Assets Planning and Delivery Group
Engineering

DS 230 DESIGN GUIDELINE

Waste Stabilisation Ponds

VERSION 2
REVISION 0

OCTOBER 2022

FOREWORD

The intent of Design Standards and Design Guidelines is to specify requirements that assure effective design and delivery of fit for purpose Water Corporation infrastructure assets for best whole-of-life value with least risk to Corporation safety and service standards. Design standards and guidelines are also intended to promote uniformity of approach by asset designers, drafters and constructors to the design, construction, commissioning and delivery of water infrastructure and to the compatibility of new infrastructure with existing like infrastructure.

Design Standards and Guidelines draw on the asset design, management and field operational experience gained and documented by the Corporation and by the water industry generally over time. They are intended for application by Corporation staff, designers, constructors and land developers to the planning, design, construction, and commissioning of Corporation infrastructure including water services provided by land developers for takeover by the Corporation.

Nothing in this Design Standard diminishes the responsibility of designers and constructors for applying the requirements of the Western Australia's Work Health and Safety (General) Regulations 2022 to the delivery of Corporation assets. Information on these statutory requirements may be viewed at the following web site location:

Overview of Western Australia's Work Health and Safety (General) Regulations 2022 (dmirs.wa.gov.au)

Enquiries relating to the technical content of a Design Standard or Design Guideline should be directed to the Senior Principal Engineer, Water Resource Recovery, Engineering. Future Design Standard and Guideline changes, if any, will be issued to registered Design Standard and Guideline users as and when published.

Head of Engineering

This document is prepared without the assumption of a duty of care by the Water Corporation. The document is not intended to be nor should it be relied on as a substitute for professional engineering design expertise or any other professional advice.

Users should use and reference the current version of this document.

DISCLAIMER

This Guideline is provided for use only by a suitably qualified professional design engineer who shall apply the skill, knowledge and experience necessary to understand the risks involved and undertake all infrastructure design and installation specification preparation work.

Water Corporation accepts no liability for any loss or damage that arises from anything in the Standards/Specifications including any loss or damage that may arise due to the errors and omissions of any person. Any person or entity which relies upon the Standards/Specifications from the Water Corporation website does so that their own risk and without any right of recourse to the Water Corporation, including, but not limited to, using the Standards/Specification for works other than for or on behalf of the Water Corporation.

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Any interpretation of anything in the Standards/Specifications that deviates from specific Water Corporation Project requirements must be referred to, and resolved by, reference to and for determination by the Water Corporation's project manager and/or designer for that particular Project.

REVISION STATUS

The revision status of this standard is shown section by section in the table below.

REVISION STATUS						
SECT.	**VER. /REV.**	**DATE**	**PAGES REVISED**	**REVISION DESCRIPTION (Section, Clause, Sub-Clause)**	**REVIEWED**	**APPROVED**
1	**2/0**	**10/2022**	**All**	**New Version/Revision**	**Derek Wilson**	**Derek Wilson**

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DESIGN GUIDELINE DS230
WASTE STABILISATION PONDS

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1 INTRODUCTION

1.1 Purpose

The Corporation has developed a suite of Design Standards and Guidelines. Designers shall comply with these Standards and Guidelines for the definition, design and specification of water services assets being acquired for the Corporation.

The purpose of the Design Standards and Guidelines is to provide:

a) Standards and guidelines applicable in the design of Corporation assets,

b) Explanatory or specific design information, and

c) Information relating to the Corporation preferences and practices which have evolved from more than 100 years of water industry experience.

1.2 Scope

This document explains the Water Corporation's design guidelines for Waste Stabilisation Ponds (WSPs) used in wastewater treatment and provides specific information relating to the Corporation's preferences.

1.3 Design Process

The design process to be followed by Designers is documented in the Corporation's Engineering Design Process and applicable Design Standards.

1.4 Standards

All materials and workmanship shall comply with latest revisions of the relevant codes and standards.

Water Corporation Strategic Product Specifications (SPS), or in their absence the latest editions of Australian Standards, or Water Services Association Australia (WSAA) Codes, shall be referenced for design and specification. In the absence of relevant Australian or WSAA Codes, relevant international or industry standards shall be referenced

In the event of conflict between standards or codes, the following hierarchy shall be used:

1. Statutory requirements of Australia and the State of Western Australia.
2. Water Corporation Strategic Product Specifications, design and construction standards.
3. Australian Standards and Codes of Practice, or Water Services Association Australia (WSAA) Codes.
4. Other international standards or codes acceptable to Australian statutory authorities.
5. Alliance preferred alternative international standards or codes acceptable to Australian statutory authorities as described above.
6. Original Equipment Manufacturers (OEM) design standards.

1.5 Referenced Documents

Documents referred to in this Design Guideline are listed in Appendix A of this Guideline.

For Corporation Standards refer to Section 7 of DS30-01 Glossary - Mechanical.

1.6 Mandatory Requirements

The use of the imperative "shall" denotes a mandatory requirement. Use of verbs other than "shall" such as "will", "should", "may" indicates recommended practice. For Australian and International Standards refer to Section 8 of DS30-01 Glossary - Mechanical.

1.7 Nomenclature

The symbols listed below relate specifically to design calculations for evaporation ponds.

BOD	biochemical oxygen demand (refers to BOD_5 in this document unless otherwise stated)
BOM	bureau of meteorology
$CaCO_3$	calcium carbonate
COD	chemical oxygen demand
DO	dissolved oxygen
EP	equivalent persons
FSA	free and saline ammonia
H_2S	hydrogen sulphide
ISS	inert suspended solids
NH_3	ammonia
NH_3–N	nitrogen as ammonia
NH_4-N	nitrogen as ammonium
NH_x–N	nitrous oxide
O.M.	organic material
ORP	oxidation–reduction potential
OUR	oxygen utilisation rate
PFD	process flow diagram
pH	measure of acidity/alkalinity (from German *potenz* = power, and *H*; the symbol for hydrogen). A logarithmic index for the hydrogen ion concentration in an aqueous solution
PS	pumping station
PST	primary sedimentation tank
PWWF	peak wet weather flow
SOUR	specific oxygen utilisation rate
SRT	solids retention time
SS	suspended solids
STED	septic tank effluent disposal
TKN	total Kjeldahl nitrogen
TN	total nitrogen
TP	total phosphorus
UASB	upflow anaerobic sludge blanket
VFA	volatile fatty acids
VSS	volatile suspended solids
WSP	waste stabilisation pond

1.8 Glossary of Terms

class A evaporation pan	The Class A Evaporation Pan is a standard device for manual measurement of evaporation (Australian Bureau of Meteorology Class A type). The pan represents an open body of water: It is filled with water and exposed on a flat plateau. Size: Height: 255 mm; Diameter: 1225 mm
equivalent population (EP)	A measure of the potential for wastewater contribution equivalent to that from a single person at their normal place of residence.
Helminth	Nematode worm – e.g., Strongyloides, Ascaris, Schistosoma, Taenia etc
peak dry weather flow	(Abbreviated as PDWF) This applies to the daily diurnal flow pattern. As a factor, it is the ratio of the peak hourly flow (usually late morning) to the ADWF measured in the three driest (non-rainfall) months of the year. (Usual range is 1.5 to 2.0)
total Kjehdahl nitrogen	(Abbreviated as TKN.) 'Total Kjeldahl nitrogen' is the total of the organic and ammonia nitrogen present in a given sample.
total nitrogen	(Abbreviated as TN.) 'Total nitrogen' is the total of all the nitrogen (organic nitrogen, ammonia, nitrite, and nitrate) present in a given sample.
peak flow	maximum instantaneous flow able to reach the WWTP. In pumped systems this is the combined instantaneous flow of all pump stations able to deliver wastewater to the WWTP when pumping at the same time. To be used for hydraulic design.
septic tank effluent disposal	(Abbreviated as STED) A STED scheme takes partially treated overflow (effluent) from household septic tanks and delivers it via a pipeline system to the WWTP.

2 OVERVIEW of WSPs

2.1 WSP Processes

WSPs consist of a series of ponds. The three basic types of pond in a treatment train are:

- anaerobic,
- facultative and
- maturation ponds (sometimes consisting of a series of maturation ponds)

A variant system may include ponds with aeration systems added.

A WSP system may comprise a single series of the three pond types, or several such series in parallel.

Wastewater flowing into a WSP system should preferably go through a preliminary treatment stage to remove rags, coarse solids, and other large materials. Preliminary treatment typically includes coarse screening (typically maximum 6mm for aerated ponds), grit removal and, in some cases (usually as an alternative) grinding (or munching) of large inorganic objects.

WSPs provide the conditions for a symbiotic relationship between algae and bacteria:

- Aerobic bacteria break down organic waste to give off carbon dioxide;
- Algae use the carbon dioxide for cell growth and, in turn, give off oxygen in daylight;
- Aerobic bacteria use the oxygen from the algae and diffused from the atmosphere for their own metabolic requirements and new cell production.

