieved by a single mixer. For a rectangular tank it would be more usual to have two or more mixers for the duty.

Mixer design is a specialised field of its own and expert help from experienced suppliers should be sought when specifying for a particular duty. However as a guide the equipment specified should comply with certain norms:

1. The amount of energy for mixing would normally lie in the range of 6 to 10 W/m^3
2. Energy inputs less than this could tend to allow settlement in the corners of the compartments and more than 12 W/m^3 would only be necessary for solids which settle quite rapidly
3. The mixers specified should produce mixing currents which can be directed and which have a wide sphere of influence. High mixing energy in localised areas would not achieve the required objective.
4. The mixers should be mechanically reliable and capable of operating for a service life of typically 5 to 10 years if not longer
5. Removal, servicing, repair and replacement of mixers should be readily possible without interrupting the process.

A number of principles have to be considered when selecting a mixer. To achieve bulk flow and mixing in a tank, a minimum thrust is required, which varies from one application to the next. Most mixer application failures are a result of insufficient thrust being generated in the tank to get the product into suspension. This leads to ineffective mixing.

Equipment which operates 24 hours a day, such as mixers in wastewater treatment works, needs to be highly efficient if the long-term costs are to be lowered. By analysing the kilowatt requirement in relation to the thrust requirement for the best mixing, the most efficient solution can be found and energy costs reduced.

7.7.3 IMPELLER MIXERS.

Impeller mixers are classified into three groups:

- Paddles.
- Turbines.
- Propellers.

The most common type of mixing device used for wastewater treatment is the rotating propeller mixer but paddle and turbine mixers are also used.

7.7.3.1 Paddle and Turbine Mixers.

Generally, turbine mixers resemble multi-bladed paddle mixers with short blades turning at moderate to slow speeds on a shaft typically located near the centre of the mixing chamber. The impeller may be shrouded, semi-enclosed, or open. The diameter of the impeller is typically 30 to 50% of the diameter of the mixing vessel.

Turbine mixers, especially in thin liquids, impart strong currents that persist throughout the vessel. Baffles or diffuser rings must sometimes be used to prevent vortexes. Turbine mixers are typically mounted vertically and in the centre of the mixing chamber, 50 to 100% of a diameter above the chamber floor.

7.7.3.2 Propeller Mixers

These mixers have high speed, low impellers and are generally used for thicker solutions. At full motor speed, small propeller mixers revolve at about 1500 rpm, whereas larger mixers turn at 375 – 750 rpm.

Typically, propeller mixers are much smaller in diameter than either paddle or turbine mixers, rarely exceeding 0.5 m in diameter, regardless of the size of the mixing vessel. Deep mixing vessels typically require two or more propellers on the same shaft. Where top entry is required for a propeller mixer, the mixer is mounted angled and off-centre.

7.7.4 SUBMERSIBLE MIXERS

Submersible mixers, which operate in a similar fashion to submersible pumps with submerged motor and impeller, are finding more applications in the wastewater management sector.

The placement and direction of the mixer are important. Effective placement is essential to optimise the mixer's performance and the solution is often not as obvious as it may seem. Given the mobile nature of a submersible mixer, positioning is made substantially easier and can be adjusted if the initial placement requires fine-tuning.

Submersible mixers are generally attached to a guide bar and hoist, which eliminate much of the civil construction associated with other types of mixers and offer related cost savings. Ease of insertion and extraction make maintenance and inspection easier, and the mixers can be adjusted from one site to the next, making it an easy retrofit option.

7.7.4.1 Pumped mixing

Pumped mixing can be achieved in two ways, either by submersible pumps at suitable points in the bottom of the tank, or by external pumps with a piping system and using jets or nozzles for discharge to set up the mixing currents.

Submersible pumps are lowest in cost as there is not much piping involved but the system is limited in that there will only be a small number of mixing jets (usually one per pump) as a large number of pumps becomes expensive. The delivery of the pumps should be set so as to give a coherent mixing pattern and aimed to reach the surface of the tank about halfway between pumps for opposed deliveries, or two thirds across the tank for parallel deliveries from one side of the tank.

External pumps are easier to service and maintain as they are external to the tank. It is possible to have duty and standby pumps, and the pipe-work can be arranged to give a relatively large number of jets at suitably close intervals in the tank. Generally a single row of jets is adequate and these are placed via down-comers at or near the bottom of the tank along one of the long sides and spaced at about 1 to 1.5 m intervals with the nozzles aimed upwards to reach the surface about two thirds of the way across the tank. The power dissipated by the pump should be about 10 W/m^3.

CHAPTER 8

8. FIXED-FILM AEROBIC PROCESSES

8.1 HIGH RATE PLASTIC MEDIA FILTERS

8.1.1 INTRODUCTION

The use of high surface area plastic media over the past twenty years has significantly improved the process performance and reliability of the traditional mineral media trickling filter for biological treatment of wastewater. Recent developments in filter design have evolved from a better understanding of the basic process fundamentals involved and from full-scale operational experiences. Improved performance is related to the need to enhance both the active biomass surface area within the filter and the associated substrate fluxes in and out of the biofilm.

Trickling filters, which can be more accurately described as non-submerged fixed film reactors (FFR's), have been used to remove organic materials from wastewater since the beginning of the last century. Operational simplicity, low energy requirements, and recent design innovations have led to a resurgence of interest in the application of FFR technology particularly to the treatment of industrial wastewater.

Improved performance of FFR systems has been related to a better understanding of the basic process fundamentals involved and an awareness of the importance of biofilm growth control to substrate removal.

During the last twenty years the development of lightweight plastic media with high surface area and voidage characteristics has significantly improved the range of organic loadings that can be applied to the media, together with the types of wastewaters which can be economically treated. This improved performance arises from the higher effective surface area for biofilm activity provided by plastic media. Of the fixed film treatment processes the high rate biofilter and the anaerobic filter are different from low rate aerobic processes in that they are not intended to produce an effluent of adequate standard for final disposal. The high rate biofilter is usually used as a pre-treatment process to reduce the COD of strong effluents before discharge to sewer, or as the first stage of a two stage aerobic process.

High rate plastic media trickling filters have been installed as the first stage of a two stage biofiltration plant, or to reduce the COD before a nitrifying activated sludge process.

There is no reason why high rate stone filters should not be designed and stone filters operating at high loadings have been reported in the literature. However plastic media is lighter so that the support structure is less expensive and has a higher voidage, permitting higher irrigation rates and hence higher loadings.

There are two types of plastic media commonly employed, the self stacking which exerts no side forces on the support structure, and the packed tower type which consists of patented rings with high surface area that are contained within the tower. The interfacial areas and performance of both types are similar.

8.1.2 PERFORMANCE

The performance of high rate biofilters is relatively insensitive to load at relatively low loadings when they tend to remove about 65% to 75% of the incoming COD or BOD. At high loadings the performance decreases with increasing loading until the point is reached where there is the onset of flooding conditions and/or deficiency in oxygen supply.

The filters produce humus sludge as normal at about 0.2 to 0.25 kg solids/kg COD removed. For high strength effluents intermediate sedimentation may be advisable. It is possible with more dilute effluents to pass the first stage effluent directly to the second stage biofilter without necessarily settling out solids generated in the first stage.

8.1.3 DESIGN

Because of their light weight and good ventilation, plastic biofilters can be constructed deeper than stone filters and media heights of 4 to 6 m are common and heights of 8 m to 12 m have been reported. The biofilters are usually circular in plan with four, six, or eight arm rotating distributors. The feed to the filter should be settled effluent or sewage, although fine mesh static or rotating screens of less than 0.5 mm gap have been used to prepare the incoming wastewater in some applications.

Loading limitations are both hydraulic and biological. Hydraulic loadings of about 30 to 40 $m^3/m^2.d$ are the usual norm, although many filters are designed to operate at about one third of this to roughly double this loading. A COD loading in the range 2-12 kg $COD/m^3.d$ is acceptable although most filters operate at less than about 6 kg $COD/m^3.d$. If the incoming COD concentration is greater than about 800 mg/ℓ it is advisable to recirculate the effluent to dilute the incoming stream.

The media usually has a specific surface area of about 100 to 120 m^3/m^2 so that this implies a COD load of about 20 to 70 g COD/ $m^2.d$. This contrasts with a rate of less than 10 g COD/ $m^2.d$ for low-rate filter

8.1.3.1 Air Requirements

The provision of air is a critical design feature in FFR performance. If adequate media voidage and underdrain area are provided, then the difference in temperature between the ambient air and air in the trickling filter will provide a natural draught which is often sufficient to provide the air for system performance.

However, in situations where there is little or no difference in air and wastewater temperature, periods of stagnant air flow can occur. Under these operational conditions the FFR will store organic material resulting in a subsequent oxygen lag phase when natural draught is resumed. The effect of these periods of stagnant air flow will depend on their frequency and duration, but could lead to biofilm accumulation and ultimately the development of anaerobic conditions within the media bed.

Ventilation also becomes limiting at higher BOD or COD loadings and results reported in the literature tend to indicate that forced-draught may be beneficial at COD loadings of greater than about 3.5 to 4kg COD/m^3.d. Beds operating at high loadings which are deficient in natural ventilation may therefore require forced-draught systems. Design of these is beyond the scope of this manual but design sequences are available in the literature as in Metcalf and Eddy (2003).

8.1.3.2 Recirculation and Hydraulic Loading

The feed onto a high rate plastic media trickling filter should not have a COD of greater than 800 mg/ℓ to 1 000 mg/ℓ. Effluents stronger than this should be provided with an effluent recycle to dilute the feed to bring it into an acceptable range. Recycle ratios of 0.5 to 1 up to 6 to 1 have been used but experience has shown that the additional benefits become limited at higher ratios. Once the media surfaces have been effectively wetted increasing the recycle rate further has minimal benefits.

High rate plastic media filters typically operate in the hydraulic loading range of 10 to 75 m^3/m^2.d
The minimum hydraulic application rate recommended by Dow Chemical from their pilot tests (WEF, 2000) is 0.5 l/m^2.s (about 40 m^3/m^2.d). Shallow tower designs require recirculation to provide minimum wetting rates above the minimum hydraulic application rate. Recirculation above this minimum rate provided little benefit. However there are many filters operating successfully at loadings around 25 to 40 m^3/m^2.d and there are also filters operating at hydraulic loadings up to 140 m^3/m^2.d

8.1.3.3 BOD and COD Loadings

Many of the design formulae are based on BOD loadings and concentrations. At the concentrations obtaining in high rate filters the COD/BOD ratios for raw sewage (usually about 2.0) will be reasonably applicable.

Germain applied the Schulze equation in 1966 to trickling filters with plastic packing (WEF, 2000) as follows:

$$\ln S_e/S_0 = -kD/q^n \qquad \text{...(8.1)}$$

where

S_e = BOD or COD concentration of settled filter effluent, mg/ℓ (g/m^3)

S_0 = influent BOD or COD concentration to the filter, mg/ℓ (g/m^3)

k = wastewater treatability and packing coefficient, (ℓ/.s)^{0,5} /m^2 (based on n = 0.5)

D = packing depth, m

q = hydraulic application rate of primary effluent, excluding recirculation (ℓ/m^2.s)

n = constant characteristic of packing used

The value for n is normally assumed to be 0.50 and pilot-plant or full-scale plant influent and effluent BOD concentration data are used to solve for k. Values for k were developed from more than 140 pilot-plant studies by Dow Chemical Company with vertical plastic packing with a specific surface area of about 90 m^2/m^3. Similar tests have been done by other suppliers for a variety of packings. Most of these tests have been done with packing depths of 6.1 to 6.7 m.

It should be noted that the clarification design and dosing cycle and method can affect pilot-plant results used to calculate a value for k (Daigger and Harrison, 1987). In summary, the value for k is affected by many factors, including the wastewater characteristics, filter and clarifier design, and operating conditions.

The commonly accepted temperature correction for k is as follows:

$$k_T = k_{20}\,(1.035)^{T-20} \qquad \text{...(8.2)}$$

Table 8.1.1 Normalised Germain equation k_{20} values from pilot studies

Type of wastewater	k_{20} value (ℓ/s)^{0,5} /m^2
Domestic	0.210
Fruit canning	0.181
Kraft mill	0.108
Meat packing	0.216
Pharmaceutical	0.221
Potato processing	0.351
Refinery	0.059
Sugar processing	0.165
Synthetic dairy	0.170
Textile mill	0.107

8.1.4 WORKED EXAMPLE

Task - Design a high rate plastic media filter as a first stage aerobic treatment stage for a small town with a cold drink factory which is putting effluent containing sugar solution to sewer. The mixture of domestic sewage and effluent has a COD of 2 000 mg/ℓ and a volume of 2000 kℓ/d. A COD of not more than 700 mg/ℓ is required for the feed to the secondary treatment stage.

<u>Trickling Filter Volume</u>

COD Load = 2000 x 2000/1000 = 4 000 kg/d

COD or BOD removal required = 100 – (700/2000) = 65%

Select a k_{20} value of 0,180 (mixture of domestic and sugar) from Table 7.1.1

Assume a temperature of 15°C

$k = 0.180 (1.035)^{15-20} = 0.152$

Select a filter depth of 6.0 m

$\ln 700/2000 = - 0.152 \times 6.0/q^{0,5}$

$q^{0,5} = 0.912/1.0498 = 0.8687$

$q = 0.9321$ (l/m^2.s) = 80.5 m^3/m^2.d

This is on the high side. A lower irrigation rate is desired bearing in mind that recirculation of about 1:1 is required at a COD of 2000 mg/ℓ

Select a filter depth of 4.0 m

$\ln 700/2000 = - 0.152 \times 4.0/ q^{0,5}$

$q^{0,5} = 0.608/1.0498 = 0.5792$

$q = 0.761$ (l/m^2.s) = 65.8 m^3/m^2.d

With recycle of 1:1 the irrigation rate will fall within the permissible range

Filter area = 2 000/65.8 = 30.4 m^2

Diameter = $(30.4/\pi)^{0,5} \times 2 = 6.22$ m

Volume = 30.4 x 4.0 = 121.6 (say) 120 m^3

8.2 BIOLOGICAL (TRICKLING) FILTERS

8.2.1 INTRODUCTION

Biological filtration (Trickling Filter) provides a unit process for wastewater treatment which does not require sophisticated technology and has relatively low power and operating costs. A biological filter (Biofilter) normally treats wastewater which has been settled in a primary sedimentation tank (PST) or a septic tank, and comprises a bed of media, which may be a granular material such as crushed stone, or synthetic packing media which may be either stackable or contained within a structure.

FIGURE 8.1.1: Typical trickling filter

Settled sewage is sprayed or trickled over the top surface of the bed, through which it percolates. The filter is aerated either by natural draft or by forced ventilation to provide oxygen to the purification process. Slime containing a large number of organisms forms on the surface of the media. As the sewage flows over this slime a series of complex bio-chemical reactions take place by which organic material is removed from the sewage. The organisms require oxygen and this is obtained from the air circulating in the filter bed. Hence maximum ventilation should be provided. The slime increases in thickness until eventually portions break away and are carried out of the bed in the effluent. This material is known as 'humus' which has to be separated from the effluent in a sedimentation tank. The effluent which has passed through the filter is directed to collector drains by slopes in the floor and conveyed to a secondary sedimentation tank (Humus Tank) where the solids are settled out. The clarified effluent is either discharged or partly recycled through the biofilter to improve performance.

8.2.2 PROCESS CONSIDERATIONS

8.2.2.1 General

Oxidation of organic matter is achieved by microorganisms living in the slime growing on the surface of the media. The active quantity of this bio-mass determines the work done in a filter. For a given quantity of bio-mass the performance of a filter will depend largely upon:

1. The amount of organic matter fed to the bio-mass (loading rate) .
2. The maintenance of an adequate oxygen supply.

3. The temperature.

8.2.2.2 Micro-Organisms

While filter performance has generally been related to filter volume, the quantity of micro-organisms present is primarily dependent upon the surface area of the media in the filter. The specific surface area varies considerably with the size and type of material used. Synthetic media tend to have higher specific surface areas than stone and for this reason filters with plastic media can be loaded at higher rates. The removal of organic load appears to be dependent upon the surface area of the bacterial slime available to the liquid, and the surface area of the media is representative of this factor.

Thick films tend to clog the filter and promote the development of anaerobic layers adjacent to the media surface, thus reducing the microbial efficiency. For a particular filter, maximum efficiency is achieved with thin films. High hydraulic loadings assist to this end, provided the organic load is not unduly increased thereby. This points to the reason why filters using high recirculation ratios usually perform significantly better than filters without recirculation.

While a certain surface area of media may be provided in a filter, only that portion of it which is wetted by the applied liquid can be utilised. For best performance, therefore, a filter should be fully wetted. Therefore below a certain value, peculiar to the media used, a low rate of hydraulic loading may limit filter performance.