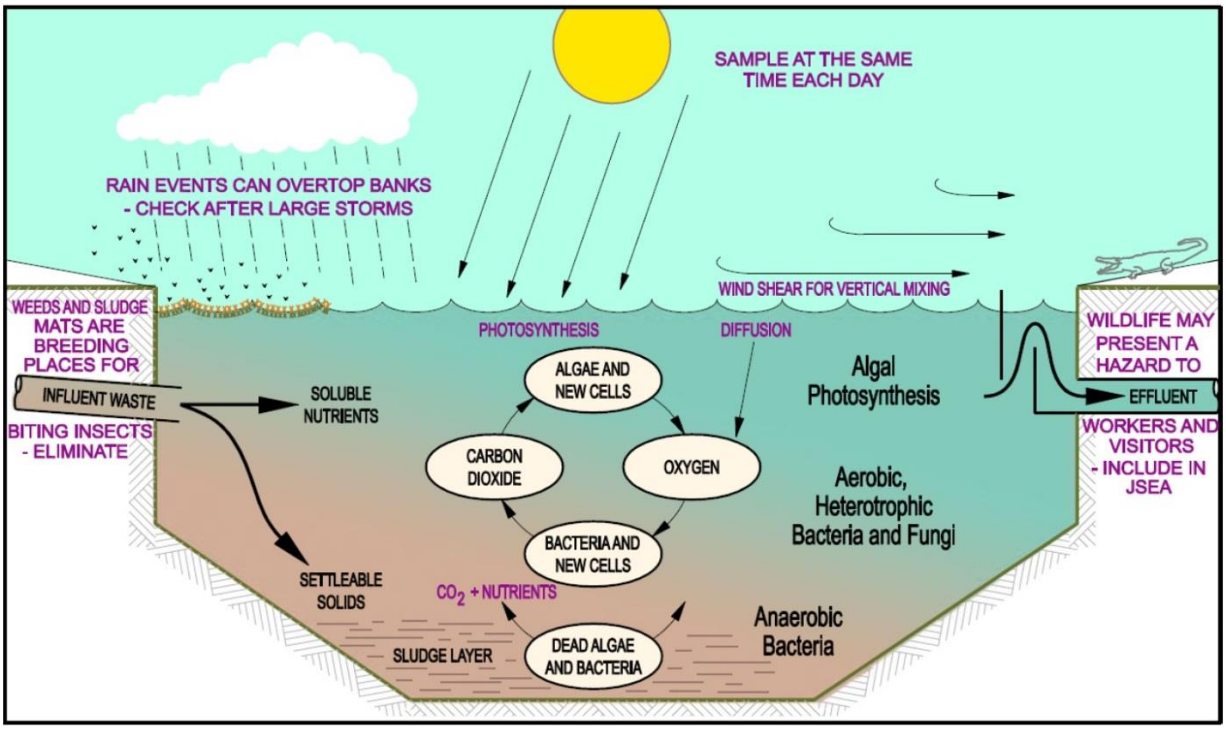


Figure ***Error! No text of specified style in document.****-1: Daytime: algae and bacteria symbiotic relationship (Source: Waste Stabilisation Pond Design Manual – Power and Water Corporation)*

At night, algae take in oxygen and give off carbon dioxide through respiration (see Figure Error**! No text of specified style in document.**-2). The carbon dioxide is stored in the water and provides food for algae during daytime photosynthesis. This storing of carbon dioxide lowers the pond pH. The using up of carbon dioxide by algae during daytime raises the pond pH.

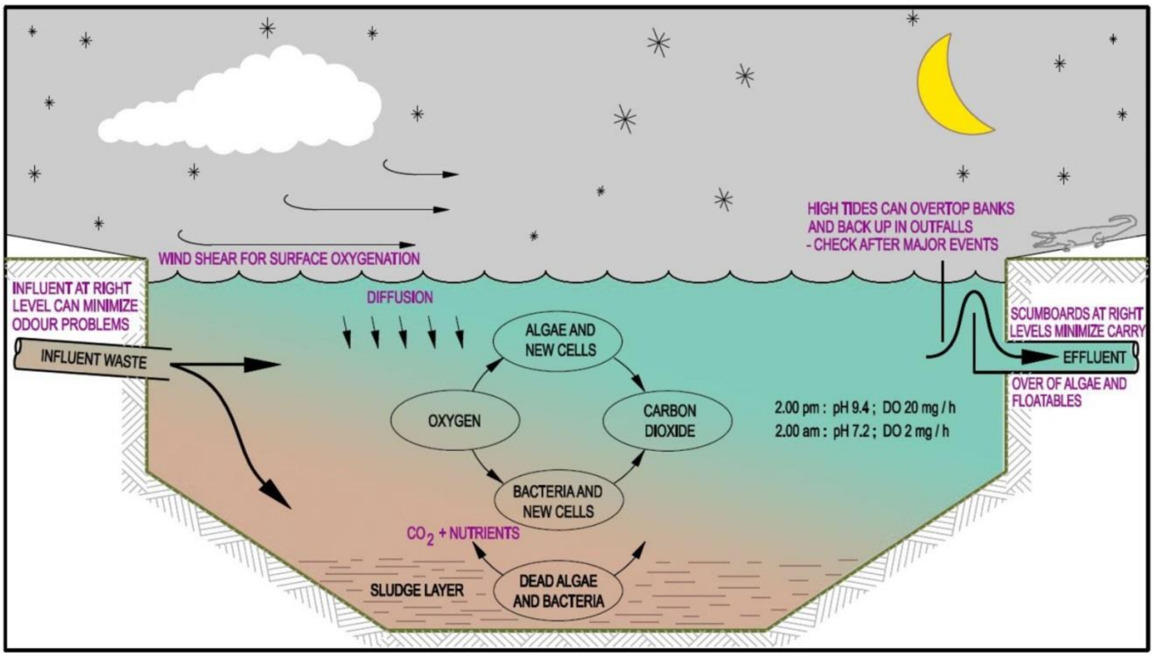


*Figure **Error! No text of specified style in document.**-2: Night time: carbon dioxide from algae respiration (Source: Waste Stabilisation Pond Design Manual – Power and Water Corporation)*

2.2 Preliminary Treatment (Inlet Works or Screening Facility)

Small WSP systems in remote areas often do not include an inlet works screening facility. The reasons for this exclusion are normally:

- the plant is considered too small to warrant a screening facility;
- there is no power available at the site;
- the availability of appropriate disposal methods in a remote area;
- avoidance of this high maintenance facility and associated activities.

In small WSPs, a grinder (or muncher) or grinder pump is sometimes installed to reduce the size of the incoming (largely) inorganic solids (e.g., rags). Although this may have certain benefits, these tend to be high maintenance items which are often neglected in the field until problems occur. The types of problems typically encountered include:

- blockages due to the grinder not being able to deal with particular items;
- wear on the grinder which, although it can lead to blockages in its own right, creates a secondary problem due to rags being "converted" into "ribbons", the end result being worse than the condition would have been without a grinder.

The installation of grinders must therefore be carefully considered, and they should not be installed without proper consideration, and assurance that maintenance will be adequate.

Notes: *It should be noted that the exclusion of a screening facility causes:*

- *the volume of material caught up in the sludge blanket to build up more rapidly, and this should be considered in determining the storage volume in the anaerobic pond (or in the facultative pond if no anaerobic pond is present);*
- *a small increase in maintenance activity due to the increased number of items likely to float to the surface, requiring manual removal;*
- *a reduction in the potential reuse opportunities of both the treated water and pond sludge.*
- *an increase in maintenance activity which has a direct impact on operational costs. This operational cost should be considered as part of the overall cost for the life of the facility.*

2.3 Waste Stabilisation Pond Variants

WSPs can be combined to have several variants, having different levels of operational simplicity and land requirements. The basic systems dealt with in this manual are:

- Anaerobic (followed by secondary facultative);
- Facultative ponds;
- Integrated Anaerobic / Facultative Ponds;
- Aerated ponds divided into:
  - Partially mixed aerated ponds;
  - Complete-mix aerated ponds followed by sedimentation ponds;
- Maturation ponds (often consisting of a series of maturation ponds).

In essence, anaerobic and facultative ponds are designed for removal of soluble Biochemical Oxidation Demand (BOD) and settling of sewage solids, whilst maturation ponds are for pathogen removal, although some BOD removal occurs in maturation ponds and pathogen removal occurs in anaerobic and facultative ponds. Most existing plants do not include anaerobic ponds, but this is not recommended in the modern context where the use of Integrated Anaerobic / Facultative ponds presents a more economic construction/civil option in addition to operational efficiency.

Generally, in WSP systems, wastewater flows from the anaerobic pond to the facultative pond and finally, if required, to the maturation pond(s).

Tertiary treatment by way of filtration, disinfection or other chemical dosing does not form part of this manual.

2.4 Pond configurations

Figure Error**! No text of specified style in document.**-3 below shows a selection of some typical pond configurations. This is a selection only, and many other configurations are possible.

Notes:

- *Maturation ponds could be seen as optional, and the diagram could represent a series of maturation ponds to achieve particular levels of disinfection.*
- *Ponds in series will always produce better results than ponds in parallel due to the cumulative treatment benefit achieved.*
- *Ponds in parallel provide facility for offline maintenance where required, assuming sufficient isolation valving is available.*

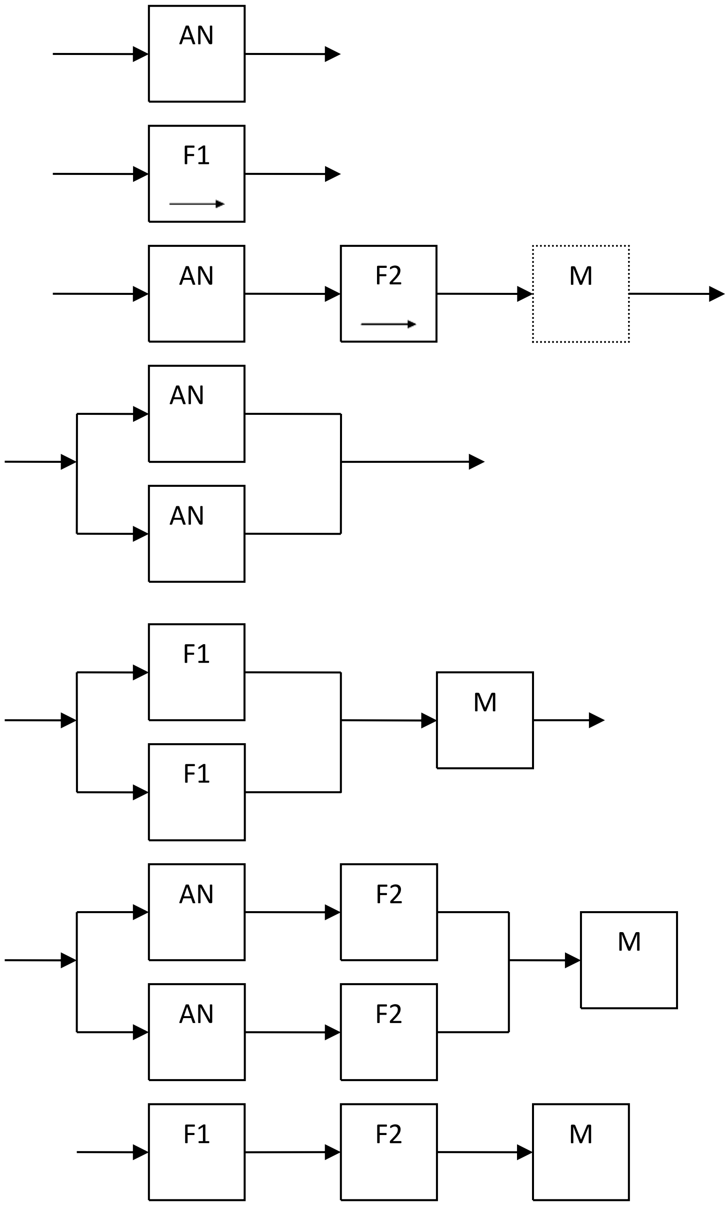


*Figure **Error! No text of specified style in document.**-3: Possible WSP configurations*

AN = Anaerobic, F1 = Primary Facultative, F2 = Secondary Facultative, M = Maturation. A series of Maturation ponds would be labelled M1, M2, and M3 etc.

2.5 Anaerobic ponds

General description

Anaerobic ponds are small but deep ponds that exclude oxygen and encourage the growth of anaerobic bacteria. They are very efficient at breaking down the influent BOD load. As they break down the BOD, the anaerobic bacteria release methane and carbon dioxide. Sludge is deposited on the bottom and a crust may form on the surface. Being anaerobic, it is expected that ammonia and alkalinity will increase.

Anaerobic ponds are commonly 3-5 m deep. It is preferred that anaerobic ponds be at least 4 to 5m deep.

Process features

Anaerobic ponds receive the initial high organic loading coming into the plant (> 100 mg/L BOD). Anaerobic conditions are maintained for the majority of their depth, and algae do not proliferate. The surface layer should, however, be maintained as an algae rich layer to prevent odour release by recycling algae rich water from the facultative or the maturation ponds onto the surface layer of the anaerobic pond. They work extremely well in warm climates with the following broad characteristics for a temperature above 20ºC

- BOD removal 60-85%
- Short retention time 1 day minimum (for influent BOD of $\leq$ 300 mg/L at 20°C)

Normally, a single anaerobic pond in each treatment train is sufficient if the strength of the influent wastewater is less than 1,000 mg/L BOD_5. For high strength industrial wastes, up to three anaerobic ponds in series might be justifiable but the retention time in any of these ponds should not be less than 1 day.

Odours are a risk with Anaerobic ponds but can be dealt with (see section 1.11).

Figure Error**! No text of specified style in document.**-4 is a diagrammatic cross section of an anaerobic pond. Note the following with respect to this diagram:

- The benched arrangement is not a specific requirement and MUST NOT exceed about 2m in width. The purpose of the bench section is:
    - to allow excavation of the deeper section during the construction phase as long reach excavators cannot always reach the bottom;
    - to provide a platform to install outlet structures;
    - to make provision for the installation of embankment protection (e.g., rip-rap) to the upper section of the pond only
- The deeper section usually has steeper embankments and is often concrete lined. Vertical walls are also acceptable if circumstances and costs allow;
- A mid-water depth discharge with the discharge point pointed downwards is required;
- Discharge at the bottom of the pond is not acceptable.

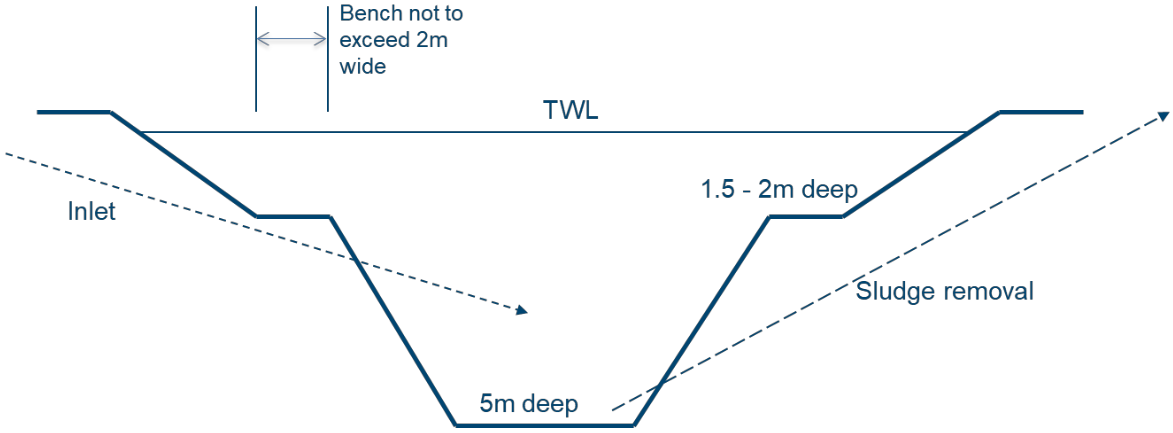


*Figure **Error! No text of specified style in document.**-4: Typical cross section through an Anaerobic Pond*

$$\text{Organic material} \xrightarrow[\text{Organisms}]{\text{Anaerobic}} \text{Anaerobic Organisms} + CO_2 + CH_4 + H_2O$$

Biogas may be collected using a gas canopy and used for energy production. For domestic wastewater applications this is usually not viable from a cost / benefit perspective.

Issues to be fully considered before constructing anaerobic ponds include the following:

- The risk of odour and ensuring that it is properly dealt with. (See section 2.12);
- Subsurface conditions (rock or sandy conditions) which would impact the cost of construction;
- The cost of construction of anaerobic ponds vs the cost of expansive facultative ponds as an alternative, and particularly where Integrated Anaerobic / Facultative ponds can be used.

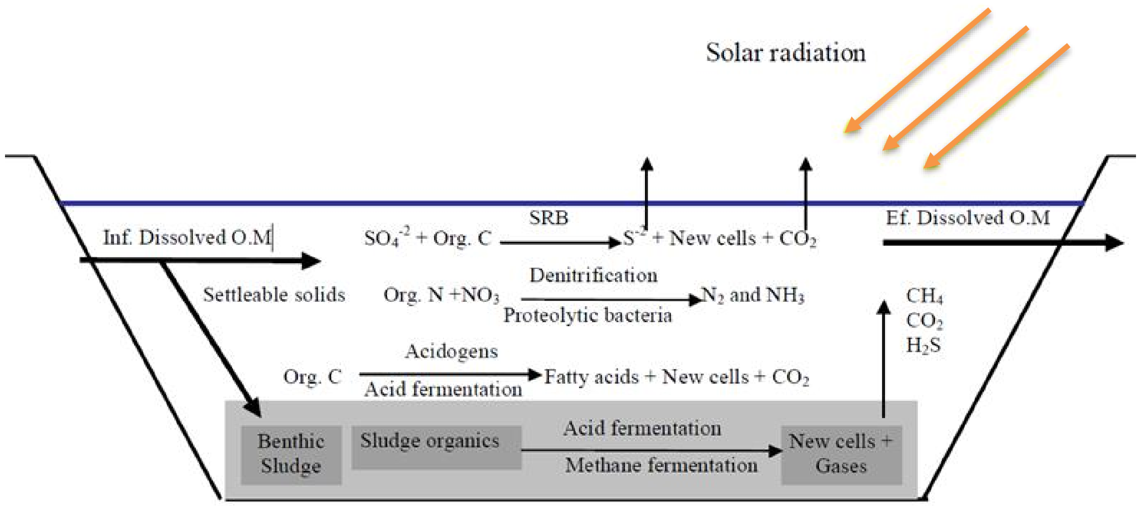


*Figure **Error! No text of specified style in document.**-5: Anaerobic Pond Activity*

Function

Anaerobic pond activity is illustrated in Error! Reference source not found.. Anaerobic ponds reduce N, P, K, and pathogenic microorganisms by sludge formation in which anaerobic bacteria break down the organic matter in the wastewater, releasing some ammonia into the atmosphere, and adding to the ammonia fraction entering the next pond.

The anaerobic pond therefore serves to:

- separate the solids from dissolved material, with the solids settling out;
- dissolve further organic material;
- break down biodegradable organic material;
- store undigested material and non-biodegradable solids as bottom sludge;
- allow partially treated effluent to overflow to the next pond (usually a secondary facultative pond);
- add to the ammonia concentration entering the next pond.

Sludge removal

Sludge removal from anaerobic ponds remains problematic. See Section 6.3 of this guideline. Anaerobic ponds can be fitted with several sludge withdrawal pipes connected to a manifold outside of the pond from where the sludge can be withdrawn. In some cases, this will be with trailer mounted sludge pumps. Rat-holing is a key problem for sludge withdrawal though, so regular (6 to 12 monthly) cycles of sludge withdrawal should be built into the operating instructions. These approaches are however not preferred.

An alternative to sludge withdrawal through a series of pipes is to use a dredge with an arm able to reach down to at least 5m. This method provides certainty and is also able to be carried out with the pond “on-line”. If a dredge is used, sludge can be left for longer periods before being withdrawn and sludge can be allowed to build up to about 30% of the anaerobic pond volume before being removed.

Guidance: Physical features of anaerobic ponds

Feature	Comment
Depth	• 5.0m depth is preferred. Depth may range from 3 to 5 deep
Inlet	• Discharge is to be at mid-water depth for both gravity and pumped systems • The discharge point is to be directed downwards.
Outlet	• Should be at the furthest point from the inlet when considering the flow path. • Outlet to be fitted with an underflow baffle to prevent scum transfer. • Underflow baffle depth of 300mm is preferred. A depth of at least 100mm is required. Where algae rich water is floated over the anaerobic pond it will be necessary to make the outlet depth similar to facultative ponds i.e., from 500 to 600mm.
Embankments	• Normally 1:1 (preferred) to 1:3 internally and 1:3 to 1:4 externally to aid maintenance. *[Note: The slope 1: N refers to 1 vertical and N horizontal]* • In Integrated Anaerobic / Facultative Pond applications, the anaerobic zone is preferably 1:1 internally, and concrete lined • Freeboard ≥ 500mm (measured vertically) • Must be protected from wave action, particularly if clay lined. Rip-rap works well to dissipate energy. • Must be designed to prevent growth of vegetation at the tops of embankments. This aids maintenance and prevents mosquito breeding. • Gauge board to be installed for indication of freeboard level.
Deep section (if benched)	• Often concrete lined and built at 1:1 gradient to aid settlement of sludge to the bottom. (Sludge "hangs up" with flatter grades) • The lower section may have vertical walls if appropriate or the entire depth may be vertical in smaller systems • Must provide for regular desludging
Pond bottom	• Should be level (a gentle slope to a low point to aid desludging is acceptable)
Pond shape	• Generally square or rectangular with a length to width ratio not exceeding 3:1

Effluent from an anaerobic pond (or series of anaerobic ponds) flows to a secondary facultative pond for further treatment.

2.6 Facultative Ponds

General description

Facultative ponds are large, shallow ponds which cover large areas. They are normally 1.2 - 2 m deep. ***A depth of 1.5m is preferred and shallower ponds should not be chosen without good reason***.