Different types of bacteria undertake the oxidation of carbonaceous and nitrogeneous material. Nitrification can only take place in the presence of appreciable concentrations of dissolved oxygen and therefore only commences once a certain degree of carbonaceous oxidation has taken place. Nitrifying bacteria thus appear in the lower levels in a filter or in the secondary stage and can be displaced by high loadings on a filter.

8.2.2.3 Ventilation

Ventilation of the filter to convey oxygen to the bacteria is of prime importance. Filters up to 4 m deep have been found to have adequate self-ventilation without openings at intermediate depths. It is important that an unrestricted flow of air is provided throughout the filter and attention to the number and size of ventilation ports is necessary.

Natural ventilation is affected by one or more of the following:

1. The elevated temperature of sewage causes the biofilter to be warmer than ambient and hence provides a convective chimney-stack effect for air to move upward through the media.
2. Winds sweeping across the surface of a filter have a venturi suction effect which again tends to move air upwards through the media.

3. The flush of water downward through the filter tends to drag air with it. This factor will of course oppose the two previous effects and may at times predominate.

8.2.2.4 Temperature

As the metabolic rate of the bacteria is temperature-dependent, lower temperatures reduce the efficiency of biofilters. The effect of temperature is not straightforward, and is more marked in nitrification than in carbonaceous oxidation. For a 40 mm broken rock media it has been reported that the BOD removal reduced from 94% to 89% for a drop in sewage temperature from 17°C to 9°C. At the same time the ammonia removal was reduced from 70% to 15%. Filters with a lower organic loading are less susceptible to temperature effects than those where the load is heavy in relation to specific surface area of media.

8.2.2.5 Void Ratio

Whether using granular or synthetic media the desire for high efficiency is likely to lead to the choice of material of high specific area. However this has the consequence that void space is reduced, and with granular material of 40 mm size and smaller, or fine plastic media it is possible, particularly with strong wastewaters, for the bio-mass growth to be such that it blocks the path of the liquid through the media, thus causing the filter to pond. Choice of media must therefore be a compromise between maximum surface area and maximum void space.

High hydraulic loading such as the use of high recirculation ratios may assist in preventing ponding and in increasing filter efficiency by removing excess bio-mass and maintaining a thin film on the media. However, with strong wastewater it is generally necessary to select media with a high percentage of voids.

8.2.2.6 Recirculation

Recirculation of effluent is a useful method of applying high hydraulic loads to filters, particularly where strong sewages are being treated, thereby fully wetting the media surface and scouring the bio-mass. Where high strength sewage is applied to a filter, recirculation may also result in denitrification, with the release of nitrogen gas to the atmosphere. It was formerly accepted that recirculation ratios of about 0.25 to 0.5:1 were most suitable and recirculation ratios seldom reached or exceeded 1:1. More recent experience has shown that recycle ratios of up to 3:1 often have significant benefits. If humus tank effluent is recycled back to the biofilter the recycle ratio must be incorporated into the hydraulic design of the humus tank which significantly increases tank size and costs. It is therefore most economical to recycle biological filter effluent from upstream of the humus tank and add it to the feed after the PST.

8.2.2.7 Single and Double Stage Filtration

Previously double-stage filtration was used where single-stage filtration would not produce an effluent of adequate quality. The term was normally applied to two stages of stone media filters and sometimes the two stages were identical and capable of being alternated as first and second stage. One would choose double filtration if excessive bed depths were required for single stage filters which could be a cause of ventilation problems. With the more recent development of synthetic plastic media, which are capable of being operated at high loadings, it is now more usual to specify two-stage filtration with a high rate plastic media filter as the first stage.

In both these methods of operation, provided adequate volume of media is available, highly nitrated effluents may be obtained. Where a wastewater of low alkalinity is to be treated (such as is common in SA's coastal regions) a corrosive effluent may result. This can be ameliorated to some extent by the use of recirculation but in many plants the addition of lime to increase alkalinity is practiced. Where two-stage or double filtration is used, intermediate humus removal is desirable but not essential.

8.2.3 DESIGN PARAMETERS

8.2.3.1 Load Assessment

The load applied to a filter is the rate at which substrate or nutrient is applied to the bio-mass on the media and may be expressed as the product of strength and volume. This is expressed as grams COD or BOD per cubic metre per day based on ADWF.

8.2.3.2 Design Loadings

Table 8.2.1 below is a guide to loading rates for biological filters to provide an acceptable quality effluent, based on the following conditions:

1. Winter conditions implying one month in which the monthly average sewage temperature is 15°C
2. Media is crushed rock of 40 mm to 63 mm size
3. Analysis is of composite samples
4. The COD of a composite sample of settled sewage taken at regular intervals through the day does not exceed 750 mg/ℓ
5. Recirculation of at least 1:1 is practised.

Higher temperature, smaller size stone (within limits) and lower strength sewage are all factors which may permit higher loadings to be used. However, the higher the quality sought in the final effluent, the less this latitude in loading becomes. It should be noted that wide variations in the performance of biological filters at similar organic loading rates occur and that the values cited in **Table 8.2.1** and **Table 8.2.2** are only indicative.

Loading rates for biological filters in areas not subjected to severe winter conditions, where for example the minimum monthly average sewage temperature is 18°C or above, may be increased by between 25 to 40% of the figures given in the **Table 8.2.1** above. Biofilters are not efficient in ammonia removal in winter and even at these loadings may not comply with the DWAF General Authorisation limit for ammonia

TABLE 8.2.1: Guide for loading rates of biological filters

Filter Depth and Mode of Operation	Design Loading g/m3.d	
	COD	N/NH_4
2 M single	200	16
2 M double	250	20
4 M single	250	20
4 M double	350	30

8.2.3.3 Dosing Siphon and Tank

See Section 8.2.4.6.

8.2.3.4 Distributors

The distributor should be designed for the PWWF plus recirculation at the design ratio.

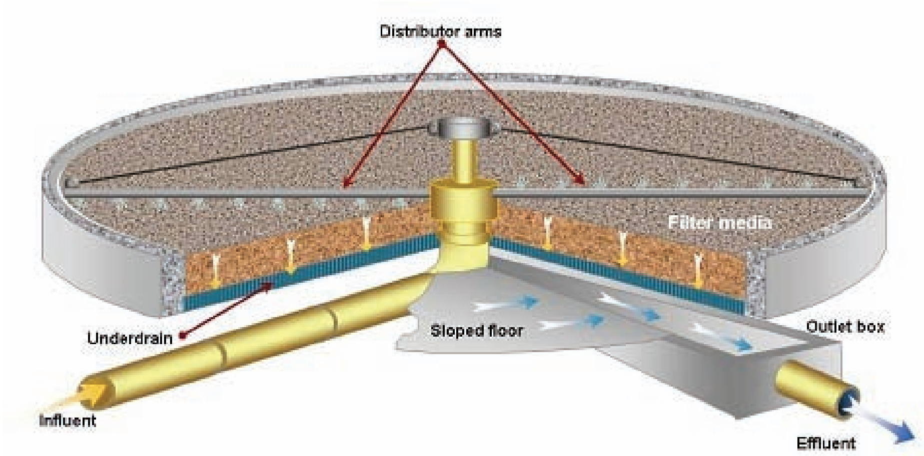

FIGURE 8.2.1: Schematic of a biological trickling filter (eWISA)

8.2.4 PRACTICAL CONSIDERATIONS

8.2.4.1 Granular Filter Media

The choice of media is generally limited by what is available locally. For maximum treatment per volume of filter, hard strong media with a high specific surface should be selected provided the void ratio is not unacceptably reduced. Specific surface areas and void ratios have not been measured in the past so that performance in relation to these parameters is not readily available. The current parameters, which are empirical, are the surface characteristics of the media and its size. It is thought that for a given size and a given loading, a media with an irregular and rough surface will produce a better effluent than one with a comparatively smooth regular surface.

The following nominal sizes are recommended in works where no special strength problems arise:

1. Single stage filtration : 50 or 63 mm
2. Double stage filtration - Primary Filter : 50 or 63 mm, Secondary filter: 40 mm

Uniformity of media has been stressed in the past. However, some small range of grading may be permissible in giving additional media surface. **Table 8.2.2** below, based upon British Standard Specification 1438:1971 "Media for Biological Percolating Filters", is presented as a guide.

TABLE 8.2.2: Media for biological filters from BSA 1438:1971

BS 410 Square Hole Perforated Plate test Sieves	Nominal size % by mass passing		
	63 mm	50 mm	40 mm
Passing 75 mm	100	-	-
Passing 63 mm	85 -100	-	-
Passing 50 mm	0 - 35	85 - 100	100
Passing 37.5 mm	0 - 5	0 - 30	85 - 00
Passing 28 mm	-	-	0 - 40
Passing 20 mm	-	-	0 - 5

Media should be substantially free of fine material, particularly clays, and organic matter.

8.2.4.2 Plastic Media

A number of proprietary makes of plastic media are on the market, the characteristics of which should be ascertained from the manufacturers. These generally have a very high void ratio and therefore have less of a tendency to pond when strong wastes are treated on them. The economics of the use of materials should be carefully considered. Experience has indicated that effluent treated on plastic filter media will not generally nitrify but the use of plastic media is well suited to high rate primary filters operated at high loadings.

8.2.4.3 Filter Aeration

An important feature in the design of biological filters is the provision for adequate aeration. This can be achieved by covering the filter floor with suitable aeration tiles, with provision for air to enter the tiles from a central channel, inlet chambers or through holes in the filter wall. Vent pipes inside the filter in lieu of holes in the wall are undesirable but can be useful in supplementing air supply.

8.2.4.4 Distributors

The distributor should be so designed as to distribute sewage evenly over the filter surface. Particular care should be taken to ensure that the distributor rotates at all times when sewage is being applied. If flow is liable to be variable or intermittent a dosing siphon should be used to achieve this end.

Reaction-driven distributors frequently operate with a rotational speed of 40 to 60 seconds per revolution, but slower speeds in the range 75 to 300 seconds per revolution have been found in practice not to have materially affected effluent quality. The materials used for the centre column, distributor arms, etc., should be durable and non-corrodible.

8.2.4.5 Depth of Filter

Biological filter plants have the advantage of being able to operate without the use of electric or diesel power. Thus, where possible, the design of the works should be based on gravity flow and this may determine the depth of filter with 2 m depth being the minimum. However, deeper filters are more economical as will be seen in **Table 8.2.1** of design loading rates.

8.2.4.6 Dosing Siphon Tanks

Dosing siphon tanks accommodate the siphon which will generally be supplied with the distributor. The siphon should be of a durable non-corrodible material. In cases where the initial daily flow is only a fraction of the design ADWF, the retention time in the dosing tank may exceed the recommended one hour. In such a case it would be necessary to provide a temporary wall within the tank to reduce the tank volume.

8.2.4.7 Potential Corrosion

A high degree of nitrification in effluent with a low alkalinity may be the cause of severe corrosion. This may be overcome by either providing for denitrification by recirculation, or by adding lime to raise the alkalinity.

8.2.5 WORKED EXAMPLE

Task **-** Design a biofilter plant for a 10 Mℓ/d works treating a strong sewage with a COD of approximately 800 mg/ℓ.

Process Selection

The wastewater is strong and would be unlikely to achieve suitable effluent quality in a single stage of treatment. Therefore select a two stage biofiltration process with the first stage comprising a high rate plastic media filter, and the second stage a stone media biofiltration process with recirculation.

Biological load

COD of wastewater = 800 mg/ℓ

Assume ammonia (TKN) concentration of 12% of COD - Concentration = 96 mg/ℓ

Assume 35% COD reduction through primary sedimentation – Settled COD = 520 mg/ℓ

COD load = 10 x 520 = 5 200 kg/d

Ammonia load = 10 x 96 = 960 kg/d

High Rate Plastic Media Biofilter

Select COD loading of 4 kg COD/m^3.d

Filter volume = 5200/4 = 1 300 m^3

Select filter height of 6 m

Area of filter = 217 m^2 - Diameter = 16.6 (say) 17.0 m

COD strength is low enough not to need recirculation

Check irrigation rate = 10000/217 = 46 m^3/m^2.d

This is over the recommended limit of 30 m^3/m^2.d so the filter area needs to be increased and the height reduced.

Select filter height of 4 m

Area of filter = 325 m^2 - Diameter = 20.3 (say) 20.0 m

Check irrigation rate = 10 000/325 = 30.7 m^3/m^2.d which is OK

Hence a single high rate filter of 20 m diameter and 4.0 m height without recirculation is specified

Low Rate Biofiltration

Assume 65% COD removal in high rate filter and no ammonia removal

COD load = 5 200 x 0.35 = 1 820 kg/d

Ammonia load = 960 kg/d

Select a COD loading of 250 g/m^3.d and an ammonia loading of 20 g/m^3.d for single stage filtration on 4 m deep filters from Table 7.2.1

Filter volume based on COD = 1 820 x 1 000/250 = 7 280 m^3.

Filter volume based on ammonia = 960 x 1000/20 = 48 000 m^3

Had the loadings for double filtration been selected, the filter volumes would have been 5 200 m^3 on COD, and 32 000 m^3 on ammonia. Had the ammonia/COD ratio been assumed to be 10%, the filter volume on ammonia would then have been 26 700 m^3. For a sewage of normal strength (COD of

600 mg/ℓ and 10% TKN) the filter volume would further reduce to 20 000 m^3. Thus, depending on the assumptions made, the biofilter volume can range between 20 000 and 48 000 m^3 which has huge cost implications and illustrates the extreme sensitivity of the biofilter process to the requirement for nitrification. It also shows the dangers inherent in designing on COD load alone. In practice one would probably put in four biofilters of 4 m depth with a total volume of about 25 000 m^3 and even this would be an expensive option. This would comprise four filters of 45 m diameter and 4.0 m depth.

8.3 ROTATING BIOLOGICAL CONTACTORS (RBC)

8.3.1 INTRODUCTION

The Rotating Biological Contactor (RBC) belongs to the family of aerobic biological attached-growth (fixed film) wastewater treatment systems. The RBC may also be referred to as a Rotating Disc Process, Rotating Biological Filter and Rotating Biological Surfaces.

FIGURE 8.3.1: Rotating bio contactor

The RBC is in effect an oxygen mass transfer device and generally consists of a large number of circular discs spaced uniformly along the length of a shaft. The shaft is mounted across a trough above water level such that approximately 40% of the disc area is submerged in the wastewater to be treated. The discs are usually between 1 and 3.5 m in diameter and are manufactured of a material, or in a manner, which gives a large surface area to void ratio. The contactor or rotor is rotated at between 1 to 5 rpm.

A biomass film 1 to 4 mm thick develops on the discs. As the contactor rotates, a film of wastewater attaches to the biofilm and is carried through the air resulting in aeration of bio-film and wastewater, and biological breakdown of the compounds in the wastewater thereby reducing COD and ammonia. Shearing forces eventually cause excess biomass to be stripped intermittently from the rotating discs. To produce a clear effluent the sloughed biomass is removed in a sedimentation tank, and the final effluent is chlorinated.

RBCs are usually supplied in standard sizes with fixed surface area to trough volume for each particular disc diameter, and are ideally suited to small plants treating domestic sewage. In general RBCs are used in conjunction with septic tanks, in which case the latter precedes the RBC. RBCs are generally used to treat settled sewage either from septic tanks, or from primary sedimentation tanks (PST's), and may also be used in conjunction with pond systems.

8.3.2 PROCESS CONSIDERATIONS

8.3.2.1 RBC with Septic Tank

When an RBC is used in conjunction with a septic tank, the septic tank and RBC form a unit and the operation of the former has an effect on the performance of the latter. Some proprietary designs are offered which incorporate the RBC unit into a single structure together with the septic and final sedimentation tanks. For control purposes it is considered preferable if the septic tank is physically separated from the RBC. Each process unit should then be considered individually, as well as in combination.

The septic tank serves to remove, retain and partially stabilise floatable and settleable solids introduced with the raw sewage and humus sludge. The latter is removed from the RBC effluent in the final sedimentation tank, and is normally returned to the septic tank. The return of humus sludge may result in limited denitrification. This can be further stimulated by recycling of the effluent from the final sedimentation tank to the septic tank by increasing the underflow rate, and also generally improves the treatment efficiency. If sufficiently conservatively designed the effluent after chlorination generally complies with the General Authorisation limits.

8.3.2.2 Effect of Influent Substrate Concentration and Flow Rate on RBC Performance

RBC's are more sensitive to load than most other processes. For a fixed influent substrate concentration the effluent substrate concentration increases almost linearly with increases in flow rate while the percentage substrate removal decreases accordingly.

For a fixed flow rate the effluent substrate concentration will increase almost linearly with an increase in influent substrate concentration. At low flow rates the percentage substrate removal is relatively independent of the substrate concentration but at high flow rates the percentage substrate removal decreases as the influent substrate concentration increases.

Because of this linearity between flow rate and effluent substrate concentration the term 'hydraulic loading', defined as the influent flow rate divided by the total wetted contactor surface area, has been used for the design of RBC's as well as the more usual BOD or COD loadings.

8.3.3 DESIGN BASIS

8.3.3.1 Septic Tank

The sizing of septic tanks is usually based on the frequency of de-sludging and the daily per-capita contribution. For an RBC system the size of septic tank should be calculated similarly. As for normal septic tanks, a minimum retention period of 24 hours at ADWF should be allowed for in the septic tank excluding the sludge and scum accumulated.

The septic tank should be designed according to the criteria given in the relevant section of this manual (Chapter 4).

8.3.3.2 RBC Rotors

The RBC rotors can be designed on hydraulic loading but this assumes an average COD or BOD concentration in the sewage feed. As the biological loading (COD and ammonia) is limiting it is preferable to size the units on a biological loading basis.

8.3.3.3 Humus Tank

The sludge settles at similar rates to other fixed film processes and the design criteria given in the relevant section of this manual are applicable.

8.3.3.4 RBC Design

Organic loading is the primary design parameter for the RBC process. This is generally expressed as the organic loading per unit of media surface area per unit of time, (kg COD per m^2 of disc area per day). Wastewater temperatures above 14°C have minimal affect on organic removal and nitrification rates. However, below 14°C various correction factors must be utilised to determine the needed additional media surface area.

In determining design loading rates on RBC's, the following parameters are relevant:

1. Design flow rates and primary wastewater constituents
2. Total influent COD concentration
3. Soluble influent COD concentration
4. Percentage of total and soluble COD to be removed
5. Wastewater temperature
6. Primary effluent dissolved oxygen
7. Media arrangement, number of stages and surface area of media in each stage
8. Rotational velocity of the media
9. Retention time within the RBC tank(s)
10. Influent sulphide concentrations

11. Peak loading.

In addition to the above parameters, loading rates for nitrification will depend upon influent DO concentration, influent ammonia nitrogen concentration and total Kjeldahl nitrogen (TKN), diurnal load variations, pH and alkalinity, and the allowable effluent ammonia nitrogen concentration.

8.3.3.5 Loading Rates

When the peak to average flow ratio is 2.5 to 1.0 or less, average conditions can be considered for design purposes. For higher flow ratios, flow and load equalisation should be considered. For average conditions the allowable design loading should not exceed 30 g COD per m^2 (some criteria allow as high as 100 g COD per m^2) of disc media surface per day on the first stage shaft(s) of any treatment train. High density media (closely spaced discs) should not be used for the first stage RBC's.

For peak conditions, the design loading shall not exceed 24 g COD per m^2 of disc media surface per day for the first high density media shaft(s) encountered after the first two shafts or rows of shafts in a treatment train.

For average conditions for complete nitrification the overall system loading shall not exceed 6 g of soluble COD per m^2 of disc media surface per day (or say 10 g of total COD per m^2 of disc media surface per day). This soluble (filtered) COD loading to all shafts should be used to determine the total number of shafts required.

Note:

1. The above criteria are based on US practice and local experience in meeting the General Authorisation limit for ammonia. UK criteria allow up to 14 g COD per m^2 of disc media surface per day. As disc area is expensive one can design at loadings slightly above 10 g of total COD per m^2 of disc media surface per day using the UK criteria but in such a case conservative sizing of the septic tank is recommended and provision should be made for recycle of secondary effluent for balancing peaks and for denitrification.
2. For domestic sewage the criteria of 10 g of total COD per m^2 of disc media surface per day is roughly equivalent to the alternative criteria recommendation of 1 g ammonia per m^2 of disc media surface per day.

8.3.3.6 Effect of Staging

The arranging of discs in a series of stages has been shown to increase treatment efficiency significantly. Different microbial biomass develops in each successive stage, adapted to the changed composition resulting from successive stages of treatment. Carbonaceous oxidation of biodegradable COD occurs at the initial stages followed by nitrification in the latter stages.

The RBC process exhibits, within limits, first order kinetics. The improved residence time distribution obtained with staging increases the substrate removal rate. When more than two stages are provided the required wetted area could be slightly reduced as shown in the **Table 8.3.1** below:

TABLE 8.3.1: Reduction in wetted area of RBC when using more than two stages

No. of Stages	Correction Factor
3	0.95
4	0.90
More than 4	0.86

8.3.3.7 Effect of Population, Size and Flow Variation

The design relationships given above have been determined for steady state conditions. If a pre-determined constant quality effluent is required (such as the General Authorisation limits) provision should be made either to balance the flow and the load by suitable recycling or a correction factor should be used for sizing the RBC. Suggested correction factors used for domestic sewage are given in **Table 8.3.2** below:

TABLE 8.3.2: Suggested correction factors for design of RBC's for domestic sewage

Population	Correction Factor
400	1.3
400-1500	1.3 to 1.1
1500-5000	1.1 to 1.0

Note:
Because of the balancing effect of septic tanks these factors should be used with circumspection on properly designed septic tank/RBC systems.

8.3.3.8 Effect of Temperature

The temperature of sewage has little effect on RBC performance between 14°C and 30°C. A correction factor for cold areas could be used as shown in **Table 8.3.3** below.

TABLE 8.3.3: Suggested correction factors for design of RBC's for domestic sewage (cold areas).

Temperature °C	Correction Factor
14	1.00
12	1.05
10	1.15
8	1.30
6	1.40

8.3.3.9 Effect of Rotational Speed

The rotational speed is one of the few variables the designer can control. When this is increased, the percentage substrate removal increases up to a limiting value determined by the particular RBC under consideration. It has been reported that the percentage substrate removal increases with the rotational speed raised to the 0.1 power, while the corresponding power required to rotate the contactors increases at an exponential rate. As it is difficult to find a common basis for comparing the effect of different rotational speeds on different diameter contactors, almost all RBC process studies have attempted to maintain speeds of between 1 and 5 rpm, with little improvement noted at higher speeds.

8.3.3.10 Empirical Design Formula

The design parameters given above will enable an efficient design to be achieved. As a design check, the following formula should ensure that a septic tank/RBC system treating domestic sewage will produce a satisfactory effluent:

Disc area per capita = 7 m^2 per person.

8.3.3.11 Recycling

Denitrification is often required to ensure complete nitrification, particularly on the Eastern and Southern Coastal seaboards and elsewhere where the potable water is of low alkalinity. With a septic tank/RBC system this may be achieved by recycling from the humus tank using the underflow line at about 0.5:1 through the septic tank with the recycle pump being controlled by a 24 hour timer. Suitable programming also assists in reducing peak load factors on the RBC discs.

8.3.3.12 Practical Considerations

A simple inlet works as described should be provided on larger installations although grit removal is not essential with septic tanks.

Extended power failures may cause drying of exposed sections of the discs resulting in an imbalance. On subsequent start-up this may cause excessive torque resulting in gear stripping. Mechanical and electrical equipment should be sized accordingly.

RBC's are relatively capital intensive and are therefore more suited to steady sewage flow situations.

8.3.4 WORKED EXAMPLE

Task **-** Design an RBC plant for a housing estate with a projected population of 1 000 persons with an effluent to comply with General Authorisation limits

Load Estimation

Assume a per capita contribution of 200 l/person per day (Chapter 2.3)

Volume of wastewater = 1 000 x 0.2 = 200 kℓ/d

Assume a per capita COD contribution of 130 g COD/person per day (Chapter 2.4)

COD load = 1 000 x 130/1 000 = 130 kg/d

Assume an ammonia content of 10% of COD (ex Table 2.2.1)

Ammonia load = 130 x 10/100 = 13 kg/d

Process Selection

Select a process comprising a septic tank followed by the RBC rotors, followed in turn by a sedimentation (humus) tank to settle out humus solids. Humus tank underflow to be returned to the septic tank, together with additional effluent recycle for denitrification and flow balancing purposes.

Septic tank Sizing

Using the formula given in Section 4.1.3.2 and allowing for 6 months intervals between desludging of the septic tank the septic tank volume is given by

$V = 1\,000(0.2 + 0.1 \times [0.5]^{0,5}) = 270\ m^3$

Select a two compartment tank with the first compartment twice the volume of the second compartment and a depth of 2,2 m.

Tank dimensions = 19.2 x 6.4 x 2.2 with first compartment = 12.8 x 6.4 x 2.2 and second compartment = 6.4 x 6.4 x 2.2

The tank should be fitted with the normal appurtenances as set out in Section 4.1.4

RBC Sizing

Assume 35% COD Reduction through septic tank

COD load onto RBC = 130 x 0.65 = 85 kg/d

Ammonia load = 13 kg/d

Select COD loading of 10 g/m^2 of disc per day

Select ammonia loading of 1 g/m^2 of disc per day

Disc area calculated on COD = 85 x 1 000/10 = 8 500 m^2

Disc area calculated on ammonia = 13 x 1 000/1 = 13 000 m^2

As is the case for biological filters the ammonia oxidation criteria are more stringent and result in a larger plant. With effective recycling and warm winters it might be possible to specify a 10 000 m^2 plant, but for climates with cold winter temperatures the larger plant is safer.

Selecting 3 m discs - surface area per disc = $\pi \times 3^2/4 \times 2$ = 14 m^2

Number of discs = 13 000/14 = 929 discs

One would probably specify 4 rotors with 240 discs or 6 rotors with 150 discs

<u>Sizing of Humus Tank</u>

Assuming that the peak flow reduces through the septic tank and rotor banks to a factor of less than 2,0 it will be safe to size on average flow.

Select a surface loading (upflow rate) of 1.0 m/h on average flow (Section 6.1.4.10)

Tank area = (200/24)/1.0 = 8.33 m^2

Tank diameter = $2 \times (8.33/\pi)^{0,5}$ = 3.26 (say) 3.3 m

The tank should be set out according to the provisions of Section 6.1.4

8.4 SUBMERGED MEDIA REACTORS

8.4.1 INTRODUCTION

Submerged media reactors are fixed film reactors where the support media are submerged in the effluent being treated and oxygen to the biofilms on the support media is supplied by aeration of the tank contents. The process can be applicable on large or small scale plants, but in this country is most widely used in package plants for small scale applications. The process provides a secondary aerobic treatment stage for sewage which has been pre-settled in a sedimentation tank, or on small plants more usually in a septic tank.

The effluent from the settling tank or septic tank is then conveyed to the aerobic reactor(s). The reactor is either an upflow or downflow (co- or countercurrent to the air) unit with submerged fixed media which is designed for a specific residence time, circulation rate, and biological loading. Air is introduced via coarse or fine bubble diffusers, or eductors or venturis to maintain a sufficient oxygen level in the reactor. The media may be either of the stackable self-supporting type or proprietary random pack rings.

The beds of media, onto which the biofilm adheres, create an environment which facilitates contact between the biomass and the carbonaceous material, and has a high tolerance to shock loads. Most suppliers claim a higher biomass per unit volume than from activated sludge but this is not always supported by the literature where the published figures for both types of plant are similar.

The reactor(s) are then followed by a settling or humus tank to settle out solids sloughed off the media, and the final effluent is then usually disinfected and discharged.

FIGURE 8.4.1: Typical submerged media reactor

8.4.2 DESIGN

The package type plants are usually constructed using standard sized HDPE tanks or similar. This is not favoured for the septic tank anaerobic stage where a single two-compartment concrete tank is preferred to a multiplicity of smaller plastic tanks. However for single house or very small housing complexes a design compromise may be necessary.

For the aerobic stage a plant with two or three plastic tanks or compartments may in fact be preferable. There is evidence in the literature that nitrification organisms only establish in the biological slimes attached to the media when the COD has been reduced to less than about 50 mg/ℓ. As the carbonaceous organisms predominate in the early stages of treatment, the provision of a three stage plant with the third tank adapted to nitrifiers has significant benefits as regards effluent quality. Two lightly loaded tanks can work satisfactorily, but single aerated media tanks are often unsuccessful. Again however, for very small (single house) installations, a design compromise may be necessary.

The package plants are also fortuitously often designed with side stream venturis to draw in air to swimming pool type recycle pumps. Thus when supplying air to the reactors, one is also introducing a recycle stream that recycles the contents of a particular tank twice or three times per hour. This has significant process benefits. For larger plants or tanks a suitable blower with coarse or medium bubble diffusers is recommended together with a pumped recycle. As the tanks are seldom very deep the transfer efficiencies are low. For design purposes it is recommended that an oxygen transfer efficiency of no more than 3 to 4% be used with the usual estimation of oxygen requirements.

There is very little published information on design loadings for plants of this type. However, reasonably good design criteria can be obtained from other fixed film processes. A COD loading of about 4 to 6 g COD/m^2.d and an ammonia loading of not more than 1 g/m^2.d are recommended. For typical media with a specific surface area of 110 m^2/m^3, this relates to a COD load of 450 to 650 g COD/m^3.d, and an ammonia load of 100 g/m^3.d.

The humus tank should be sized according to normal criteria. The sludge yield from this process is about 0.15 to 0.2 kg ds./kg COD. The underflow from the humus tank should be sized for about 0.5:1 relative to feed and operated on a timer control to recycle effluent back to the septic tank. This assists in balancing peak flows and loads and stimulates denitrification which assists in conservation of alkalinity.

Monitoring of a number of plants has shown that plants sized to the above criteria with suitably set recycles will usually meet the South African COD and ammonia discharge standards.

CHAPTER 9

9. SLUDGE TREATMENT

9.1 INTRODUCTION

In a wastewater works there may be several types of sludges, depending on the processes selected. The type of sludge and its characteristics play a large part in determining the type of treatment, dewatering, and disposal.

9.1.1 TYPES OF SLUDGE

The following types of sludge are most likely to be encountered through the processes discussed in this manual.

1. *Septic tank sludge* which comprises the accumulated solids settled out from the sewage over an extended period of time (6 months up to about 5 years). The sludge is partially stabilised but is not fit for drying beds or land application under normal circumstances. It can be composted but is usually tankered away from small plants and disposed at large plants.
2. *Primary Sludge* which comprises the solids settled out of sewage in primary sedimentation tanks. It is usually anaerobically digested but can be aerobically digested (not considered in this manual), or heat treated (also not covered) or composted. It is sometimes dewatered and either incinerated or taken to landfill.
3. *Humus Sludge* which comprises the biological solids or slimes sloughed off the media in a fixed film process (biological filter, RBC, or submerged media reactor). On small plants it is usually fed back to the septic tank, and on larger plants is combined with the primary sludge for combined treatment
4. *Waste Activated Sludge* is produced in the activated sludge process. It is low in energy and does not digest well, although this is sometimes attempted. On small plants it is usually run to drying beds for dewatering or occasionally to sludge lagoons. On larger plants it may be dewatered using dissolved air flotation (DAF) or linear screens, belt presses or centrifuges.

9.2 ANAEROBIC DIGESTION

9.2.1 INTRODUCTION

Treatment of sludge is an important part of the overall sewage treatment process and includes the treatment of both primary sludges, which are produced during settlement of raw sewage; and secondary sludges, which are derived from the settlement of humus sludge from biofilters or waste activated sludge. There are a number of options available for sludge treatment, one of which is anaerobic digestion and which is discussed in this section. Ultimately, the choice of treatment will depend on a number of factors such as the volume of sludge to be treated, complexity of the plant, availability of land for disposal and cost implications.

Since the mid 1990s most of the South Africa laws and regulations concerning the environment, potable water and wastewater have been replaced or updated and this has impacted on the legal requirements pertaining to wastewater sludge disposal. The Guidelines for the Utilisation and Disposal of Wastewater Sludges (WRC TT 261/06) should be consulted before undertaking the design of any sludge treatment process. These documents can be obtained from the Water Research Commission, Pretoria.

9.2.2 SLUDGE THICKENERS

As the first step of sludge handling and digestion, the need for sludge thickeners to reduce the volume of sludge should be considered.

The use of thickeners is advisable on works where thin sludges (<2% to 4%) are generated by the primary sedimentation tanks (PST's) or where automatic timed desludging of the PST's is planned. Gravity thickeners are then used to produce a thickened sludge of 5% to 7%, which limits the heat requirements for the primary digester and increases the effective retention time.

,Particular attention should be given to the pumping and piping of the concentrated sludge when considering and designing thickeners. Sludge retention should be limited to avoid premature possible onset of anaerobic conditions in sewage sludge. Sewage sludge should be thickened to at least 5% solids prior to transmission to digesters.