Two types of facultative pond are defined, being:

- primary facultative ponds, which receive raw wastewater (i.e., the system has no anaerobic pond) and
- *secondary facultative ponds*, which receive wastewater from:
    - a primary facultative pond, or
    - an anaerobic pond, or
    - an anaerobic tank or Imhoff tank, or
    - a STED system.

Process features

Facultative ponds are designed for BOD removal on the basis of a relatively low surface loading rate (e.g., 100-400 kg BOD/ha/d at temperatures between 20°C and 25°C). This permits the development of a healthy algal population, as the oxygen for BOD removal by the pond bacteria is mostly generated by algal photosynthesis. Oxygen is also transferred to the surface layers through wind action across the pond. The algae cause the facultative ponds to be coloured dark green, although they may occasionally appear red or pink (especially when slightly overloaded) due to the presence of anaerobic purple sulphide-oxidizing photosynthetic bacteria. The concentration of algae in a healthy facultative pond depends on loading and temperature but is usually in the range 500-2,000 μg *chlorophyll a* per litre.

The ponds have an upper aerobic layer, with an anaerobic sludge layer on the floor, and a facultative layer between. The surface zone, being the most irradiated, experiences high levels of microbial growth and oxygen production, creating the aerobic zone. With depth, there is less sunlight penetration available for photosynthesis, and thus less oxygen production. Eventually, the dissolved oxygen is depleted, leading to an anoxic (facultative) zone. Suspended solids entering the system as well as dead microalgae settle to the bottom of the pond to form an anaerobic layer where sludge is digested by anaerobic bacteria.

Algae populations require sunlight. They develop and produce oxygen in excess of their own metabolic requirements. It is this excess of oxygen that is utilised by bacterial autotrophs for the removal of BOD. The algal production of oxygen occurs near the surface of facultative ponds and to the depth to which light can penetrate (typically up to about 500 mm).

It is important to note this depth because water transferred from the facultative pond to the next pond should be withdrawn from 500 to 600mm (preferably 600mm) below the surface through a baffled system to ensure that algae is not unnecessarily transferred from pond to pond in the process.

As a result of the photosynthetic activity of algae, there is a diurnal variation in the concentration of dissolved oxygen (see **Figure Error! No text of specified style in document.**-7: Diurnal variation of dissolved oxygen in a facultative pond (Source: Mara, 2003)). Following sunrise, the dissolved oxygen level gradually rises in response to photosynthetic activity to a maximum in the mid-afternoon, after which it falls to a minimum during the night when photosynthesis ceases and algal (and bacterial) respiratory activity begin to consume oxygen. The position of the 'oxypause' (the depth at which the dissolved oxygen concentration reaches zero) also changes, as does the pH, because, at peak algal activity carbonate and bicarbonate ions react to provide more carbon dioxide for the algae, leaving an excess of hydroxyl ions resulting in a pH rise to > 9.4, which rapidly kills most faecal bacteria.

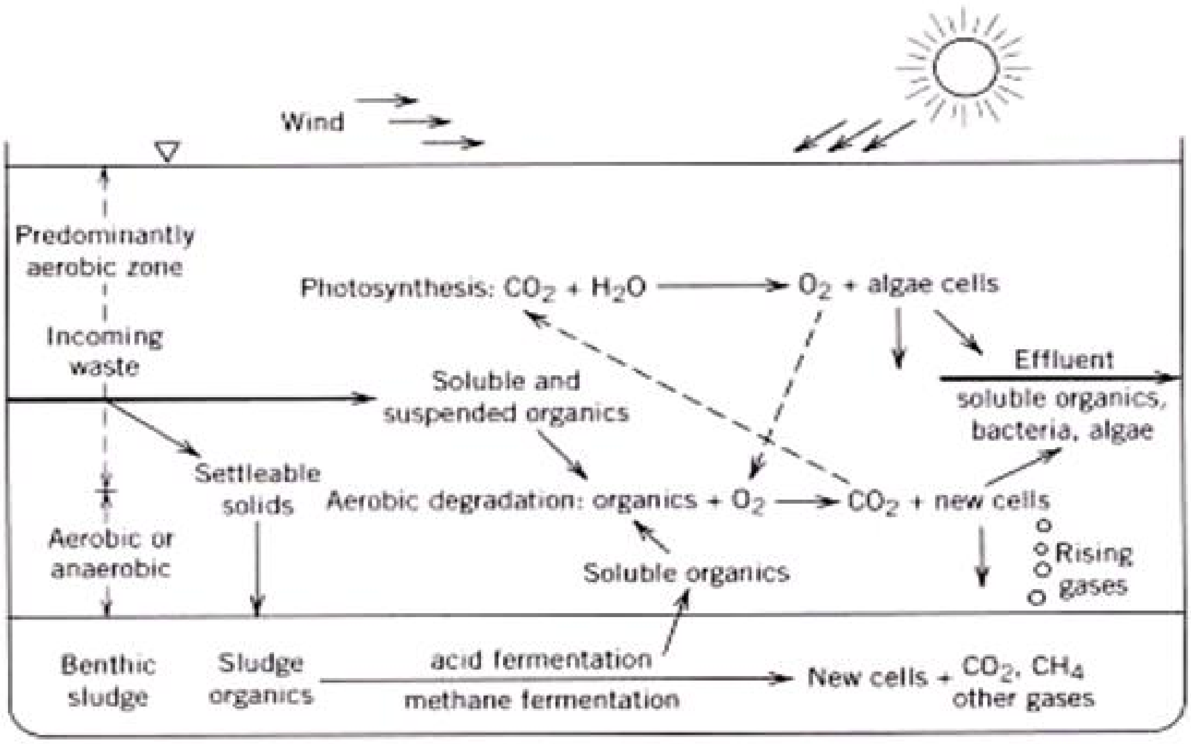


Figure ***Error! No text of specified style in document.****-6: Typical cross section through a Facultative Pond*

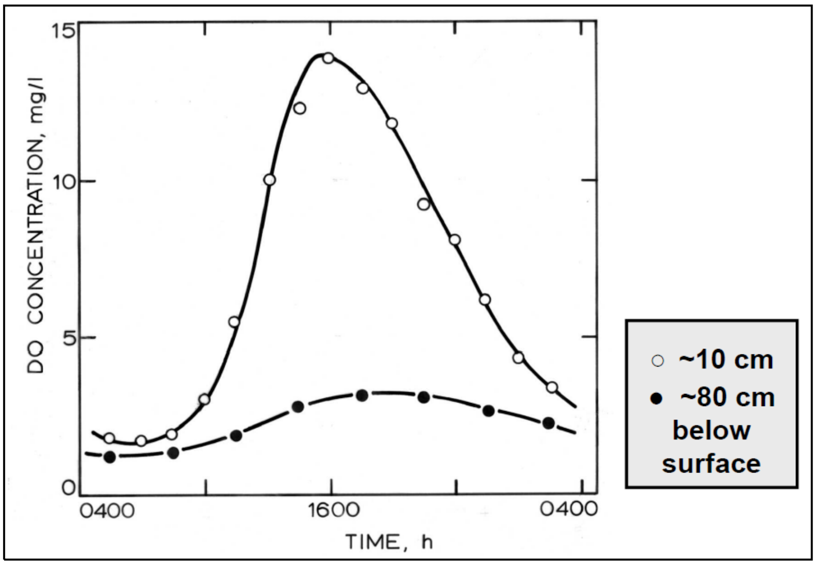


Figure ***Error! No text of specified style in document.****-7: Diurnal variation of dissolved oxygen in a facultative pond (Source: Mara, 2003)*

Function

Effectively operating primary and secondary facultative ponds should remove soluble BOD, settle sewage solids, reduce onsite odours, and provide quantifiable pathogen reduction.

Facultative ponds serve to:

- treat the wastewater anaerobically through separation, dissolving and digestion of organic material. (It should be noted that a primary facultative pond will exhibit a deeper anaerobic zone at the bottom than a secondary facultative pond, and will also build up a sludge layer much more rapidly);
- aerobically break down most of the organic solids near the pond surface;
- reduce the number of disease-causing microorganisms;
- allow some loss of ammonia contained within the wastewater to be released into the atmosphere through volatilization, although recent studies have demonstrated that most of the ammonia is taken up by the algae for synthesis;
- store residues from digestion, as well as non-degradable solids, as bottom sludge.

In primary facultative ponds (those that receive raw wastewater) the above functions of anaerobic and secondary facultative ponds are combined. Around 30% of the influent BOD leaves a primary facultative pond in the form of methane.

Primary facultative ponds should be used for lower strength wastewater (BOD < 400mg/L) and in locations where anaerobic ponds would be unacceptable or difficult to construct.

Where the source water is from a STED scheme, the first pond should be designed as a secondary facultative pond.

Guidance: Physical features of facultative ponds

Feature	Comment
Depth	• **1.5m depth is preferred.** Depth may range from 1.2 to 2m deep
Inlet	• Preferably at mid-water depth but may be between mid-water depth and the bottom to prevent thermal short circuiting. • Should ***not*** be directed towards the outlet. If unavoidable, an inlet baffle should be used to dissipate the influent velocity. • The inlet pipe is to be turned parallel with the entry or shortest embankment.
Outlet	• Should be at the furthest point from the inlet when considering the flow path. • Outlet is to be fitted with an underflow baffle to prevent short circuiting and minimise algae transfer. • Underflow baffle depth of 600mm is preferred. Depth may range from 500 to 600mm. • Spillways between ponds protect flows over the embankments and washouts
Embankments	• Normally 1:3 internally and 1:3 to 1:4 externally to aid maintenance. • Freeboard ≥ 500mm • Must be protected from wave action, particularly if clay lined. Rip-rap works well to dissipate energy. • Freeboard must consider the pond maximum dimension, wind-fetch, and embankment run-up. • Must be designed to prevent growth of vegetation at the tops of embankments. This aids maintenance and prevents mosquito breeding.
Pond bottom	• Preferably level but may be slightly graded. Hollow pockets are not acceptable.
Pond shape	• May vary with topography. The key feature is to prevent short circuiting and dead spots. Generally, a width to length ratio of 1:2 to 1:3 is preferred although 1:4 is acceptable if regular desludging (especially at the front end) is ascertained. Long narrow facultative ponds tend to form "high load" sub-zones at the front end that become anaerobic and potentially odorous.
Baffles	• Baffles in facultative ponds create subzones which are likely to cause exceedance of the pond loading limit (or even the pond failure envelope) and should not be used except as short deflection baffles designed to avoid short circuiting. • Stub baffles to help direct pond momentum are useful. See 5.5.2 • Outlet shields help prevent sludge build-up at outlets when prevailing winds drive scum in that direction. See 5.5.2

Effluent from secondary facultative ponds flows to maturation ponds but may go directly to tertiary treatment facilities if they are adequately designed for this purpose.