9.2.3 ANAEROBIC DIGESTION

FIGURE 9.2.1: Typical anaerobic digester (eWISA)

Sewage sludge may be rendered less noxious by anaerobic or aerobic digestion under controlled conditions. Anaerobic digestion is by far the most common as it can be very simple in concept with low specific power requirements, and also has a much longer history of use. If properly designed and operated this process should produce a stable, non-objectionable material unattractive to house flies, and will also effect a reduction in the solids content of the sludge.

Digestion renders the sludge suitable for drying and (subject to usage restrictions) also suitable for subsequent use as a fertiliser or soil conditioner.

The anaerobic digestion process consists of two distinct steps which are mutually dependent:
(i) In the absence of oxygen and given sufficient storage time, complex organic compounds are decomposed by enzymes and micro-organisms to simpler and smaller organic compounds, and fatty-acid forming bacteria convert these to volatile acids such as acetic, propionic and butyric acids
(ii) Given further storage and reaction time, these acids are then converted by methane-forming bacteria to stable organic forms, with carbon dioxide and methane being liberated.

The process if operated correctly produces both fully oxidised and fully reduced carbon compounds. It is slightly exothermic and very little sludge is metabolised. The sludge resulting from the process will have been stabilised so that it is no longer putrefactive although approximately 60% of the volatile solids will remain.

Greater efficiency can be obtained by heating the digester contents, but this is seldom practiced in small works which are operated at ambient temperatures. There are in fact three regimes for anaerobic digestion :
(i) Cold digestion which occurs at ambient temperatures
(ii) Mesophillic digestion where the sludge is heated to about 35 ºC (blood heat)
(iii) Thermophillic digestion where the sludge in the tanks is held at about 55 ºC.

As the temperature increases the required retention time diminishes through increased reacton rate, and the digestion efficiency improves. Thermophillic sludge is also effectively pathogen-free.

The reaction rate is also increased by mixing in the digestion tank to ensure that raw sludge which is fed to the digester is brought into good contact with the actively digesting sludge, so that it is contacted by the active bacteria and micro-organisms.

The concentration of the raw sludge which is fed to a digester is an important factor. If the sludge is too watery, i.e. the solids concentration is low, the effect will be to reduce the effective solids retention time in the digester and a loss of efficiency will result. Also the heat requirements for thermophillic or mesophillic operation will become excessive.

Sludge digestion can be divided into two phases. The function of the primary phase of digestion is to advance the acid and methane phases as far as possible, while that of the second phase is to enable the remainder of the process to be completed, but more importantly to allow for the remaining solids to be concentrated. Smaller works generally combine both phases in a single digester.

Three distinct zones can generally be distinguished in a normal unmixed operating digester:

1. The lower zone of concentrated solids which may also contain grit passing the detritors.
2. A central zone of supernatant liquor which is relatively free of solid matter (the so-called middle third)
3. A layer of scum on the surface.

If two stage digestion is employed, active fermentation and mixing will occur in the primary digester. The three zones [1 to 3 above] are then more clearly defined in the secondary stage and allow for a denser sludge to be withdrawn from the secondary digester. For smaller works, however, the use of two-stage digestion is usually not economical.

Concentrated sludge from the lower zone (from the single stage or second digester as appropriate) is withdrawn to the dewatering stage. It is good practice to remove sludge from the bottom of the cone to try to avoid blockages caused by grit. The supernatant liquor is also separately withdrawn and returned to the inlet works, downstream of the flow recorder.

The scum layer presents difficulties as it is not easily removed, but if left in place, becomes thicker and thicker and eventually occupies too much of the digester volume and needs to be cleaned out.

Cold and mesophillic anaerobic digestion destroy viruses and pathogens significantly but do not destroy a large proportion of Ascaris ova which remain and can be viable in soil for up to two years. However this is not a problem with thermophillic digestion.

The anaerobic digestion of organic matter results in the generation of large quantities of methane which has a high calorific value and as such has the potential for power generation, heating and incineration. In small works methane is seldom made use of but larger works normally use the methane gas to heat the sludge entering the mesophillic digesters.

Thermophillic digesters are seldom used in practice as they have high energy requirements (approximately double that of mesophillic digestion) which often results in negative energy balances and a need for supplementary fuel for heating. The heat losses can be substantial so that lagging of the surfaces is usually necessary. Thermophillic digestion also results in a greater degree of solubilisation into the sludge liquor (SNL) which becomes very strong and difficult to treat.

9.2.4 DESIGN OF UNHEATED SINGLE STAGE DIGESTION

9.2.4.1 Population Basis

For sedimentation tanks yielding raw sludge of 5% solids and greater, followed by single stage digestion, the required digester volume is:

Cold unmixed digestion - 160 litres per capita
Cold mixed digestion - 100 litres per capita

Satisfactory mixing means that the tank contents are adequately turned over within 6 to 8 hours.

The above figures provide for humus sludge but do not provide for waste activated sludge. If 5% sludge solids concentration in the raw sludge cannot be achieved, the above figures should be increased volumetrically pro rata.

9.2.4.2 Hydraulic Residence Period Basis

The digester volume may also be calculated on a sludge loading basis. The daily sludge volume may be estimated at 0.5% to 1.5% of the average daily wastewater flow (ADWF). This will have a sludge solids concentration of between 2% and 5%.

Alternatively, and preferably, the sludge volume may be estimated by calculation from the suspended solids removal by primary sedimentation. Suspended solids removal generally ranges between 200 mg/ℓ and 400 mg/ℓ with the lower figure applicable to weaker or dilute wastewaters (relatively low COD) and the higher figure being used for stronger wastewaters. For design purposes knowledge of the wastewater strength or at least the demographics of the works catchment area is therefore necessary.

The sludge volume is then calculated from the estimated sludge concentration. Plants with sludge thickeners will produce thick sludges (typically 5% to 7%). Plants with Dortmund type tanks or tanks with large sludge hoppers for consolidation will produce sludges of approximately 4% to 5.5%, and tanks with rotating bridge scrapers will give sludges of 2% to 4%.

Digester Volume

Unheated digestion with no mechanical mixing - 80 days sludge retention

Unheated digestion with satisfactory mixing - 50 days sludge retention.

9.2.4.3 Mixing

As mentioned above satisfactory mixing entails turning over the digester contents every 6 to 8 hours. Taking a digester of 2000 m^3 as an example this would imply the use of a circulating mixing pump of 250 to 330 m^3/h capacity (70 to 90 ℓ/s).

Alternatively using a mixing energy input of about 7 W/m^3 for the same digester would necessitate the use of a 14 kW mixer or the use of a pump drawing about the same energy.

9.2.5 DESIGN OF HEATED (MESOPHILIC) DIGESTION

The total digestion tank capacity should ideally be determined by rational calculations based upon such factors as volume of sludge added, its percent solids and character, the temperature to be maintained in the digesters, the degree of extent of mixing to be obtained, and the degree of volatile solids reduction required.

As a guide, the minimum combined digestion tank capacity outlined below will be required. Such requirements assume that the raw sludge is derived from ordinary domestic wastewater, that a digestion temperature is to be maintained in the range of 33 ºC to 37 ºC, that 40% or more of the volatile matter will be removed from the raw sludge, and that the digested sludge will be removed frequently from the system.

The sizing of digesters rests on two aspects. Firstly, the retention time is important as the methane forming organisms are slow to establish and the kinetics of the breakdown reactions of the volatile compounds in the digesters are slow.

Secondly, the volatile solids-loading is of importance. Primary sludge normally has a volatile solids content of 75% to 85% so that typically about 80% of the raw sludge solids are used to calculate the loading on the digester. A very thick sludge can feasibly have an excessive solids loading even though the retention time is more than adequate. In such cases gas production is usually more than sufficient for heating but good mixing is difficult to achieve. Conversely a thin sludge will tend to have low volatile solids loadings. In such cases even if the retention is designed to be adequate, the specific gas production per unit volume of digester will be low and there may not be enough gas for heating up to 35º C. Mixing will be easily achieved but there will be excessive water requiring to be heated and large volumes of supernatant liquor (SNL).

9.2.5.1 Sizing on Retention Time

For mesophillic well-mixed digestion, various design criteria give recommended retention times of 15 to 25 days for the primary tanks.

9.2.5.2 Sizing on Volatile solids loading

Completely-mixed systems will provide for intimate and effective mixing to prevent stratification and to assure homogeneity of digester contents. The system may be loaded at a rate of between 1.6 and 3.2 kg volatile solids/m^3.d in the active digestion units [primary digester(s)] . When grit removal facilities are not satisfactory, the reduction of digester volume due to grit accumulation should be considered. (Complete mixing can be accomplished only with substantial energy input.)

If the digester is sized correctly on hydraulic retention and a check on volatile solids loading gives a loading of less than 1.3 kg volatile solids/m^3.d, then the provision of sludge thickening before primary digestion should be strongly considered.

9.2.5.3 Moderately-Mixed Systems

For digestion systems where mixing is accomplished only by circulating sludge through an external heat exchanger, the system may be loaded at a rate up to 0.6 to 1.0 kg volatile solids/m^3.d in the primary digester(s). This loading may be modified upward or downward depending upon the degree of mixing provided.

9.2.5.4 Mixing

The provision of mixing as in Section 9.2.4.3 is applicable.

9.2.5.5 Secondary Digesters

Secondary digesters of volume 35% to 50% of that of the primary digesters should be provided for supernatant liquor (SNL) separation and decantation. Sludge from the secondary digesters will be discharged to the dewatering plant (usually drying beds).

9.2.5.6 Gas production

Stable digestion normally produces digester gas containing about 65% methane and 35% carbon dioxide at a rate of 0.8 to 1.0 m^3/kg volatile solids destroyed. The gas has a calorific value of 20 to 25 MJ/m^3. Good digestion normally reduces the volatile solids content by about 40% to 55%. With knowledge of the proposed installation it should be possible to check if the heat balance is favourable by estimating heating requirements and losses, and heating efficiencies.

9.2.5.7 Heating Requirements

Assuming for practical purposes that 1 kg of sludge has a volume of 1 litre and has the same heat capacity as water, then 1 litre of sludge will require 4.2 kJ/ºC rise in temperature (1 kℓ of sludge will require 4.2 MJ/ºC). It is therefore readily possible to calculate the heat requirements for raising the sludge temperature to 35ºC.

The heat losses should be based on heat transfer coefficients for the various materials, their thicknesses, and the mean temperature differences and should be within the scope of most chemical engineers. However for rough heat balance calculations it may be adequate to assume that heat losses add an extra 35% to 45% to the requirements for sludge heating. Higher losses may be applicable in colder climates, or where the installation is poorly insulated.

9.2.6 GENERAL ANAEROBIC DIGESTION DESIGN CONSIDERATIONS

9.2.6.1 Multiple Units

Multiple tanks are recommended. Where a single digestion tank is used, an alternate method of sludge processing or emergency storage to maintain continuity of service should be considered.

9.2.6.2 Depth

For those units proposed to serve as supernatant separation tanks, the depth should be sufficient to allow for the formation of a reasonable depth of supernatant liquor. A minimum side-water depth of 6 m is recommended on adequately sized plants. On smaller plants, tanks should be made as deep as is possible taking into consideration the geometric factors.

9.2.6.3 Maintenance Provisions

To facilitate draining, cleaning, and maintenance, the following features are desirable:

1. **Slope:** The tank bottom should slope to drain toward the withdrawal pipe. For tanks equipped with a suction mechanism for withdrawal of sludge, a bottom slope not less than 1:12 is recommended. Where the sludge is to be removed by gravity alone a 1:4 slope is recommended.
2. **Access Manholes:** At least two 900 mm diameter access manholes should be provided in the top of the tank in addition to the gas dome. There should be stairways to the access manholes.
3. **Sludge Inlets and Outlets:** Multiple recirculation withdrawal and return points, to enhance flexible operation and effective mixing, should be provided, unless mixing facilities are incorporated within the

digester. The returns, in order to assist in scum breakup, should discharge above the liquid level and be located near the centre of the tank. Raw sludge discharge to the digester should be through the sludge heater and recirculation return piping if heated digestion is used, or directly to the tank if internal mixing facilities are provided. Sludge withdrawal to disposal should be from the bottom of the tank. This pipe should be interconnected with the recirculation piping to increase versatility in mixing the tank contents, if such piping is provided. An unvalved vented emergency overflow should be provided to prevent damage to the digestion tank and cover in case of accidental overfilling. This overflow shall be piped to an appropriate point and at an appropriate rate in the treatment process to minimise the impact on process units.

9.2.6.4 Gas Collection, Piping, and Appurtenances

The entire gas system should be designed so that under all normal operating conditions, including sludge withdrawal, the gas will be maintained under positive pressure. All enclosed areas where any gas leakage might occur should be adequately ventilated.

9.2.6.5 Safety Equipment

All necessary safety facilities should be included where gas is produced. Pressure and vacuum relief valves and flame traps, together with automatic safety shut-off valves, should be provided. Gas safety equipment and gas compressors should be housed in a separate room with an exterior entrance.

9.2.6.6 Gas Piping and Condensate

Gas piping shall be of adequate diameter and shall slope to condensate traps at low points.

9.2.6.7 Gas Utilisation Equipment

Gas-fired boilers for heating digesters shall be located in a separate room not connected to the digester gallery. Such separate rooms would not ordinarily be classified as a hazardous location. Gas lines to these units shall be provided with suitable flame traps.

9.2.6.8 Electrical Fixtures

Electrical fixtures and controls, in places enclosing anaerobic digestion appurtenances, where hazardous gases are normally contained in the tanks and piping, shall comply with relevant safety provisions. Digester galleries should be isolated from normal operating areas to avoid an extension of the hazardous location.

9.2.6.9 Waste Gas

Waste gas burners shall be readily accessible and should be located at least 15 m away from any plant structure if placed at ground level, or may be located on the roof of the control building if sufficiently removed from the tank. All waste gas burners shall be equipped with automatic ignition, such as pilot light or a device using a photoelectric cell sensor. Consideration should be given to the use of natural or propane gas to insure reliability of the pilot light. In remote locations it may be permissible to discharge the gas to the atmosphere through a return-bend screened vent terminating at least 4.0 m above the ground surface, provided that the assembly incorporates a flame trap.

9.2.6.10 Ventilation

Any underground enclosures connecting with digestion tanks or containing sludge or gas piping or equipment shall be provided with forced ventilation. The piping gallery for digesters should not be connected to other passages. Where used, tightly fitting, self-closing doors should be provided at connecting passageways and tunnels to minimise the spread of gas.

9.2.6.11 Meter Requirements

A gas meter with bypass should be provided to meter total gas production.

9.2.6.12 Digester Heating

Wherever possible, digestion tanks should be suitably insulated to minimise heat loss.

9.2.6.13 External Heating

Piping shall be designed to provide for the preheating of feed sludge before introduction to the digesters. Provisions shall be made in the layout of the piping and valving to facilitate cleaning of these lines. Heat exchanger sludge piping should be sized for heat transfer requirements.

9.2.6.14 Heating Capacity

Heating capacity sufficient to consistently maintain the design sludge temperature should be provided. Where digester gas is used for sludge heating, an auxiliary fuel supply should be provided.

9.2.6.15 Hot Water Internal Heating Controls

A suitable automatic mixing valve shall be provided to temper the boiler water with return water so that the inlet water to the heat jacket can be held below a temperature at which caking will be accentuated. Manual control should also be provided by suitable bypass valves.

9.2.6.16 Boiler Controls

The boiler should be provided with suitable automatic controls to maintain the boiler temperature at approximately 80 ºC to minimise corrosion and to shut off the main gas supply in the event of pilot burner or electrical failure, low boiler water level, low gas pressure, or excessive boiler water temperature or pressure.

9.2.6.17 Thermometers

Thermometers should be provided to show temperatures of the sludge, hot water feed, hot water return, and boiler water.

9.2.6.18 Supernatant Withdrawal

Piping Size:

Supernatant piping should not be less than 150 mm in diameter.

Withdrawal Arrangements:

Piping should be arranged so that withdrawal can be made from three or more levels in the digester. A positive unvalved vented overflow should also be provided.

Sampling:

Provisions should be made for sampling at each supernatant draw-off level. Sampling pipes should be at least 37 mm in diameter, and should terminate at a suitably-sized sampling sink or basin.

9.2.6.19 Primary Digester Mixers

The digester mixer should have a design capacity which allows the turn over the digester contents in 6 to 8 hours. Various types of mixers are available but the most commonly used in small digesters is an external pump.

9.2.6.20 Raw Sludge Sump and Pumps

Provision should be made for an adequately sized sludge sump and pump capacity such that daily quantities of sludge can be transferred to the digester over a minimum period of time to allow optimum fermentation and digestion to take place. Allowances should be made for storage capacity in the sludge sump in case of a major power failure. Standby pumping equipment should be provided.

9.2.6.21 Shape of Digesters

Cylindrically shaped digesters with conical bottoms and roofs are recommended as this shape facilitates good mixing. The slope of the bottom cone should be 35° to 45° from the horizontal. Roofs should be conical. Access manholes in the roof should be at least 900 mm in diameter to allow for easy entry into the digester when renewals and/or repairs are made. An entry manhole should also be provided at ground level to permit the easy access of personnel and equipment.

9.2.6.22 Digester Pipework

In all cases care should be given to the pipe-work design, with 150 mm diameter pipes being the minimum size. In small works problems frequently arise from blockages in the draw-off pipes. Short radius bends should be avoided and provision made for rodding the pipes.

9.2.6.23 Waste Activated Sludge

In the activated sludge process the mass of sludge increases continuously and has to be wasted on a regular basis to maintain the required sludge concentration and sludge age in the reactor. Sludge can be wasted from either the return sludge system or from the reactor. The return sludge concentration is greater than that of the mixed liquor in the reactor. However the latter method is preferred as it enables the sludge age to be controlled more easily.

In either case it has been found that anaerobic digestion of waste activated sludge achieves only a minimal decrease in the volatile solids concentration.

Anaerobic digestion of waste activated sludge (WAS) is not favoured. WAS should therefore be discharged directly to drying beds or to sludge lagoons. The disposal of activated sludge from phosphate-removing plants presents particular problems, and such cases should be referred to a specialist.

9.2.7 DESIGN SEQUENCE AND WORKED EXAMPLE

Task An anaerobic digestion system is required for a 10 Mℓ/d works. The alternative possibilities should be examined.

Sludge Quantity

Assume removal of 250 mg/ℓ suspended solids by primary sedimentation and a sludge solids content of 4.5%.

Sludge solids = 10 x 250 = 2500 kg/d

Sludge volume = 2500/45 = 55.6 (say) 56 m^3/d

Cold Digestion

Unmixed – retention 80 days – volume = 80 x 56 = 4480 (say) 4500 m^3

Select two digesters at 2 250 m^3 - typical dimensions 20 m diameter with 5.2 m sidewall depth and 30º cone extending to a depth of 5.8 m

Mixed - retention 50 days – volume = 50 x 56 = 2800 m^3

Select one digester at 2800 m^3 – typical dimensions 22 m diameter with 5.3 m sidewall depth and 30º cone extending to a depth of 6.35 m

Mixing energy = 2800 x 7/1000 = 19.6 (say) 20 kW, or a 2800/8 = 350 m^3/h (97 ℓ/s) pump

Hence by using mixing for the cold digestion one can effectively nearly halve the civil costs at the expense of a 20 kW pump or mixer.

Heated (Mesophillic) Digestion

Select 20 days retention for primary digesters – volume 56 x 20 = 1 120 m^3. Select a single tank of typical dimensions 15 m diameter with 5.0 m sidewall depth and 30º cone extending to a depth of 4.3 m.

Check volatile loading   Assume 80% volatile solids in primary sludge
Volatile solids load per day = 2 500 x 0.8 = 2 000 kg/d
Volatile solids loading = 2 000/1 120 = 1.79 kg VS/m^3.d.
The loading is well within the recommended range.

Check Heat Balance. Assume 45% volatile solids reduction by digestion
Volatile solids destroyed = 2 000 x 0.45 = 900 kg/d
Assume a gas production of 0.9 m^3/kg VS destroyed
Gas production = 900 x 0.9 = 810 m^3/d
Assume a calorific value of 20 MJ/m^3 for the gas
Energy available = 810 x 20 = 16 200 MJ/day = 16.2 GJ/d
Assume a minimum sludge temperature of 10 ºC in winter
Energy requirements to heat sludge = 56 x (35 – 10) x 4.2 = 5 880 MJ/day (5.88 GJ/d)
Energy required allowing for losses of 40% = 5.88 x 1.4 = 8.2 GJ/d
Hence under the assumed conditions there will be adequate energy available.

For the secondary digester select a retention time of 10 days
Volume 56 x 10 = 560 m^3.
Select a tank 12.0 m diameter with 30º cone extending to a depth of 3.5 m, and with a 4.0 m sidewall depth and multiple draw-off points (including the base of the cone)

9.3 SLUDGE THICKENING

9.3.1 INTRODUCTION

In the treatment of wastewater sludges, the dewatering process begins once the dilute sludge is removed from the sedimentation tank or clarifier. The sludge from the settling tanks will typically have a solids concentration of 0.5 to 3%. Further sludge concentration is first accomplished by the use of thickening equipment which will increase the solids concentration to between 3 and 7% depending on the type of sludge and the process factors.

Sludges are thickened primarily to decrease the capital and operating costs of subsequent sludge processing steps by substantially reducing the volume. There are three general methods of thickening:

- Gravity Thickening
- Flotation Thickening
- Centrifugal Thickening

The main design variables to be considered in selecting a thickening process are:

1. Solids concentration and volumetric flow rate of the feed system
2. Chemical demand and cost if chemicals are employed
3. Suspended and dissolved solids concentrations and volumetric flow rate of the clarified stream
4. Solids concentration and volumetric flow rate of the thickened sludge
5. Type of sludge e.g. Primary sludge, digested sludge, waste activated sludge.

9.3.2 GRAVITY THICKENERS

Gravity thickening is the most commonly used method for raw or primary sludge thickening prior to digestion or dewatering, and is also used for thickening digested sludges and waste activated sludge prior to the dewatering stage.

For primary sludge a gravity thickener is used when the primary sedimentation tank does not provide sludge of adequate thickness for efficient digestion. In such cases a second purpose-designed settling and thickening tank similar in concept to a sedimentation tank is used when extended thickening times are required.

Gravity thickeners are generally used as pre-thickening for filter presses, belt filter presses, centrifuges and drying beds to achieve higher sludge concentration before dewatering.

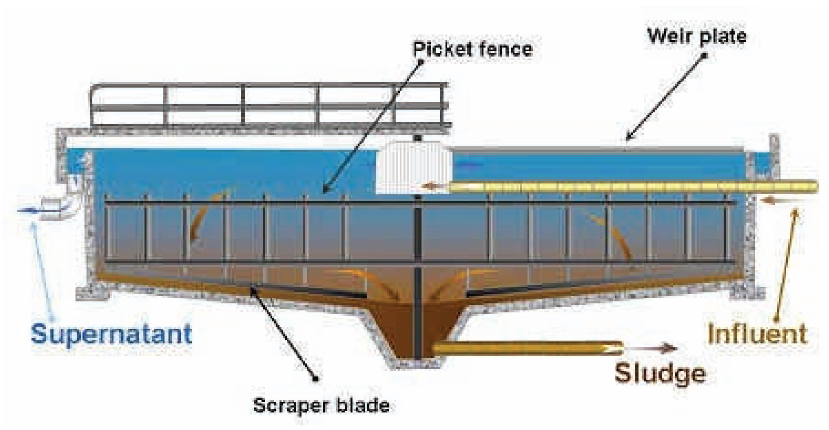

FIGURE 9.3.1: Schematic of circular gravity thickener (eWISA)

Most thickeners are circular tanks with rotating bridges with paddles to stir the sludge to assist in consolidation (picket-fence thickeners). These units are typically designed to store the accumulated solids for about 8 to 24 hours. Chemicals are sometimes added to aid the thickening process (e.g. iron and aluminum salts, polymers).

Gravity thickening of waste activated sludge is not recommended for biological phosphate-removal plants as the anaerobic conditions that occur on storage can lead to massive phosphate release. Dissolved air flotation (DAF) would be favoured in such cases

9.3.3 WORKED EXAMPLES

Task 1 - Design a thickener for a 10 Mℓ/d works for primary sludge to be fed to the digesters.

Assume 250 mg/ℓ suspended solids removal by primary sedimentation and automatic desludging of the tank to give a 0.6% TS sludge.

Sludge solids per day = 10 x 250 = 2 500 kg/d

Volume of sludge = 2 500/ 6 = 417 k-/d = 17.4 m^3/h

Select a surface loading (rise rate) of 1.0 m/h

Area of tank = 17.4 m^2 Diameter = 4.70 m

Select 20 hours sludge storage at 6% solids

Volume = (2 500/24) x 20/60 = 34.8 (say) 35 m^3

Select a 30º cone at the base of the tank

Volume of cone = 7.9 m^3

Height of sludge in cylindrical part of tank = (35 – 7.9)/17.4 = 1.56 m

Select a 3,0 m deep sidewall to give a liquid height of 1.4 m above the sludge for settlement and consolidation.

Task 2 - Design a thickener to thicken sludge from a single digester receiving a 4.5% primary sludge from a 10 Mℓ/d works where there is no supernatant liquor (SNL) draw-off.

Assume removal of 250 mg/ℓ suspended solids by primary sedimentation and a sludge solids content of 4.5%.

Sludge solids = 10 x 250 = 2500 kg/d

Sludge volume = 2500/45 = 55.6 (say) 56 m^3/d

Select 4 hours per day desludging time

Assume 80% volatile solids in primary sludge

Assume 45% volatile solids reduction by digestion

Volatile solids destroyed = 2 000 x 0.45 = 900 kg/d

Sludge solids remaining after digestion = 2 500 – 900 = 1 600 kg/d

Sludge concentration = 1 600/56 = 28.6 g/ℓ = 2.86 %

Select a surface loading (rise rate) of 0.7 m/h

Area of tank = (56/4)/0.7 = 20 m^2 Diameter = 5.0 m

Select 24 hours sludge storage at 6% solids

Volume = 1600/60 = 26.6 (say) 27 m^3

Select a 30º cone at the base of the tank

Volume of cone = 9.6 m^3

Height of sludge in cylindrical part of tank = (27 – 9.6)/20 = 0.9 m

Select a 2.5 m deep sidewall to give a liquid height of 1.6 m above the sludge for settlement and consolidation.

Task 3 - Design a gravity thickener for waste activated sludge from a 10 Mℓ/d extended aeration plant where sludge wasting takes place from the aeration tank.

<u>Sludge Quantity</u>. (Refer Chapter 6 on Activated Sludge)

Assume a wastewater COD of 600 mg/ℓ and a sludge growth coefficient of 0.3 kg ds/kg COD.

Sludge solids per day = 10 x 600 x 0.3 = 1 800 kg/d

Assume a MLSS concentration of 3 500 mg/ℓ (= 3.5 kg/m^3)

Volume of waste sludge = 1 800/3.5 = 514 m^3/d

<u>Thickener sizing.</u>

Select sludge wasting period of 6 hours per day

Flowrate = 514/6 = 86 m^3/h

Select surface loading (Upflow rate) of 0.5 m/h

Tank area = 86/0.5 = 172 m^2 ; Tank diameter = 14.8 (say) 15 m

Assume sludge consolidates to 2%

Settled sludge volume = 1 800/20 = 90 m^3

Select 15° cone at bottom of tank

Volume of cone = 1/3 x 172 x (15/2 tan 15°) = 115 m^3

Hence the sludge will all consolidate into the cone and the tank will not require a deeper sidewall. A normal 2.5 m sidewall will permit the tank to turnover on approximately a daily basis.

9.3.4 DISSOLVED AIR FLOTATION (DAF)

9.3.4.1 Introduction

Applications employing DAF systems range from potable water treatment to industrial effluent treatment and sludge (typically activated sludge) thickening. Advantages linked with DAF systems include the fact that they are high-rate processes when compared with more traditional gravity-based settlement systems. This means that a reduction in space requirements can be achieved, and in terms of sludge thickening, a thicker sludge can be produced. Additionally, DAF systems offer the operator some degree of flexibility, subject to design, with regard to the system's operating parameters

DAF systems may be designed for pressurisation and air dissolution of the total flow or, more commonly, the incoming sludge enters the flotation vessel where it comes into contact with a portion of recycled, treated effluent (sometimes termed whitewater). The percentage of the total effluent flow into which air is dissolved under pressure and subsequently recycled will be determined by several factors. Increasing the pressure within the vessel where the air is being dissolved ensures that a higher concentration of air dissolves into the liquid phase than is possible at atmospheric pressure. Once this portion of saturated effluent enters the flotation tank, the pressure is released back to atmospheric pressure. This immediately results in the recycled flow becoming supersaturated, resulting in the generation of microbubbles as the dissolved air comes back out of solution. These bubbles attach to, and form within, the solids or chemical flocs entering the vessel, causing them to float to the surface where they are retained and subsequently removed by a mechanical skimmer.

In the case of rectangular flotation tanks, the skimmer mechanism consists of a series of paddles or ‘flights’ which run on a belt or chain and skim just below the surface of the tank removing the ‘float’ into a trough for further treatment or, in some instances, recovery of materials. The alternative of circular DAF tanks may incorporate rotating skimmer blades feeding a ‘float’ trough or involve use of a circulating, revolving scoop. In cases where some gross solids may be present and there is risk of gradual accumulation of sludge build-up on the flotation tank floor, the design may also incorporate a floor scraper.

9.3.5 DESIGN

Inevitably the design details for any given effluent treatment system will be dependent on a number of specific factors. There are, however, several key design parameters which are commonly applied when considering and assessing the design of a DAF system. The parameters listed below are, where applicable, accompanied by design figures. These figures are provided as an indication of the range of figures one may encounter.

9.3.5.1 Air / Solids Ration

The air:solids (A:S) ratio may be reported as a volume:mass ratio or a mass:mass ratio and will be application specific. To give an idea of the range of A:S ratios commonly applied, typical values range between 0.005 – 0.06 mℓ/mg which, at 20°C and atmospheric pressure (say 101 kPa) is equivalent to 0.006 kg – 0.072 kg of air per mg of solids to be removed. For wastewater sludges it is typically in the range 0.03 to 0.06 kg/kg.

9.3.5.2 Hydraulic Loading

The DAF hydraulic loading rate is a measurement of the volume of effluent applied per unit effective surface area per unit time. This results in process design figures expressed as equivalent upflow velocities with units of m/h. This figure should be application specific but as a general guide the figures which should be expected would be between 2 m/h and 10 m/h. A key consideration with regard to this design parameter is whether the loading rate includes the recycled volume as well as the influent wastewater volume being applied per unit area of the system.

9.3.5.3 Solids Loadings

Solids loadings are normally given in units of mass per unit area per unit time ($kg/m^2.h$). Typical figures encountered range from around 2 $kg/m^2.h$ up to 15 $kg/m^2.h$, although again the design will be application specific, depending on the nature of the solids to be removed and the extent to which chemical aids are used.

9.3.5.4 Recycle Ratio

The recycle ratio is determined as the fraction of the final effluent produced which is returned and saturated under pressure prior to entering the flotation vessel where the pressure is subsequently released and the bubbles are generated. The recycle ratio can vary immensely with recycle ratios being typically 15 - 50% for water and wastewater treatment application. However, for activated sludge flotation thickening, recycle ratios of 100 - 200% have been applied.

Air dissolution rates are proportional to absolute pressure (i.e. system gauge pressure + atmospheric pressure) in accordance with Henry's Law of partial pressures of gases adjacent to liquids. Thus, for a given application, the higher the operating pressure of the air / water saturation vessel, the lower the required percentage recycle, and vice-versa. Operating pressures can therefore vary widely but are typically in the range 3 – 7 bar (300 – 700 kPa).

9.3.5.5 Saturation of effluent

The production of saturated water from which the micro-bubbles are generated is normally achieved in two ways. The first which is common to potable water treatment involves passing the required flow of treated effluent through a packed bed system which is pressurised using a pump which is often a centrifugal pump. In systems where solids are likely to be encountered, e.g. sludge treatment, the saturation vessel is likely to be empty to prevent the fouling of any packing materials. The percentage of saturation which can be achieved will depend on the design of the system but, with good design, saturation efficiencies of up to 80 - 95% can be expected.

9.3.5.6 Flow Regime

To ensure that DAF systems operate as designed, it is important to ensure that the system does not encounter sudden changes in the flow regime. For this reason some form of flow balancing or regulation is recommended to ensure a consistent flow rate. Another consideration is to develop a flow path through the flotation tank which ensures the maximum removal of solids via their entrainment in the air microbubbles generated.

9.3.6 OPERATING ASPECTS

DAF units can be applied successfully to a range of different waters, wastewaters and sludges and demonstrate certain specific advantages over more conventional solids removal processes. DAF plants can be designed to be small, compact and robust systems with a high rate of operation. DAF systems are capable of coping with reasonable variations in influent water quality, and to some extent variations in flow. Disadvantages of DAF systems may include the increased service and maintenance costs when compared with traditional sedimentation systems and the increased operating costs due to the energy requirements of the system. The heart of a DAF plant is the saturator and the recycle pressure release system. If these are well designed and effective the plant will usually perform satisfactorily.