2.7 Integrated anaerobic / facultative ponds

An appropriate approach, designed to benefit from the advantages of both anaerobic and facultative ponds whilst mitigating their respective disadvantages is to integrate them into a single pond (See **Figure Error! No text of specified style in document.**-8**8** below). The Integrated Anaerobic / Facultative Pond has a deep (anaerobic pit) zone, into which the raw sewage is fed, to promote settling of solids and anaerobic treatment of the influent under normal anaerobic pond conditions. The anaerobic pot effluent rises to flow into the upper layer of the pond which operates as a secondary facultative pond. The pond is designed so that algae rich water from the upper facultative zone remains aerobic and algae rich, thus reducing (and eliminating) the risk of odour nuisance. This feature is further aided (if possible) by allowing the prevailing wind to drive algae rich water along the length of the facultative pond and over the surface layer of the anaerobic end of the pond (**Figure Error! No text of specified style in document.**-9).

From a design perspective the anaerobic pit (i.e., the portion (or volume) below the facultative pond, and below 1.5m deep) is sized in the same manner as any anaerobic pond, and the facultative pond is designed as a secondary facultative pond in the normal manner.

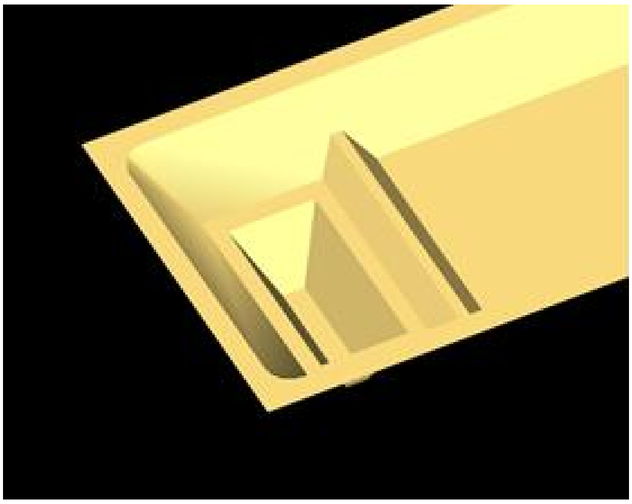


*Figure **Error! No text of specified style in document.**-8: Image of an Integrated Anaerobic / Facultative pond*

Guidance: Physical features of integrated anaerobic / facultative ponds

Guidance for the design of Integrated Anaerobic / Facultative ponds is the same as for the individual ponds in sections 2.5 and 2.6 above.

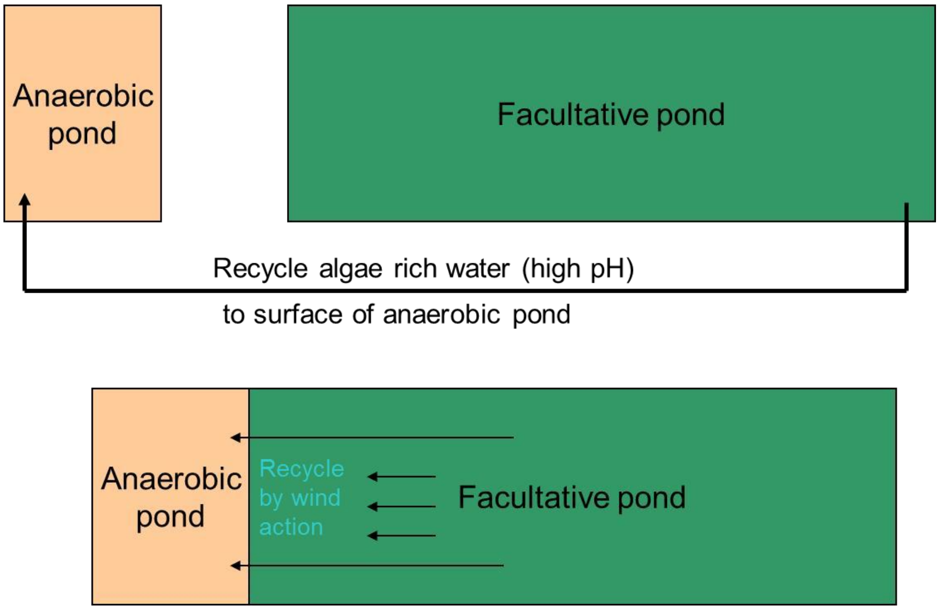


*Figure **Error! No text of specified style in document.**-9: Integrated Anaerobic / Facultative Pond (below) to aid in odour control*

Effluent from Integrated Anaerobic / Facultative ponds flows to maturation ponds

2.8 Maturation ponds

General description

Maturation ponds receive their flow from secondary facultative ponds, aerated ponds or settling ponds (operating in conjunction with aerated ponds).

Maturation ponds are primarily designed and sized for the removal of pathogens. They also act as polishing ponds for the removal of BOD, nutrients (nitrogen and phosphorus) and suspended solids.

Process features

Maturation ponds are shallow, being 0.85 to 1.2m deep (***around 1m depth is preferred***) to allow light penetration to the bottom and aerobic conditions to prevail throughout the pond depth. High width to length ratios of up to 1:10 are acceptable to achieve a plug-flow regime. They demonstrate less stratification throughout their depth than facultative ponds.

A warning is in order: Although a depth of 1m is preferred, a depth of less than 1 m in clay lined ponds may encourage the growth of macrophytes that are able to establish themselves in shallow ponds. Making ponds less than 1m deep should therefore be avoided.

The size and number of maturation ponds needed in series is determined by the retention time required to achieve a specified effluent pathogen concentration. Very high rates of pathogen removal are achieved in correctly sized maturation ponds, with a 6-log reduction possible through 3 maturation ponds in series.

In the absence of effluent limits for pathogens, maturation ponds can act as a buffer for facultative pond failure whilst remaining useful for nutrient removal.

In a series of maturation ponds, the algal diversity increases, but the biomass decreases from pond to pond.

The key mechanisms for the removal of pathogenic organisms are:

- Bacteria and viruses: temperature, solar radiation, pH, food shortages, predator organisms and toxic compounds;
- Protozoan cysts and helminth eggs: sedimentation.

Maturation ponds can be compared with the upper aerobic layers of facultative ponds, which demonstrates why they must be shallow to achieve:

- High penetration of solar (UV) radiation which aids disinfection;
- High pH due to photosynthetic activity (which can also aid in the removal of phosphorus);
- High DO concentration which is more efficient at coliform removal.

Maturation ponds should reach high levels of coliform removal (E > 99.9 or 99.99%) by employing one of two configurations:

- Three or four ponds in series;
- A single pond with baffles creating a long plug-flow system.

Photosynthesis in both facultative and maturation ponds contribute to an increase in pH. In conditions of high photosynthetic activity (which is more prevalent in a series of maturation ponds), the pH can rise to above 9.0, providing ideal conditions for the stripping of ammonium in addition to the consumption of ammonium by the algae. [Note: Although some claims of high levels of ammonium removal by volatilisation exist, this has not been supported by pilot scale studies].

Organic phosphorus taken up by the algae and bacteria is removed from the system with the final effluent. Phosphate, which represents the higher proportion of phosphorus in the system, precipitates under conditions of pH > 9.0, with removal levels as high as 60 to 80%.

Function

Although some removal of pathogenic organisms (bacteria, viruses, protozoan cysts, and helminth eggs) occurs in anaerobic and facultative ponds, the main purpose of maturation ponds is for pathogen removal.

Maturation ponds serve to:

- Remove pathogenic organisms;
- Polish the effluent from upstream processes which includes BOD, and nutrients;
- Constitute an alternative to disinfection by more conventional means such as chlorine;
- Store residues of digestion as bottom sludge.

Guidance: Physical features of maturation ponds

Feature	Comment
Depth	• 1.0m depth is preferred. Depth may range from 0.85 to 1.2m deep
Inlet	• Preferably at mid-water depth but may be between mid-water depth and the bottom to prevent thermal short circuiting. • If unavoidable, an inlet baffle should be used to dissipate the influent velocity. • The pipe turned parallel with the shorter embankment.
Outlet	• Should be at the furthest point from the inlet when considering the flow path. • Outlet is to be fitted with an underflow baffle to prevent short circuiting and minimise algae transfer. • Underflow baffle depth of 400mm is preferred. Depth may range from 100 to 500mm. A depth of 100mm will only apply to the last pond in circumstances where there is a series of at least 3 maturation ponds or where some special circumstances require this.
Embankments	• Normally 1:3 internally and 1:3 to 1:4 externally to aid maintenance. • Freeboard ≥ 500mm • Must be protected from wave action, particularly if clay lined. Rip-rap works well to dissipate energy. • Freeboard must consider the pond maximum dimension, wind-fetch, and embankment run-up. • Must be designed to prevent growth of vegetation at the tops of embankments. This aids maintenance and prevents mosquito breeding.
Pond bottom	• Preferably level but may be slightly graded. Hollow pockets are not acceptable.
Pond shape	• May vary with topography. The key feature is to prevent short circuiting and dead spots. Generally, a length to width ratio of 1:2 to 1:3 is acceptable, but maturation ponds benefit from plug flow conditions, so that aspect ratios of 1:10 are quite acceptable. A long plug flow regime may also be achieved with baffles.
Baffles	• Baffles in maturation ponds can be very effective in improving pathogen removal efficiencies because the promote plug flow.

Effluent from maturation ponds is returned to the environment or directed for reuse, preferably following filtration to ensure that suspended solids levels are minimised.

2.9 Aerated facultative ponds (partially mixed aerated ponds)

General description

Partially mixed aerated ponds are nominally facultative because the level of aeration energy introduced to the ponds is sufficient to meet the aeration requirement only. The oxygen is supplied by mechanical aerators instead of through photosynthesis. The introduced aeration energy is not sufficient to meet mixing requirements of the entire pond, and sludge will settle in the pond for anaerobic decomposition. A complete mix system requires up to 10 times more energy to maintain all solids in suspension.

Important notes

The use of aerated facultative ponds *is not* the preferred approach to WSP systems. In new plants they should only be considered if all other options have been soundly dismissed. They can be useful when a compact plant with a small footprint is required.