9.3.7 CENTRIFUGAL THICKENERS

There are a large number of thickeners of patented and non-patented designs on the market. Some rely on gravity to drain the sludge through a mesh screen or porous plate, which may be flat or curved in section, such as disc or linear or gravity belt screens. Other designs rely on centrifugal force which may or may not be assisted by gravity. In these designs an Archimedian or shaftless screw forces the sludge though a tube which usually reduces in diameter and which has a mesh or porous plate for its outer wall. The pressure squeezes the water out of the sludge and through the mesh producing a thickened or even a final dewatered sludge as its final product. Typical of these are the rotary drum thickener screen and the rotary press which are described below.

9.3.8 ROTARY PRESS.

The principle of operation is simple. Sludge is fed into a rectangular channel, and rotated between two parallel revolving stainless steel chrome plated screens. The filtrate passes through the screens as the flocculated sludge advances within the channel. The sludge continues to dewater as it travels around the channel, eventually forming a cake near the outlet side of the press. The frictional force of the slow moving screens, coupled with the controlled outlet restriction, results in the extrusion of a very dry cake.

9.3.9 ROTARY DRUM SLUDGE THICKENERS

Rotary drum sludge thickeners have cylindrical drums with progressive series of screen elements that are mounted horizontally on four shaft-mounted wheels and supported on a tubular frame. The smallest openings are used to screen the influent sludge, followed by coarser elements as the sludge thickens. The screening element is either woven mesh or wedgewire. The screens are used to remove free liquids from a variety of sludges, including those with appreciable amounts of microbial particles. Sludge containing 0.5% - 3% solids can be thickened to 3% - 15%, depending on the type of sludge. The influent enters a headbox, where the energy is dissipated, and the flow is evenly distributed onto the interior sidewalls of the drum. Solids are retained on the screen surface and the liquid flows radially through the screen openings. Splash guards direct the liquid filtrate to a central drainage area, and the solids are transported axially, by flights, to the open end of the drum. The rotation of the drum allows the entire screening surface to be continuously or intermittently washed by a fixed, external, spray bar fitted with a bank of spray nozzles.

9.3.10 DISCUSSION

It can be seen from the above descriptions that centrifugal thickeners can be classed as either sludge thickeners or as dewatering units, as there is a grey intermediate area where the sludge can be regarded as a well thickened sludge for the next stage of dewatering or as a poorly dewatered sludge if there is no secondary dewatering plant. The rotary screw press described in the dewatering plant section could equally well have been included here.

9.4 SLUDGE DEWATERING

9.4.1 INTRODUCTION

Dewatering is primarily a physical operation that separates the liquid and solid portions of the dilute sludge generated during the precipitation/clarification process. The resulting material is a stiff fluid. Dewatering is usually performed in a series of steps utilising two or more pieces of equipment, with each subsequent step reducing the percentage of liquid in the sludge. Dewatering can be a single stage process or a two stage process where the dewatering equipment follows the thickening plant.

After thickening, further dewatering of the sludge is achieved through the use of mechanical dewatering devices. The most widely used mechanical dewatering devices are centrifuges and belt presses. Screw presses and rotary presses are also used on some plants. Mechanical dewatering will produce a sludge with an approximately 10 to 40% solids content depending on the nature of the sludge and the type of equipment being used.

9.4.2 BELT PRESSES

Belt press type dewatering machines are used for dewatering sludge produced in the treatment processes of water, sewage and wastewater. Probably the most common application is the dewatering of waste activated sludge (WAS) either on its own or in combination with raw or anaerobically digested sludge, as a second stage after dissolved air flotation (DAF)

Process Description: The incoming sludge is first conditioned by polyelectrolyte to form flocs before entering the dewatering zones of the unit. The conditioned sludge is initially dewatered in a gravity drainage zone, and then in subsequent dewatering steps, subjected to gradually increasing pressure between two filter belts. The sludge cake discharged from the belts has a relatively low moisture content.

9.4.3 PERFORMANCE

With a feed concentration of about 2% to 4%, a belt press will produce a sludge cake of about 16% to 18% on WAS, about 22% to 26% on a mixture of raw and WAS, and about 25% to 30% on a mixture of WAS and digested sludge.

9.4.4 OPERATIONAL ASPECTS

A belt press is slow moving and relatively quiet in operation. It produces a drier cake, sludge for sludge than a centrifuge. It is reliable in operation if well maintained but does need attention to belt adjustment, and roller and pulley replacement on a not infrequent basis.

9.4.5 CENTRIFUGES

Centrifuges operate using centrifugal force at many times the force of gravity (typically 2 000 x g) to increase the sedimentation rate of solid sludge particles. The most common centrifuges found in wastewater treatment dewatering applications are the continuous bowl centrifuges. The two principal elements of these are the rotating bowl and the inner screw conveyor. The bowl acts as a settling vessel; the solids settle due to centrifugal force from its rotating motion. The screw conveyor which rotates at a slight speed differential to the bowl picks up the solids and conveys them to the discharge port. Although there are many different designs of centrifuges, they all are provided with a polymer-conditioning unit for conditioning the incoming sludge using cationic polymers. Polymers are usually fed inside the bowl because the high shear forces may destroy flocs if they are formed prior to entry. Also, large particles settle rapidly in the first stage of the bowl. Thus, economical solids recovery can be achieved through internal feeding of polymers after the large particles have settled.

9.4.5.1 Process Description

Sludge at 1% to 3% is pumped into the feed tube of the centrifuge where it is dosed with polyelectrolyte on entry to the machine. The bowl of the centrifuge has a cylindrical section containing the pool of sludge where the solids settle under centrifugal force, and a conical section which has a dry area outside of the pool for sludge to drain. The rotating scroll operating at a slight speed differential to the bowl scrapes the solids out of the pool and up the beach area where it drains and dries before discharging out of the end of the cone. The clarified liquid or centrate discharges over a weir at the other end of the bowl. Adjustable variables on most designs are pool depth and differential speed which sets the residence times of sludge in the unit.

9.4.5.2 Performance

The centrifuge is suited to any type of sludge or mixtures thereof. On WAS it will produce a cake of about 12% to 14%. On raw sludge it will form a cake of about 25% to 28%, and on digested sludge a cake of about 35% to 40%. Mixtures of WAS and raw run at about 18% to 22%, and WAS and digested at about 25%.

9.4.5.3 Operational aspects

A centrifuge is a high speed piece of precision machinery. It runs at very high rotational speeds and G-forces, and good operational attention and maintenance is necessary. If seriously out of balance the machine can jump off its mountings and create serious damage and danger to operating staff. For this reason centrifuges are almost invariably fitted with vibration monitoring to detect out of balance loads and will trip out automatically.

The machines are also subject to severe abrasion forces from grit in the sludge. Early units wore out quickly and required frequent rebuilding. Present day machines will have ceramic wear plates and will operate for at least a year and sometimes two or three years before requiring refurbishment. The machines have a high energy consumption and a high start-up current. They are also noisy and their rooms are usually designated as noise areas. However they are reliable and consistent in operation and probably the most versatile if variable sludge compositions need to be treated.

9.4.6 SCREW PRESS

The rotary dewatering press is a screw press used for sludge dewatering. The rotary dewatering press is ideal for small applications and is used extensively for the dewatering of sewage sludges. Capacity ranges from 2 to 12 m^3/h. The unit consists of a wedge wire drum and internal screw. The outlet of the machine contains a dewatering cone, which can be automated for certain applications. The entire unit is enclosed in a stainless steel main case with inspection hatches. Sludge is fed into the screw press and is compacted to an increasing extent by the internal screw conveyor. This forces the water out of the sludge through the containing screen. Depending on the nature of the sludge relatively dry cakes can be obtained.

9.4.7 SLUDGE DRYING BEDS

9.4.7.1 Introduction

Sludge drying beds are the oldest method of dewatering and are still the most widely used. They have the advantage of dewatering to a stage where the sludge is effectively dry and can be stacked and stored. Drying beds are still the first choice on small plants where the additional complication of mechanical dewatering equipment is not warranted. Dewatering on beds is brought about by drainage, evaporation and decantation.

FIGURE 9.4.1: Typical sludge drying beds (eWISA)

Decantation is the first stage of the dewatering process and usually is only applicable to waste activated sludge which contains free water. Many beds for drying waste activated sludge have adjustable weirs or draw-offs which decant the clear supernatant and which greatly accelerate the overall drying process. The drying bed then initially behaves as a settling tank in the early stages of running the sludge onto the bed.

Drainage, which accounts for the major part of the dewatering process, depends on the drainage characteristics of the sludge and the porosity of the media.

Evaporation, a much slower process, accounts for final drying. The rate of evaporation is similar to that from a free water surface during the initial stages when the sludge has a liquid surface. As the liquid surface recedes into the interstices of the sludge, the rate of evaporation declines drastically, and the sludge usually starts to crack.

Climatic conditions greatly affect sludge dewatering on drying beds and influence drying time. The drying time is shorter in regions of greatest sunshine, low rain-fall and low humidity. Where summers are longer and in dry regions where humidity is low, conditions are more favourable for sludge drying bed use.

9.4.7.2 Description of Typical Drying Beds

Drying beds comprise the following basic features:

1. A concrete floor slab generally laid to slopes of 1:12
2. Low brick or concrete walls providing for a series of beds
3. Filter media grading from 20 mm crushed stone or gravel at the bottom to medium coarse sand at the top for digested sludges, and to fine sand for waste activated sludges
4. A drainage system to collect the drained liquor to a central sump
5. A system of pipes and valves to discharge the sludge onto the beds, with precast concrete splash pads at the inlet points to stop erosion of the top of the filter media.

The total depth of the filter media should be about 300 mm.

Drying beds for waste activated sludge should also be provided with adjustable overflow weirs to enable clarified liquor to be decanted from the surface, as this greatly increases the rate of drying.

9.4.7.3 Digested Sludge Drying Beds

The area of drying beds required should be based on:

(i) The total volume of digested sludge to be withdrawn in a given time (i.e. per week or month)
(ii) The depth of sludge to be run onto the beds, with 200 mm generally considered a maximum
(iii) The prevailing climatic conditions which can affect drying time from 10 to 30 days per bed.

A combination of the above will result in drying bed areas in the range 0.1 to 0.2 m^2 per capita.

Alternatively the beds can be sized on a solids loading of 10 to 15 kg/m^2 and a drying time (turnover) of 20 to 40 days. Long drying times are necessary in KZN during the wet and humid summer months and especially in the Western Cape during the wet winter months. In the drier parts of the country the turnover time will be quicker.

9.4.7.4 Waste Activated Sludge

Due to the thin layer of sludge which results from dewatering waste activated sludge on a sludge drying bed, the dry sludge has a lower viable Ascaris count than anaerobically digested sludges dewatered on sludge drying beds.

If the activated sludge process is to be operated on a sludge age basis, the beds should be sized for the shortest sludge age likely to be used, as this will give the highest amount of sludge solids to be wasted daily. For some degree of sludge stability this will usually not be less than 10 days.

The design should then allow for:
1. The volume of mixed liquor to be wasted daily
2. A depth of mixed liquor on the beds not exceeding 300 mm
3. The prevailing climatic conditions which can affect drying time from 3 to 10 days per bed per application.

As a check for the area based on the factors given above, the beds should provide for a solids loading rate of about 2 to 3 kg dry solids/m^2.

9.4.8 WORKED EXAMPLES

Task 1 **-** Design sludge drying beds for the secondary digested sludge from a 10 Mℓ/d works for the digesters sized in Example 9.2.7

Sludge solids remaining after digestion, based on 250 mg/ℓ suspended solids removal by primary sedimentation, 80% volatiles in the primary sludge, and 45% reduction in volatile solids by digestion = (10 x 250) x 0.2 + (10 x 250) x 0.8 x 0.55 = 1 600 kg/d

Assume a secondary digested sludge solids content of 6%
Sludge volume = 1 600/60 = 26.7 (say) 27 m^3/d

Select solids loading for drying beds of 15 kg/m^2 and a drying (turnover and removal) time of 30 days
Drying bed area = 27 x 30/15 = 54 (say) 60 m^2
Select 4 beds of dimensions 5 x 3 m which allows sludge to be run to one bed per week.

Task 2 - As an alternative to gravity thickening and mechanical dewatering for the 10 Ml/d extended aeration plant in Example 9.3.2.1, it is required to size drying beds for the waste mixed liquor from the plant.

Waste sludge solids per day based on 10 Mℓ/d of wastewater having a COD of 600 mg/ℓ and a sludge growth index of 0.3 kg ds/kg COD = 10 x 600 x 0.3 = 1 800 kg/d

Volume of waste mixed liquor based on a MLSS concentration of 3 500 mg/ℓ = 1800/3.5
= 514 m^3/d.

Select bed solids loading of 2.5 kg/m^2 and a drying time of 7 days
Bed area = 1 800/2.5 x 7 = 5 040 (say) 5 000 m^2
Select 10 beds each 20 x 25 m permitting two beds to be run per day for 5 days per week

CHAPTER 10

10. DISINFECTION

10.1 INTRODUCTION

Wastewater in general and raw sewage in particular is considered a hazardous waste because it contains human waste, and human wastes contain, to a greater or lesser extent all known human pathogens.

Typically, raw sewage contains high numbers of micro-organisms and each unit process used in treating wastewater reduces the number of micro-organisms as shown in the **Table 10.1.1** below.

TABLE 10.1.1: Removal of bacteria by different processes

Process	% Removal
Coarse screens	0 - 5
Fine screens	10 - 20
Grit chambers	10 - 25
Primary sedimentation	25 - 75
Trickling filters	90 - 95
Activated sludge	98 - 99

There still remain high numbers of pathogenic micro-organisms in the treated wastewater even after the best possible biological treatment. Some additional treatment is required to ensure that the effluent is safe for discharge to a public stream., The most common additional process is disinfection.

10.2 PURPOSE AND METHODS OF DISINFECTION

The goal of water disinfection is to remove or inactivate pathogenic micro-organisms. Disinfection is not synonymous with the sterilisation of water, in which all organisms are killed. In disinfection, the primary pathogenic micro-organisms targeted for inactivation include bacteria, viruses and protozoan cysts.

The disinfection of water has been practised for several hundred years, even though initially there was no understanding of the principles involved. Historical records show that the boiling of water had been recommended at least as early as 500 B.C.

Chlorine was identified as a chemical element in the early 1800s. Because of its characteristic colour, the name chlorine was derived from the Greek word chloros, meaning pale green. It was not until sometime later, however, that its value as a disinfectant was recognised.

The possible methods of wastewater effluent disinfection are many and include natural processes (predation and normal death), environmental factors (salinity and solar radiation), both of which occur in maturation ponds, and methods having certain industrial applications (ultrasonics and heat). However only those methods that have possible general applications for wastewater and water re-use, will be explored.

10.3 CHLORINE

Chlorine has been the dominant disinfectant of wastewater. It is available in different forms. Because of the importance of chlorine in wastewater treatment it will be discussed in more detail.

10.3.1 LIQUIFIED/GASEOUS CHLORINE

This is the basic elemental chlorine and is used in large quantities for sanitary purposes; restaurant sanitisers, potable water treatment, wastewater treatment, swimming pools, cooling waters and other industrial process water treatment, but not sold to untrained users. Liquified-gaseous chlorine is the principal form of chlorine used in wastewater disinfection. It is also used for odour control, destruction of hydrogen sulphide, prevention and control of septicity and for elimination of activated sludge bulking.

10.3.2 HYPOCHLORITE

This can be provided either in the form of sodium or calcium hypochlorite. Sodium hypochlorite is provided as a clear water solution available in concentrations of up to 15% available chlorine by weight. Calcium hypochlorite is available either as a dry granular white powder or in tablet form, in strengths of 70% available chlorine by weight.

10.3.3 ON-SITE HYPOCHLORITE GENERATION

Complete systems are available in South Africa for the on-site manufacturing of chlorine and hypochlorite solutions by electrolysis from salt (sodium chloride) which also diminishes the potential hazard of handling the liquified gaseous chlorine in pressurised containers. The solution strength of this hypochlorite solution is normally about 0.7% as chlorine.

10.4 CHLORAMINES

A combination of chlorine and ammonia, chloramines are less efficient as a biocide, as compared with free chlorine, but react to form fewer undesirable organic by-products.

10.5 BROMINE, BROMINE CHLORIDE AND IODINE

Bromine and bromine chloride are relatively soluble in water (more so than chlorine), however, bromine is too hazardous a chemical to handle in the treatment of wastewater. Bromine chloride is easier to handle, but requires special dosing equipment.

Bromine compounds have an advantage over various chlorine compounds in that bromine residuals usually die away rapidly with little toxic effects on aquatic life and somewhat lower dosages can be applied.

Iodine is a grey-brown crystalline solid, which is only slightly soluble in water. It has been used as an effective method of water treatment on an emergency basis and on very small potable water applications.

10.6 OZONE

Ozone is another unstable gas that must be produced at the point of use. It is produced by the reaction of oxygen-containing gas (air or pure oxygen) in an electric discharge. It is a very powerful oxidant. From recent investigations, it appears that ozone, in combination with either chlorine or chlorine dioxide, could solve the disinfection problem of both bacterial and viral contamination in wastewater effluents. However the process is capital-intensive and has relatively high power consumption.

10.7 ULTRAVIOLET RADIATION

Special lamps (mercury vapour) produce ultraviolet radiation. The disinfection reaction from the radiation occurs on the thin film surfaces of water where micro-organisms can be exposed to the radiation reaction. With wastewater, lethal action cannot be exerted through more than a few centimetres, which limits its application to wastewater of high quality and with low solids contents and minimal turbidity..

10.8 CHLORINATION PRACTICE

Chlorination may be carried out by direct use of gaseous chlorine obtained from cylinders via purpose-designed chlorinators, or by dosage of hypochlorite solution which contains chlorine. Normally the dosage equipment would be sized for a maximum dosage of about 10 mg/ℓ with normal dosage levels being about 3 to 5 mg/ℓ. If the effluent is pumped it is beneficial to provide an interlock on the dosage pump so that both pumps operate together thus setting constant dosage. For gravity flow one normally uses a constant chlorine feed rate and provides a relatively large contact tank for equalisation.

10.9 GAS CHLORINATORS

Chlorine is obtained in 68 kg or 900 kg cylinders. The chlorine is compressed to a high pressure in the cylinders so that it is stored in liquid form. Chlorine is normally drawn off as gas from the cylinders. When the gas is drawn off, liquid chlorine evaporates to refill the headspace. The evaporation process consumes latent heat and cools the container. There is, therefore, a limit on the rate of draw-off of chlorine from a cylinder unless container-warming facilities are provided. The maximum draw-off is approximately 1.5 - 2 kg/h for a small cylinder and 7.5 - 10 kg/h for a large cylinder, at normal ambient temperature. This draw-off depends on prevailing ambient temperature, however, and should be confirmed with the suppliers.

A chlorinator basically consists of a vacuum regulator with adjustable gas flow measurement by needle valve via a rotameter tube for indication, and an ejector on a pressure water line. The water passing through the ejector creates a vacuum, which sucks the chlorine gas into the water at the ejector and into solution. The solution of chlorine in water is then added to the effluent at a suitable point. The amount of gas drawn into the water at the ejector is regulated by the needle valve and measured on the rotameter tube. Safety devices are normally built into the chlorinator to close off the gas in the event of water supply failure and to prevent water entering the gas lines.

Wet chlorine gas or chlorine solution is extremely corrosive and normal materials cannot be used. Most metals, lubricants and packing compounds are attacked. Special materials are therefore used.

10.10 HYPOCHLORITE FEEDERS

Sodium hypochlorite (15% free chlorine) is available in black plastic 20 litre containers. It deteriorates on standing and large stock holdings should therefore be avoided. The solution can be drip fed or preferably dosed via a metering pump into the effluent. If generated on site it will be much more dilute at about 0.7% and larger volumes of liquid requiring larger dosage pumps will be required. Although the solutions are alkaline it remains corrosive due to the strong oxidising power of the chlorine, and careful selection of materials is required.

Calcium hypochlorite (HTH) contains 70% free chlorine and is available in drums as granules or tablets. It can be dosed in water-suspension form but care needs to be taken to avoid particles causing blockages. Alternatively, on small plants, tablet dispensers can be used, and they are refilled daily.

10.11 SAFETY PRECAUTIONS

Apart from being corrosive, chlorine is also very poisonous and stored calcium hypochlorite can be explosive under certain conditions. The storage and use of chlorine therefore requires rigid adherence to safe practice procedures. Full details regarding these can be obtained from the suppliers who, from time to time, also conduct courses in the use and handling of chlorine.

Ventilation of the storage and dosage area must be designed to prevent accumulation of chlorine gas, which is heavier than air. Floor level discharge ducts and fresh air fans entering near the ceiling as is a semi-open type of structure are common practice.

10.12 CHLORINE CONTACT TANKS

Chlorine disinfection is not instantaneous and can take tens of minutes or even several hours depending on various factors such as pH and ammonia concentration. When ammonia is present chlorine reacts to form chloramines which are slow disinfectants and can take up to 24 hours to react to completion. It is therefore normal when disinfecting with chlorine to provide contact tanks for the disinfection reactions to take place. The criteria range from 20 minutes contact at peak flow up to 24 hours storage.

It is normally not economical or practical to provide contact tanks the same size as the aeration tank unless it is possible to construct a maturation pond system. In normal circumstances therefore it is common to provide tanks of 1.5 to 2 hours retention at average flow, and to operate at somewhat higher chlorine dosages to get some free chlorine disinfection which is much more rapid. However this can be relatively expensive on larger plants.

FIGURE 10.1: Typical chlorine contact tank

CHAPTER 11

11. CHEMICAL TREATMENT

11.1 ALKALI ADDITION

11.1.1 INTRODUCTION

The effluent discharged from a wastewater works is subject to limitations on its pH level which needs to be between 5,5 and 9,5 in most cases, although the Special Standard is more stringent at between 6,5 and 7,5.

The final effluent pH can be below the lower limit if the ammonia concentration in the sewage is high and the alkalinity is low. Low alkalinity sewages are common in KZN and the Eastern and Western Cape. In such cases the process of nitrification uses up all the alkalinity in the sewage and the pH can drop to as low as 4,0. Often in such cases the ammonia remains high because the sewage has run out of alkalinity and cannot nitrify further.

Activated sludge plants can be modified to denitrify and recover some of the alkalinity but fixed film plants (Biological filters, RBC's and Submerged media reactors) often cannot achieve much denitrification and tend to have low pH effluents. In such cases it may be necessary to add an alkali to the sewage at a suitable point in the plant.

11.1.2 ALKALI'S USED

11.1.2.1 Lime.

The most commonly used alkali is lime which is usually the lowest in cost and sometimes has other benefits to the process. It is usually fed as a dry powder into a slurry mixing tank and added after the septic tank or primary sedimentation tank as the case may be. Sufficient lime should be added to bring the effluent pH up to about 7.0, with a residual alkalinity in the final effluent of about 50 mg/ℓ as $CaCO_3$.

11.1.2.2 Soda Ash.

Soda Ash or sodium carbonate is also used occasionally. It is highly soluble and can be added by dosing pump as a 5% solution. It is also added after the septic tank or primary sedimentation tank as the case may be. Sufficient soda ash should be added to bring the pH of the effluent up to about 7.0, with a residual alkalinity in the final effluent of about 50 mg/ℓ as $CaCO_3$.

11.1.2.3 Caustic Soda.

As implied by its common name Caustic Soda (sodium hydroxide) is more hazardous to handle than the other chemicals but is used on occasions. It is usually supplied as caustic lye solution at about 50% strength. As for the other chemicals it should be added after the septic tank or primary sedimentation tank as the case may be. Sufficient caustic should be added to bring the effluent pH up to about 7.0, with a residual alkalinity in the final effluent of about 50 mg/ℓ as $CaCO_3$.

11.1.3 DOSAGES

Dosages may range from zero up to as high as 100 mg/ℓ but are usually around 50 mg/ℓ.

11.1.4 PHOSPHATE REMOVAL

Chemical removal of phosphates by metal salts significantly increases the acidity of sewage effluents. It may be necessary to add alkali to neutralise this acidity as well as compensating for the drop in pH from nitrification. In such cases the dosage of alkali can easily be more than doubled.

11.1.5 LABORATORY TESTS

Laboratory tests on alkali demand should be carried out for both low alkalinity and phosphate removal purposes before designing the dosage systems.

11.2 METAL SALT ADDITION

11.2.1 INTRODUCTION

Metal salts (Usually iron or aluminium) are added to the wastewater treatment process for two reasons. Firstly iron and aluminium salts are coagulants and are commonly used for flocculation at potable water works. This coagulating effect can significantly improve the performance of primary sedimentation tanks at wastewater works by increasing COD and suspended solids removal from the wastewater in the settlement process. Secondly both iron and aluminium precipitate phosphate out of solution and are therefore commonly used in parts of the country where there is a phosphate limit on effluents discharged to watercourse.

11.2.2 ENHANCED PRIMARY SEDIMENTATION

The addition of low dosages (typically 25 mg/ℓ) of ferric chloride to the incoming wastewater can improve the removal of suspended solids by primary sedimentation by about 30%. Higher dosages give diminishing returns on suspended solids removal and add to the sludge load entering digesters. They also add to the conductivity and TDS of the water which is also subject to limits set by DWAF for discharge. The process is not something that should be designed into a works but can be used as a palliative if the aerobic process of a plant is overloaded and there is spare digester or sludge handling capacity.

11.2.3 PHOSPHATE REMOVAL

Relatively high dosages of ferric chloride, ferric sulphate, or aluminium sulphate are used to precipitate phosphate out of wastewater effluents. This may be required when the effluent discharged from a works has to comply with the 1 mg/ℓ maximum level set for phosphorus in sensitive catchments. Depending on the incoming phosphate level the dosages can be typically in the range of 150 mg/ℓ to 200 mg/ℓ.

In an activated sludge process the metal salt is often added to the mixed liquor where the continual recycling of sludge containing the metal hydroxides enhances removal efficiency. This also maintains phosphate removal for limited periods even when there are breakdowns in the dosage system.

For biological filter plants the metal salt can be added at the primary sedimentation tanks which as seen above will enhance suspended solids removal. However there is the danger of insufficient nutrient phosphate being available for good purification on the filter, although successful operation in this mode has been reported. The alternative is to add the salt after the biological filters and remove the floc in the humus tanks.

The requirements for good flash-mixing at the point of addition of the salt is relevant to good practice on wastewater works as well as to waterworks, particularly on biological filter plants.

It would also be good practice to design primary sedimentation and humus tanks at lower peak upflow rates when addition of metal salts is contemplated on a works. The settlement rate of alum flocs in particular, is lower than normal humus solids.

CHAPTER 12

12. PUMPS

12.1 CLASSIFICATION OF PUMPS

In this manual, pumps have been broadly classified into the following groups:

- Rotodynamic, including centrifugal, axial flow and mixed flow
- Positive displacement, including reciprocating, progressive cavity and peristaltic or diaphragm pumps
- Air lift and ejector pumps
- Screw pumps.

12.2 ROTODYNAMIC PUMPS

These incorporate a wheel or rotor of some kind whose rotating blades or vanes impart acceleration to the liquid passing through the pump. The speed of rotation will vary with the application and could be 500 rpm for a large raw sewage pump and up to 2950 rpm for a small high head clean water pump.

Of the pumps forming the group of rotodynamic pumps, the Centrifugal type is the most common. Each pump of this type comprises two principal components:

- The impeller, that drives the water into rotary motion
- The pump casing, which guides the water into the eye of the impeller and leads it away at a higher pressure.

The impeller is mounted on a shaft, supported in bearings and driven through a rigid or flexible coupling by a driver. This is usually an electric motor but could be an internal combustion engine, steam turbine or water turbine.

The pump casing includes the suction and discharge pipe connections, supports the bearings, and houses the rotor assembly. Sealing devices are included to prevent liquid escaping around the shaft and between the impeller and the casing. The detail design of a centrifugal pump is influenced by the nature of the liquid to be pumped. When pumping clean water a closed impeller will be used and efficiencies of 80 to 95% can be achieved. For high pumping heads, multi-stage pumps can be constructed and for boiler feed duties, heads can be in excess of 300 Bar (30 000 kPa)

For the pumping of raw wastewater containing large solids, grit and stringy matter a different type of impeller is required. This requires large passages to allow solids to pass and may be of the closed design (with a plate each side of the vane) or the open design (with vanes mounted on a single rotating plate). Sewage pumps must always be specified to suit the maximum size of solids to be pumped and this may be called "freeway" with a capability of passing, say 80 mm diameter solids or "fullway" with a capacity of passing any solid coming down the suction pipe.

Fullway pumps are usually less efficient than freeway pumps but this disadvantage is offset by the lesser attention that they require. There are many designs of impellers for handling raw sewage and these can be a simple S-shaped vane on a back plate, a single channel closed impeller, an open multi-vane impeller or even a vortex impeller. This last impeller type is recessed into the back cover of the casing and the impeller vanes do not protrude into the pump casing. They work by inducing a strong vortex in the liquid passing through the casing and are virtually un-chokeable. They have a low efficiency rate, around 35%, and can only generate low heads.

Other special types of pump incorporate cutting blades that cut or grind solid matter into pieces small enough to be handled by the pump. These are also less efficient and the cutting blades require regular replacement, especially where grit is present Some pumps are provided with rubber linings to resist the abrasive effects of grit in the sewage.

It can be expected that any sewage pump will eventually become blocked from time to time and pipework should be arranged so that it is possible to quickly access the interior of the pump to clear a blockage. Where possible, a solution is to purchase a pump having "Back-pull-out" construction, in conjunction with a spacer coupling In this design the spacer in the coupling is removed and the bearing bracket, back cover of the casing and rotating element is removed to give clear access to the interior of the pump casing without disturbing the pipework or the motor. Where the particular design of the pump does not provide this feature, adequate handholds with quickly-removable covers should be provided on the pipework.

The centrifugal pump is sometimes known as a "radial flow" pump and has some disadvantages. The freeway or fullway impeller tends to have a flat head/quantity curve so that a small change in operating head produces a large change in quantity pumped. Conversely if the head reduces, the quantity increases and the power requirement also increases so that larger sized motors become necessary to cater for this condition. Where several pumps run in parallel, this phenomenon is exaggerated and the selection of suitable motors must allow for this requirement.

In an axial flow pump, liquid approaches the impeller axially and the forward flow of liquid is parallel to the shaft axis. The impeller resembles a ship's propeller and these pumps are also known as propeller pumps. These pumps are suitable for handling large quantities at low heads but do not have good solids handling capabilities. They are suitable for recirculating effluents or similar duties. The axial flow pump has a steep head-quantity curve so that the quantity varies only slightly over a wide change in head and the power falls off as the flow increases. It does, however, absorb much more power as the flow is reduced towards zero and arrangements should be made to ensure that such pumps can never be run against a closed delivery valve.

The mixed flow pump has impellers that incorporate a combination of the centrifugal impeller with a twisted vane, similar to the propeller of the axial flow pump. Because of this combination, it is possible to design a pump that incorporates the good features of both the radial flow and the axial, to give good solids handling, a fairly steep head-quantity curve and an almost flat power curve This design is best suited for large raw sewage installations with high power motors.

All the rotodynamic pumps may be installed with the shafts horizontal, vertical or at an intermediate angle, although it is most usual for the horizontal arrangement to be adopted, with the driving motor on a single baseplate. This has the advantage of easy access for cleaning and maintenance. The vertical arrangement is preferred when it is necessary to reduce the area of the pump station and the motors can be mounted directly on top of the pumps or located at a higher level, well above any possible flood level. Some designs can be suspended from a baseplate at floor level to hang directly into the wet well or suction sump. This makes maintenance difficult and is not encountered often as the whole pump has to be pulled out of the sump.

All types of pump must be filled with water before starting and the easiest way is to arrange for the pump to be located in a dry well, below the level of the liquid to be pumped. This may not always be possible and several options are available These include the provision of a small vacuum pump to evacuate the air from the pump casing before starting the pump. It is possible to provide a priming tank to allow the pump to fill itself with liquid during start-up but these tend to become clogged with solids on sewage duty. It is also possible to provide a foot valve on the suction pipe and to fill the pump with water before starting This arrangement has two serious disadvantages, in that these foot valves often leak and they present a high head loss to the suction Whatever the method of priming, there must always be an air release valve at the highest point on the pump casing to vent off any accumulated air or gas.

A number of makers are able to supply self-priming sewage pumps that overcome these difficulties. It is necessary to initially fill these pumps with water but thereafter they will rapidly self-prime on start-up. It must be ensured that a reliable automatic air release valve is fitted to the pump, before the non-return valve on the delivery side.

A further variation is the submersible pump, in which the pump is directly connected to a submersible electric motor and is positioned below the surface of the liquid. These are often used for small lift stations in remote areas and have the advantages of being installed in a manhole safe from vandalism and other damage. They are simple but can be expensive to repair.