Overloaded conventional facultative ponds with no area to expand could be converted to aerated facultative ponds by the addition of aerators. This is not ideal though as the ponds would be too shallow, and aeration efficiency is compromised. In considering the upgrading of existing facultative ponds, priority should be given to the use of an Integrated Anaerobic / Facultative pond conversion. The cost of aeration is significant, and whole of life (WoL) cost comparisons should be completed to prove that aerated facultative ponds are in fact more cost effective.

When converting conventional facultative ponds to aerated facultative ponds, the following must be considered:

1. Depth of the pond: Aerated ponds should normally be about 3m deep, but if a conventional facultative pond of about 1.5m deep is converted steps need to be taken to ensure that the floor of the pond (clay or other liner type) is protected from damage;
2. Embankment protection from wave action caused by the aerators;
3. Pond hydraulics and mixing, and how these are affected by the installation of aerators;
4. Sludge settling and removal, and how this will be safely achieved;
5. The type of aeration equipment installed. Brush aerators which aim to minimize disturbance with depth may be a good option;
6. Safe access to the aerators for maintenance. A simple fixed, floating or concrete jetty can be useful;
7. Disinfection is reduced, and Helminth egg removal is not possible.
8. Level of noise caused by the aeration system

Process features

In terms of the distribution of the heterotrophic biomass the pond behaves like a conventional facultative pond. The design is similar to that of facultative ponds with respect to the kinetics of BOD removal. They can be designed with up to 3 cells in series with most of the required aeration being applied to the first cell. In systems with fewer cells, the aerators are normally positioned at the front end, allowing the settlement of sludge at the back end. ***If adequate space for sludge settling (i.e., a dedicated settling zone at the end of the facultative pond) is not provided prior to flowing to the maturation pond, a settling pond is required similar to that provided for complete mix aerated facultative ponds.***

As the pond is partly aerated, the solids settle to form a bottom sludge layer with settling zones and sludge depths being determined by the impact of the aerators.

Only the soluble BOD and finely particulated BOD are held in suspension and undergo aerobic decomposition.

Aerated ponds are not as effective as facultative ponds in removing ammonia or phosphorus.

Diurnal changes in pH and alkalinity that improve removal rates for ammonia and phosphorus in facultative ponds do not occur in aerated ponds.

Function

A benefit of aerated ponds is that they can achieve good nitrification of ammonia ***if sufficient retention time is allowed*** for the growth and maintenance of nitrifying bacteria, whilst achieving reliable BOD removal.

Disadvantages of aerated ponds are:

1. They are more complicated to design and construct;
2. They have high operation and maintenance costs;
3. Increased reliance on operator attendance;
4. Sludge removal is more frequent than normal facultative ponds because of zonal build-up, with the added complication of having to remove / relocate aeration equipment for the desludging operation;
5. Reduced treatment capability and capacity during desludging operations.

The key design criteria are detention time and depth.

Detention time:	Usually in the range of 2 to 10 days – must be sufficient for adequate BOD removal.
Depth:	Requires an aerobic layer of about 2m to oxidise the gases from anaerobic decomposition of the sludge layer. A depth of 2.5 to 4.0m is required.

Notes:

- *At times it has been necessary to add aerators to existing facultative ponds because they have become overloaded, or there is a need to increase capacity.*
- *Although this practice is strongly discouraged, cost imperatives often leave the designer with little alternative.*
- *In general, existing facultative ponds will not comply with the depth requirements of aerated facultative ponds.*
- *In these cases, vertical shaft aerators are problematic due to floor scouring and limited ability to distribute air over a wide area leading to poor aeration efficiency.*
- *In retrofitted shallow facultative ponds brush aerators are likely to prove to be useful, as they are designed to deliver air horizontally across the surface of the pond.*

Guidance: Physical features of partially mixed aerated facultative ponds

Feature	Comment
Depth	• 3m preferred, but 2.5 to 4.0m is acceptable
Inlet	• Not as critical as normal facultative ponds because aeration helps mitigate short circuiting issues, but similar principles should still be considered. • To be positioned a safe distance from the nearest aerator, and with due consideration for maintenance access to the aerators.
Outlet	• Should be at the furthest point from the inlet when considering the flow path. • Outlet to be fitted with an underflow baffle to prevent transfer of scum and minimise algae transfer. • Where a settling pond is provided, the underflow baffle may be as little as 100mm below the surface, but an adjustable baffle is recommended. • With no settling pond, an underflow baffle depth of 500 to 600mm is required.
Embankments	• Normally 1:3 internally and 1:3 to 1:4 externally to aid maintenance. • Freeboard ≥ 500mm • Must be protected from wave action, particularly energy from the aeration system. Rip-rap works well to dissipate energy. • Freeboard must consider the pond maximum dimension, wind-fetch and embankment run-up that may also be caused by the aerators. • Must be designed to prevent growth of vegetation at the tops of embankments. This aids maintenance and prevents mosquito breeding.
Pond bottom	• Preferably level but may be slightly graded. Hollow pockets are not acceptable. • Concrete pads are required below vertical shaft aerators to prevent scouring of clay liners or damage to plastic liners.
Pond shape	• May vary with topography but are often rectangular.

Effluent from an aerated facultative pond normally flows either to:

1. *a settling pond, or*
2. *a maturation pond which would incorporate settling.*

2.10 Complete-mix aerated ponds followed by sedimentation ponds

General description

The comments made in section 2.9 in relation to the use of aerated facultative ponds are even more relevant to complete mix aerated ponds, mainly because of the energy requirements to achieve complete mixing.

A settling pond following a complete-mix aerated pond is strongly recommended. ***Where a settling pond is not provided,*** sufficient space (area and depth) must be provided at the front end of the maturation pond for settling. Regular removal of settled sludge from the settling pond is a process requirement and must be adequately catered for.

Process features

Complete mix aerated ponds are activated sludge plants operated without a sludge return. They can treat raw wastewater or effluent from anaerobic ponds. The aerators serve two functions, being:

1. Meeting the process oxygen requirements;
2. Maintaining the suspended solids (biomass) dispersed through the depth of the liquid medium.

Under no circumstances will the effluent from complete mix aerated ponds be suitable for direct discharge due to the high level of suspended solids. For this reason, ***further pond treatment is a requirement*** to allow settling and stabilisation of solids. Although settling can be achieved in the front end of a maturation pond, **a dedicated settling pond is the strong preference.** Detention times in settling ponds are in the order of 2 days. The settling pond should be compared with a clarifier in a normal activated sludge plant. It does not contribute to the BOD removal from the system but retains a high percentage of the transferred solids.

A key difference between conventional facultative ponds and aerated ponds is that aerated ponds are very poor at pathogen removal, with removal levels comparable with that of conventional activated sludge plants. If disinfection in the ponds is a requirement, more maturation ponds will be required than for a conventional facultative pond system. Any gain in footprint in the aerated pond will therefore be lost in the maturation pond area.

Settling ponds require regular desludging, and provision must be made to do this with a maximum interval of 5 years. Where maturation ponds are used for settling, the front end of those ponds will also require desludging at similar intervals. Pond access for desludging and sludge drying beds or laydown area for geobags is required.

Function

Complete-mix aerated ponds are similar to aerated facultative ponds in that they are good at BOD conversion and achieving high levels of nitrification in the ponds (subject to having sufficient retention time).

Because aerated ponds are effectively activated sludge plants without a recycle (return sludge) system, the level of suspended solids will only reach a certain value, dictated by the influent substrate or BOD loading. The concentration of suspended solids in the pond is therefore quite low – and could range from as low as 100mg/L to possibly 350mg/L. This low level of suspended solids (say 100 to 300mg/L) also reflects that aerated ponds are very low efficiency systems when compared with conventional activated sludge plants.

Guidance: Physical features of complete mix aerated ponds

Feature	Comment
Depth	• 3m preferred, but 2.5 to 4.0m is acceptable
Inlet	• To be positioned a safe distance from the nearest aerator, and with due consideration for maintenance access to the aerators.
Outlet	• Should be at the furthest point from the inlet when considering the flow path. • Outlet to be fitted with an underflow baffle to prevent transfer of scum. • The underflow baffle may be as little as 100mm deep, but an adjustable baffle is recommended.
Embankments	• Normally 1:3 internally and 1:3 to 1:4 externally to aid maintenance. • Freeboard ≥ 500mm • Must be protected from wave action, particularly energy from the aeration system. Rip-rap works well to dissipate energy. • Freeboard must consider the pond maximum dimension, wind-fetch and embankment run-up that may also be caused by the aerators. • Must be designed to prevent growth of vegetation at the tops of embankments. This aids maintenance and prevents mosquito breeding.
Pond bottom	• Preferably level but may be slightly graded. Hollow pockets are not acceptable. • Concrete pads are required below vertical shaft aerators to prevent scouring of clay liners or damage to plastic liners.
Pond shape	• May vary with topography but are normally rectangular.

Effluent from a completely mixed aerated facultative pond normally flows either to:

1. *a settling pond (strongly preferred), or*
2. *a maturation pond which incorporates settling.*

2.11 Understanding effluent quality from ponds

Aerated ponds

The BOD_5 in the ***effluent*** stream is made up of two components, - a carbonaceous oxygen demand and a nitrogenous oxygen demand. This is due to the likely presence of nitrifying bacteria in the effluent stream from aerated ponds. In the influent stream it is reasonable to assume that no nitrifiers are present because conditions for their growth do not exist. In the effluent, the BOD_5 will therefore be inflated by nitrification that occurs in the BOD_5 test itself. To obtain a correct carbonaceous BOD (CBOD), the test must be done with a nitrification suppressant.

Both aerated and unaerated ponds

The effluent $CBOD_5$ measured will be largely the result of algae which in turn makes up 60 to 90% of the effluent TSS. On average, CBOD = 0.5TSS, and therefore determination of effluent TSS must be carried out on filtered samples to determine the soluble carbonaceous effluent BOD (SCBOD) which is a true and fair measure of the pond effluent quality.

2.12 Managing Odour

Anaerobic ponds can be odorous, but this need not be the case if they are designed correctly, and steps are taken in design to ensure that odours are not generated.

Hydrogen Sulphide in the form of H_2S is the gas primarily responsible for odour generation and is caused when H_2S gas escapes from the water body (normally in the anaerobic zone, but also from other ponds if they are not desludged in a timely manner). Hydrogen Sulphide is formed by the anaerobic reduction of sulphate and is in the form of either:

- Dissolved H_2S gas in solution, or
- The bisulphide iron HS^- with S^{2-} formed at high pH.

Odour will not be a problem if:

- The recommended design loadings are not exceeded.
- The pH in the upper layers of the anaerobic pond is maintained at an elevated pH.
- The sulphide concentration in the raw wastewater remains below 500mg SO_4/L, but for the purposes of design, a limit of 300mg/L is recommended.

It should be noted that small quantities of sulphur are good (> 3mg/L) because they:

- help precipitate heavy metals;
- are lethal to Vibrio Cholerae.

Figure 2-10 shows how the distribution of H_2S, and HS^- changes with pH. In aqueous solutions hydrogen sulphide is present as either dissolved hydrogen sulphide gas (H_2S) or the bisulphide ion (HS^-). At the pH values normally found in anaerobic ponds (around 7.5), most of the sulphide is present as the odourless bisulphide ion. Odour is only caused by escaping hydrogen sulphide molecules as they seek to achieve a partial pressure in the air above the pond which is in equilibrium with their concentration in it (Henry's law). Thus, for any given total sulphide concentration, the greater the proportion of sulphide present as HS^-, the lower the release of H_2S. Odour is not a problem if the recommended design loadings are not exceeded (See Table 4-1 Table 4-1).

In catchments with a history of high sulphide concentrations, the designer should include checking influent sulphide levels when determining influent characterisation.

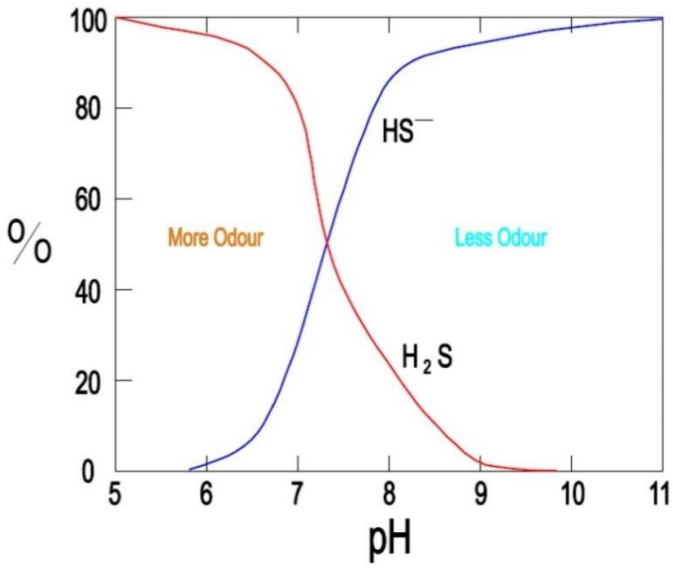


Figure ***Error! No text of specified style in document.****-6: Distribution of H_2S, and HS^- with changes in pH*

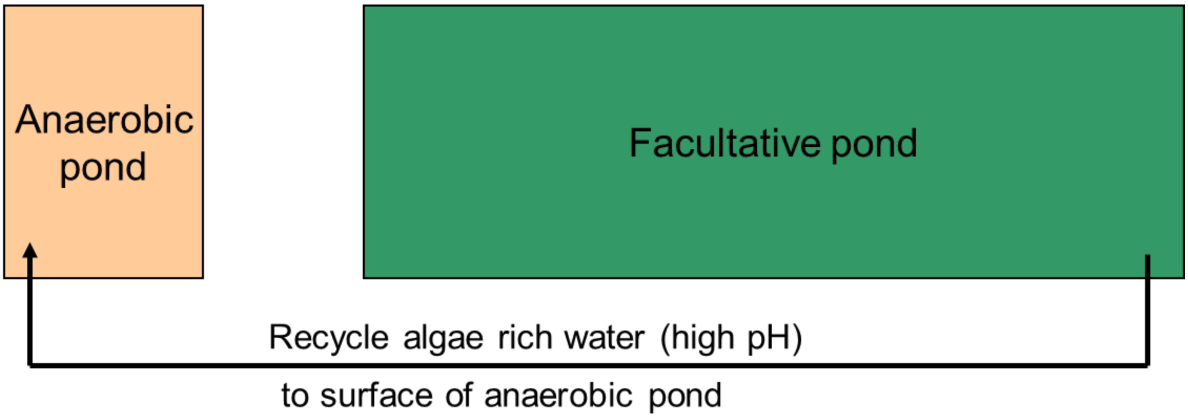


*Figure **Error! No text of specified style in document.**-7: Recycling of algae rich water over the anaerobic zone to suppress odour release*

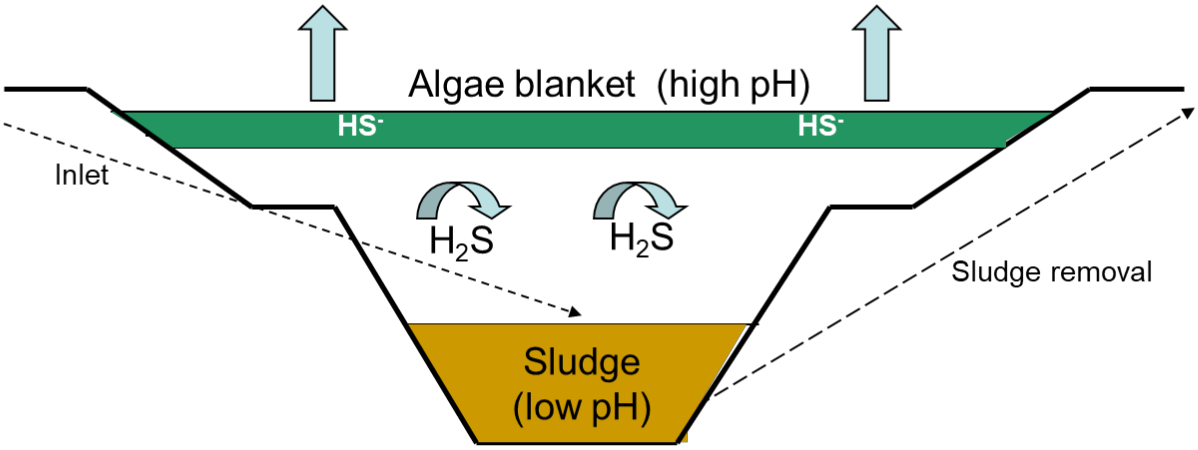


*Figure **Error! No text of specified style in document.**-8: Illustration of cross section through an anaerobic pond to control odour release.*

3 BASIS OF DESIGN (Input Parameters)

3.1 Wastewater characteristics and flow

Wastewater characteristics should preferably be determined from actual samples, but in the absence of such information, the following guidance is provided.

It is important to note that ponds are generally designed based on BOD_5 loading. As BOD_5 measurements can be misleading due to testing delays on samples coming from regional WA, samples collected for design should also test for COD and the COD/BOD ratio checked for alignment with typical values. Plant design parameters (design flow, BOD and/or COD) should be shown on the overall site General Arrangement Plan.

3.2 Biochemical Oxygen Demand (BOD_5)

Typical range is 30 to 75g BOD per capita per day. Generally, use 65g BOD per capita per day for Australian conditions.

Where BOD information is not available, or where regional test results indicate a lower BOD, a minimum BOD of 250mg/L is recommended for design. Wastewater sourced from STED schemes will be an exception to this rule, and proper sampling is required.

3.3 Temperature

Water temperatures in the facultative and maturation ponds are significantly affected by the prevailing air temperatures. The advice given here is indicative and generally applies to WA conditions. It is always best to use site data, but in the absence of this information, minimum and maximum water temperatures can be estimated from:

- Min temperature: Mean air temperature of coolest month (plus 2 – 3°C)
- Max temperature: Mean air temperature of warmest month (minus 2 – 3°C)
- Temperatures for other months should be interpolated.
- **Figure 3-9**: Estimating Water Temperature

Mean monthly air temperatures can be obtained from the Bureau of Meteorology (BoM) website at: **http://www.bom.gov.au/climate/averages/tables/ca_wa_names.shtml**

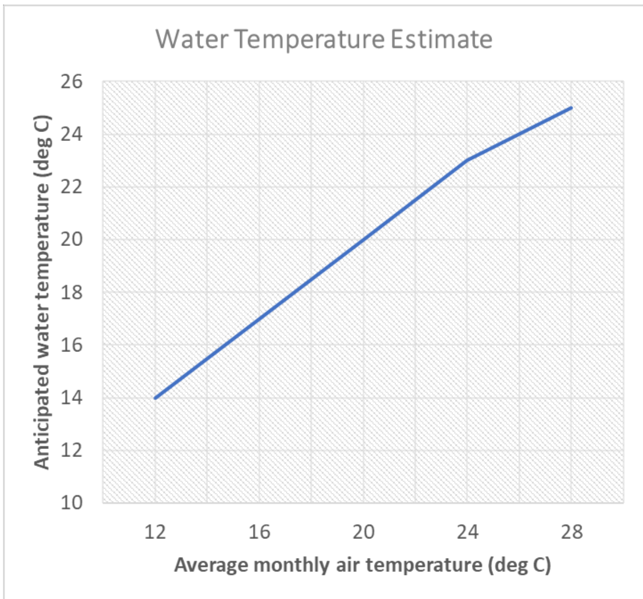


Figure 3-9: Estimating Water Temperature

Comment:

Water temperature in the anaerobic pond tends to be a bit warmer than the downstream ponds in winter due to the effect of the warmer influent stream. For the purposes of design this is generally not considered except in very cold climates where the influence of the warmer influent stream could have a marked effect on the activity of the anaerobic pond.

3.4 Flow

Normal *residential domestic flows* vary from about 170L/ capita per day to 250L/ capita per day.

In situations where some form of communal accommodation is the major source of wastewater, flows are much higher. This includes hostels, mining camps, prisons, etc., where normal flows range from 250L/ capita per day to 350L/ capita per day. Local knowledge or information from a similar near-by facility will assist in finding the correct values. An example of very high flows has been from the Federal Government Detention centre on Christmas Island which experiences flows of 400L/capita/day.

Flow data to determine the average dry weather flow (ADWF) is best obtained from actual records where they are available.

3.5 Flow measurement

Influent

Where the inflow to the plant is from a pumping station (i.e., a pressure main), a magnetic flowmeter should be used for influent flow measurement. The reference (example) drawings for the installation are in set BN85-056-001.

For a gravity fed plant a standard magnetic flowmeter may be used if:

- the device is in a location where the pipe always runs full, and velocities are adequate;
- the pipeline is not lowered locally to accommodate a full pipe, as this will result in grit and debris accumulation leading to inaccurate readings.

A flat bottom flume is the preferred flow measuring device for gravity systems.