12.3 POSITIVE DISPLACEMENT PUMPS

These theoretically, deliver a fixed quantity irrespective of the delivery head but in practice, the quantity does drop off as the head increases. The simplest type of positive displacement pump is the reciprocating or piston pump, with a piston moving forwards and backwards in a cylinder, to suck water into the cylinder and discharge it to a higher pressure. This type of pump has been in service for several hundreds of years for water supply and for pumping sewage and other fluids. Many hundreds of farmers' borehole pumps are still of this type, often powered by windmills. They are also commonly used as hand pumps for water supply in small communities.

In the wastewater industry today, reciprocating pumps are usually used as specially designed high head pumps to handle sewage or sludge or in small sizes as chemical dosing or metering pumps, since every stroke delivers the measurable amount of fluid. The reciprocating type of pump cannot produce a steady rate of flow and it is advisable to provide an air vessel, on the discharge side, to dampen the pressure fluctuations. Although these pumps are self-priming, they should not be run dry as they will rapidly overheat and seize.

A number of rotary positive pumps are also found in wastewater plants and these include rotary lobe pumps and progressive cavity pumps. The rotary lobe pump is developed from the much older gear pump and comprises two elastomer-coated rotors connected together by an integral gear system with synchronised timing gears. The two rotors, which typically have two or more lobes, run without touching each other or the outer casing. The liquid is drawn into the inlet port and into the pockets between the lobes and chamber walls and is discharged in the direction of rotation of the outer lobes into the discharge nozzle. The discharge flow is continuous and smooth and is relatively non-agitating and non-shearing. The pump is self-priming and can be run dry for short periods. It is ideally suited to the pumping of a wide range of sludges.

A pump of similar capability, but different construction, in the progressive cavity pump, which comprises a hard steel rotor of helical spiral form which rotates in a stator of natural or artificial rubber with a similar internal helical spiral form As the rotor turns, it contacts the stator, along a continuous sealing line, to create a series of sealed cavities that progress to the discharge end. The cavity fills with liquid as it opens at the suction end and this trapped liquid is transported along the rotor to the discharge and is gradually discharged in an axial direction

This type of pump is self-priming but must never be run dry. The initial starting load is high because of the dry contact between the rotor and the stator, but this drops off immediately once the pump starts to turn.

Progressive cavity pumps are widely used for handling all types of slurries and sludges and can handle small solids. Where significant quantities of grit are present in the sludge, operating speeds must be kept below 350 rpm to minimise wear on the components.

A recent development in the pumping of sludges and slurries is the peristaltic pump. This is a simple design and comprises a semicircular chamber housing a rubber hose A system of rotating rollers progressively squeezes the hose, pushing along any fluid in front of the rollers and sucking more fluid in behind it. A sufficient number of rollers are provided to maintain a constant flow through the hose. This type of pump is used in wastewater plants overseas but has not yet found general application in South Africa. It is self-priming, can be run dry and can handle a certain amount of solid matter. It is vulnerable to damage to the hose by sharp objects, although the hose can be easily and quickly replaced when necessary.

A variation of the reciprocating pump uses a flexible diaphragm in place of the piston This form of construction eliminates the wear between the piston and cylinder but still has the problem of suction and delivery valves becoming fouled or jammed. Various patterns of diaphragm pump are in use and these include mechanically driven versions for dewatering excavations, compressed-air operated versions for sludge handling and a diaphragm dosing pump for metering duties.

12.4 COMPRESSED AIR OPERATED PUMPS

These are of particular use in the pumping of sludges and have the advantage of being adjusted to suit the rate of sludge flow, by varying the length of stroke and the number of strokes per minute. The need to provide a supply of compressed air makes this pump expensive to purchase and to operate

Compressed air is also used to operate several other types of pump used in the wastewater industry. These include the airlift pump, which is comprised of a tube with a supply of compressed air, immersed in the liquid. The mixture of air and water is less dense and the pressure outside the tube forces this lighter mixture to the top of the tube where it is discharged. Airlift pumps can be used for activated sludge return where heads are lower than about 2.0 m and they have no moving parts requiring maintenance. They have limited flexibility in operation and are low in efficiency.

Another compressed air device is the air ejector lift station to lift fairly small quantities of raw sewage to the main outfall sewer. This device comprises a steel vessel arranged to collect the raw sewage. When the vessel is full, a supply of compressed air is blown into it to force the collected sewage up a rising main to the main sewer. These units were in regular use some years ago but have tended to be replaced by small submersible pumps in lift stations. Compressed air can also be used to power ejectors for pump priming or for emptying sumps.

12.5 ARCHIMEDIAN SCREW PUMPS

These comprise a rotating torque tube, carrying one or more sets of spiral flights, located in a concrete or steel trough and supported by a combined thrust and radial bearing at the top and a sealed radial bearing at the bottom Pockets formed between the spiral flights and the trough, trap the liquid and move it up the incline in a continuous manner.

Screw pumps can handle large solids and high grit content and also have the facility of being able to pump the exact quantity entering the pump, from zero flow to full design flow, without any problems. The pump can be left running dry for an indefinite period. The screw pump is rather restricted in terms of head, being limited to about 8.5 m in the larger sizes and reduced as the diameter decreases. This limitation can be overcome to an extent by arranging two pumps in series. It requires a large amount of space and the civil construction costs are high. The design of the bottom, submerged bearing is critical and must be given special attention.

FIGURE 12.5.1: Typical Archimedian screw pumps

12.6 PUMP DRIVERS

Most pumps found in a wastewater plant will be driven by electric motors but other types of drive can also be encountered. These include internal combustion engines, for temporary installations or for emergency service to back-up electric sets, and steam or water turbines. Wherever possible, the driver is connected directly to the pump through a flexible coupling but it may be preferable to incorporate a vee-belt drive or gearbox to change the speed.

In the case of self-priming sewage pumps, it is not possible to adjust the flow by reducing the impeller diameter and these pumps are regularly driven through a belt drive. Progressive cavity pumps are limited to low speeds when required to handle high grit loads and these can also be driven through belts or gearboxes.

The most common electric motor is the squirrel-cage induction type. This has a fixed stator, accommodating the stator winding, and an aluminium rotor, housing a number of rotor bars. The motor design provides a number of pairs of electrical poles and the rotational speed of a motor is given by the formula:

rpm = frequency of supply x 60 Number of pairs of poles

Thus a 2 pole motor would run at a speed of 3 000 rpm and a 4 pole motor 1 500 rpm. This is the theoretical speed and in practice, the actual operating speed is slightly lower because of electrical "slip". The amount of slip is influenced by the size of the motor, larger motors having lower slip than smaller sized motors. A 10 kW 2 pole motor will run at 2800 rpm, whereas a 300 kW motor will run at 2981 rpm under full load conditions. At less than full load, the motor will run slightly faster.

A serious disadvantage of the induction motor was that the speed is fixed by the number of poles. This has now been overcome by the use of variable frequency drives, which enable a standard induction motor to run at any required speed between zero rpm and full rated speed, or even faster than full speed. This variable speed drive has another advantage in that starting is soft and gentle and acceleration can be controlled to reduce starting load on the electrical supply system It can easily be arranged for the motor speed to be controlled by a rate of flow device, so that a pump can match the change of flow of sewage influent to a pump station.

A much simpler version of this same device is the electronic "soft starter" which has the same features but without the variable speed facility. This gives the soft start capability that protects the motor and the supply system. It is possible to arrange for one soft starter to be shared between several motors of the same size and this can reduce initial costs considerably.

12.7 PROTECTIVE DEVICES

The current trend is increasingly towards full automatic control of pump stations and it is important to ensure that the plant is protected against abnormal operating conditions. The motors will always be provided with over-current protection and larger motors may also be fitted with devices to monitor winding temperature, bearing temperature and vibration levels.

Installations that run infrequently must be provided with electric heaters in motors and switch-gear panels to prevent any condensation of moisture. Where motors are not fitted with integral heaters, it is possible to arrange for a low voltage supply through the windings to keep them warm and dry when not in operation.

The pumps must be protected against dry running and this can be done by level switches in the suction sump, a limit switch on the non-return valve to sense that the valve is at least partly open or by actual flow measurement, or a combination of these. The control system must also monitor the discharge pressure and shut down the plant if the pumping head suddenly falls well below design levels.

Other devices can be arranged to detect blocked screens, flooding of the pump house, abnormal gland leakage or even detect attempts to vandalise the installation.

Other control devices may be provided to protect the system against water-hammer or pipeline surge. These may include special control valves or air-filled surge-control vessels and will be specially designed to suit the individual system.

The control system can be interfaced to a central control station by telephone or radio links.
Every pump should be equipped with at least a pressure gauge on suction and delivery branches to provide an indication of its performance.

CHAPTER 13

13. SAFETY ASPECTS

13.1 INTRODUCTION

Two aspects which are of major concern to persons involved in the operation and management of sewage treatment works are:

1. Occupational health hazards

These include exposure to chemicals, dust, fumes, gases and vapours, smoke, biological infection, temperature, humidity, and oxygen deficiency.

2. Physical safety

This includes the prevention of fires and explosions, the protection of persons from having accidental contact with electrical equipment and moving machinery, and general safety aspects such as prevention of falling and drowning and accidents resulting from the handling of materials.

The following proposals are intended to make designers more aware of the operational hazards in order that safety may become a consideration in their future designs. All designs should be in accordance with the Machinery and Occupational Safety Act 1983 (Act No.6 of 1983) and Regulations promulgated in terms of the Act.

13.2 EMPLOYEE WELFARE

The welfare of staff employed on a sewage treatment works should be a top priority. A change room/dining area and ablution facilities should always be provided. These should be in a building situated a short distance away from the operational area and consist of:

1. A change room including adequate seating, a system of two lockers per employee (one for working clothes and the other for personal clothing),wash hand basins, with hot and cold water, soap, disinfectant dispensers and hand towels
2. Shower and W.C, facilities which should remain separate from but linked to the dressing area. Hot and cold water is required for the showers.
3. A dining area which may be incorporated in the ablution building but should remain separate from the other facilities. Wash hand basins with hot and cold water may also be included.

It is desirable that all of the above have wash-down facilities, adequate floor drainage and water-repellent wall surfaces.

13.3 GENERAL SAFETY

13.3.1 FENCING

A sewage works should be fenced to prevent the unauthorised entry of persons.

13.3.2 HANDRAILING

Standard hand railing shall be fitted around all open tanks, sumps and channels whose walls are less than 900 mm above ground level. Hand railing and kick-plates should be installed on all platforms, roofs and catwalks where the level of these is more than 2 m above ground level and on all bridges across tanks.

13.3.3 WARNING SIGNS

Warning signs should be installed in prominent positions. These should preferably be standard symbolic signs which will not only serve the purpose of forewarning visitors, but will also constantly remind those who work in the area of the dangers present.

FIGURE 13.3.1: Examples of correct safety signage

13.3.4 LIGHTING

Exterior floodlighting should be installed for night operations, which may also include plant maintenance, inspections and sampling.

13.3.5 SAMPLING AREAS

Safe areas should be provided for the collection of samples or the setting up of automatic samplers. A simple egg crate type floor incorporating a banded sampling hole with cover and hand railing surround is sufficient.

13.3.6 FIRE

The correct type of fire-fighting apparatus should be located at easily accessible points in laboratories, sludge handling and sludge storage areas and workshops. The choice of apparatus should be discussed with the local Fire Officer.

13.3.7 PIPELINES

All exposed pipe-work should be colour-coded in accordance with SABS 10109-1-1995. An identification chart should be displayed in a prominent place. In particular pipes carrying effluent and any taps on these should be clearly marked.

13.3.8 VALVES

All valves on a works should be standardised for direction of opening and closing, and should be readily accessible.

13.4 GENERAL STRUCTURES

13.4.1 VENTILATION AND LIGHTING

Fixed and cross-ventilation and adequate natural and artificial light should be provided in all buildings. In areas such as engine rooms, laboratories, etc., where smoke, fumes, gases or vapours may be present, forced mechanical ventilation may be necessary. The ventilation outlets should be so located as to prevent contamination of fresh air inlets to other buildings.

13.4.2 FLOORS

Non-slip surfaces to floors, ramps and stairs are necessary especially in buildings where spillages may occur. In addition, wash-down facilities, adequate drainage and water repellent surfaces to walls should be incorporated. Non-slip surfaces can be achieved by impregnating epoxy paint with sand. It is preferable that walking and working areas in storerooms and workshops be clearly distinguished. Where pipes or cables are required to cross a floor, they should be laid in ducts built into the floor and covered with a grating or plate, or in pipes laid under the floors.

13.4.3 STAIRWAYS AND LADDERS

Properly designed stairways of 1 m minimum width are preferred to fixed ladders. Safety cages are essential where ladders over 2.5 m high are used,. For higher ladders and stairways, rest landings are required. Fixed ladders should be incorporated on the inside of all open tanks and wells and life preservers located nearby. Ladders and step-irons are subject to corrosion and unless properly maintained may present a hazard in the long term due to corrosion. Consideration should be given to the use of stainless steel in situations where the rate of corrosion will be high. An alternative solution is therefore to have light aluminium ladders of different lengths available which can be used to gain access to tanks and other structures.

13.4.4 DOORS

Doors leading from buildings, wherein potentially hazardous operations are carried out, should be of the self-closing and outwards opening type. Doors should be large enough to allow machinery to be moved into and removed from the building.

13.4.5 LOCATION OF OPERATIONAL EQUIPMENT

Operational equipment, pipe-work, valves etc., should be located for ease of access, operation and maintenance but if possible these should not be at eye level.

13.4.6 WALLS AND CEILINGS

Walls and ceilings should be painted in light colours.

13.5 LABORATORY

13.5.1 EXITS

Where possible, the design of a laboratory should incorporate two easily accessible exits which are remote from each other.

13.5.2 INTERIOR DOORS

Interior doors leading to or from a potentially hazardous area should be of the self-closing, swing type with a wire-reinforced clear glass panel set at eye level, and clearly marked.

13.5.3 FIRE RESISTANT FINISHES

Wall surfaces, ceilings and furniture should be made of acid and fire-resistant materials.

13.5.4 ADDITIONAL INFORMATION

,Laboratory hoods and fume cupboards should be considered where additional or special ventilation is required.

13.6 WORKSHOP

13.6.1 NOISE

In the workshop building layout, consideration should be given to the reduction of noise.

13.6.2 EXITS

As in the case of the laboratory, two easily accessible exits which are remote from one another should be incorporated in the design.

13.6.3 WASTE DISPOSAL AREA

The workshop area should include a well vented, roofed area where airtight receptacles can be stored for the disposal of solvent and combustible wastes.

13.7 MACHINERY

13.7.1 BUILDINGS

Buildings that house engines, electrical motors and related equipment should, where possible, be located above ground level. Consideration may be given to the location of noisy equipment in isolated or sound-proofed buildings.

13.7.2 HOISTS

Fixed crawl beams, or eyes, should be provided where heavy equipment or machinery may be required to be lifted and should, if possible, be positioned directly above the load. Sufficient space is required between the load and the hoist trolley to enable loads to be lifted and removed without having to dismantle other equipment.

13.8 LAYOUT OF PLANT

Adequate space should be provided between items of plant as this enables ease of operation, removal, inspection and maintenance. Safety guards should be provided to all exposed moving parts - e.g. couplings to motors.

13.9 STOREROOM

13.9.1 STORAGE AREAS

All storage areas should be fireproof structures and have concrete floors. They should be located above ground level, and have adequate ventilation. The area should be protected from direct sunlight since temperature control is essential. Where lubricants, chemicals and gas bottles are required to be stored, separate areas should be provided for each. View windows into the storage rooms with light and ventilation switches located on the outside should be considered in layouts.

13.10 CHLORINE

Leak detection devices should be installed in areas where chlorine is used or stored. Such equipment will signal the leakage of gas. Storage for gas masks should be provided outside each area where chlorine is used or stored.

13.11 PUMP STATION

13.11.1 BUILDINGS

Fluorescent lighting should be installed in dry wells approximately 2.5 m above the floor level to provide adequate lighting for maintenance and operational work and to reduce the amount of shadow. Access to wet wells should preferably be from the outside of a pump station.

13.11.2 PUMPS

Both suction and delivery pipework should be valved to prevent sludge or gas from entering the work area when the pumps are removed or dismantled.

13.11.3 WATER SUPPLY

A water supply under a good pressure should be provided in order to hose down both wet and dry wells.

13.11.4 DRY WELLS, ETC.

Dry wells which house pumping sets should be provided with an automatically-operated gland leakage pump.

13.12 ELECTRICAL INSTALLATIONS

All electrical equipment and wiring should be properly insulated, earthed and adequately marked. Electrical installations or apparatus including cables, lights, meters, switches, instruments etc., should comply with SABS 10108-2005.

13.13 POTABLE WATER

Cross connections between the potable water supply and sludge or sewage pipelines should be prevented at all times.

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