Effluent

Effluent flow measurement can be achieved either by a flume, or a V-notch weir.

Device	Drawing reference	Australian Standard	Comment
Flume	BN85-004-001	AS3778.1	Not useful if zero flow at night.
V-notch	BN85-003-001	AS3778.1	Very good at low flows

Table 3-1: Flow measuring devices

3.6 Peak Flows

Process design is based on average dry weather flows (ADWF). ***Hydraulic design*** takes account of daily diurnal flow variations as well as peak wet weather flows (PWWF). This information is best obtained from historical data, but in the absence of diurnal flow data, the diurnal flow pattern may be generated from **Figure 3-**, the Y-axis being a factor of the ADWF. This graph shows how the peak dry weather flow (PDWF) peaking factor increases for smaller plants, whist attenuation in the collection system reduces the PDWF in larger plants. It is not uncommon for flows in small plants to drop to zero at night.

Peak wet weather flows (PWWF) tend to be in the form of sustained peaks which could last over several days in extreme conditions. Actual plant data should preferably be used but in general PWWF factors of 2.2 to 2.5 are applicable in gravity systems in the southern half of WA. A few southern collector systems have high infiltration rates that can push the PWWF factor to over 3.5. In regions subject to cyclonic conditions, PWWF factors could be as high as 5.0 in gravity systems.

In pumped systems, the peak flow reaching the plant is the combined flow that the various pumping stations in the collection system can deliver if they are all pumping together. This does not necessarily occur during wet weather events, and this peak (although short in duration) could also occur in an area that isn't fully developed. During wet weather events, the sustained high flow could be equal to the maximum combined flow that all the pumping stations in the system can deliver.

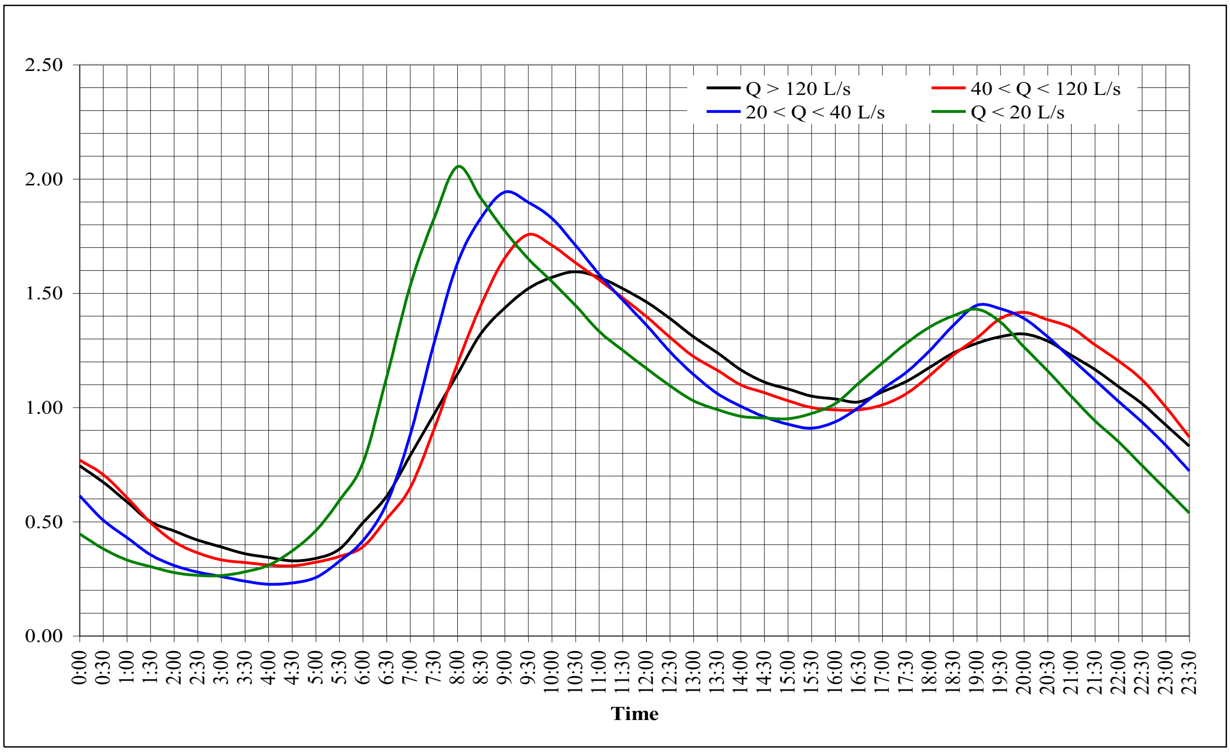


Figure 3-2: Generic hydrographs indicating diurnal ADWF patterns for WWTPs at different flows.

3.7 Net Rainfall and Evaporation

Net evaporation from the facultative and maturation ponds must be considered. The evaporation minus rainfall for each month must be considered, as this may have an impact on the sizing of the ponds. Evaporation from the anaerobic pond is often ignored, as crusting significantly limits evaporation. Rainfall on the anaerobic pond does, however, contribute to the total flow, and should be included.

Net Evaporation (e) = Evaporation minus rainfall

Mean monthly rainfall and evaporation figures can be obtained from the BoM website at: **http://www.bom.gov.au/climate/averages/tables/ca_wa_names.shtml**. Where rainfall and/or evaporation data are not available, interpolations need to be made from adjacent weather stations and/or isohyets.

Evaporation rates obtained from the BoM website must be adjusted to account by the Lake:Pan evaporation ratio applicable. Usually a figure of 0.75 is a good average. More information can be obtained from Evaporation data for Western Australia (dpird.wa.gov.au).

3.8 Nutrients

For domestic wastewater the following are a good guide:

Phosphorous:	12mg/L (likely to range from 11 to 16mg/L)
Ammonia:	45mg/L (likely to range from 40 to 75mg/L)
Total Nitrogen:	60mg/L (likely to range from 55 to 110mg/L)

3.9 Pathogens

Designed based on the Faecal Coli Count (CFU) and is normally taken as 10^8 CFU / 100mL.

In situations with high per capita flows (such as those mentioned above), a slightly lower value of 10^7 CFU / 100mL is justified.

3.10 Helminth eggs

Highly dependent on the source of the wastewater. If local knowledge is not available, and the presence of Helminth eggs is suspected, there could be as many as 60 eggs/litre of water. Generally, in Western Australia, Helminth egg counts are found to be below 40 eggs/litre, and only need to be considered in design north of the 20th parallel.

3.11 Sulphides

High sulphide levels will cause odour problems. Ensure that H_2S levels in the influent are below 300mg/L to be sure that odour does not become a problem.

4 WASTE STABILISATION POND PROCESS DESIGN

4.1 Guide

Waste stabilisation pond process design equations are given in this section with anecdotal comment to provide context for the mathematics. Design equations are generally based on the 'Mara' method.

It is important to note that ponds are generally designed based on BOD_5 loading. As BOD measurements can be misleading due to testing delays in regional WA, samples collected for design should also be tested for COD and the COD/BOD ratio checked for alignment with typical values.

The minimum number of ponds in series shall be two, with three generally provided depending on the method of disposal/reuse. Pond design should also assess the loading and treatment capacity on a seasonal basis (e.g., peak holiday periods, weather, working/mining towns), allowing for changes in influent strength and volume as a well as process temperature. The coldest temperatures will normally be the controlling design temperature.

4.2 Anaerobic Ponds

4.2.1 Volumetric BOD loading

Anaerobic ponds are designed on the basis *of Flow, BOD and Water Temperature (T)*

The **volumetric** BOD loading (λ_v, g/m³/d) is given by:

$$\lambda_v = \frac{L_i Q}{V_a} \tag{4.1}$$

where L_i = Influent BOD (mg/L) (= g/m³),

Q = Flow (m³/d) (= kL/d)

V_a = Anaerobic pond volume (m³).

Notes:

- λ_v increases with temperature
- $100 < \lambda_v < 400$ g/m³/day;
- Keep $100 < \lambda_v < 350$ g/m³/d, the lower value in order to maintain anaerobic conditions and the upper value as a safety factor to avoid odour release;
- Check that $\lambda_v > 100$ g/m³/d at the anticipated commissioning flows;
- Ensure that influent $SO_4 < 300$ mg/L to minimize the risk of odour release

Water Temperature T (°C)	Volumetric Loading $\boldsymbol{\lambda_v}$ (g/m³/d)	BOD Removal (%)
≤ 10	100	40
10 - 20	20T - 100	2T + 20
20 - 25	10T + 100	2T + 20
> 25	350	70

Table 4-1: Permissible volumetric BOD rates and percentage BOD removals in anaerobic ponds as a function of temperature

With a value of λ_v selected, the anaerobic pond volume can be calculated from equation (4.1). The mean hydraulic retention time in the pond (θ_a, day) is determined from:

$$\theta_a = \frac{V_a}{Q} \tag{4.2}$$

Tests:

Carry out the following:

- Ensure that the minimum retention time (θ_a) is greater than 1 day
- If $\theta_a < 1$ day, increase V_a to make $\theta_a > 1$ day
- Confirm that the month with the coldest water temperature has been used
- Check that holiday resident numbers don't dictate pond sizing outside of the coldest month

4.2.2 BOD removal

The percentage BOD Removal is calculated from the last column of Table 4-1: Permissible volumetric BOD rates and percentage BOD removals in anaerobic ponds as a function of temperature

The effluent BOD L_e (mg/L) can then be calculated from:

$$L_e = \frac{(100 - \%)L_i}{100} \tag{4.3}$$

4.2.3 Suspended Solids

No clear method for estimating suspended solids (SS) exists for any pond in a WSP treatment train. Von Sperling does provide a method for estimating SS in UASB plants. This may be used to provide ***an indication only*** of SS in the effluent from anaerobic ponds.

$$SS = 102 \times \theta_a^{-0.24} \tag{4.4}$$

4.2.4 Nutrients

Nitrogen

Nitrogen is hydrolysed to ammonia in anaerobic ponds. The result is that anaerobic pond effluent generally has higher concentrations of ammonia than the influent. The pH of anaerobic ponds is about neutral. Both Soares et al. (1996) and Silva et al. (1995) showed that in anaerobic ponds with a retention time of about 1 day, and temperatures around 22 to 26°C, the ammonia increased by about 26%. The work of Silva et al (1995) further showed reductions in organic nitrogen of about 50%, and overall the TKN dropped by about 7%.

Phosphorus

Although an increase in orthophosphate is likely, and some phosphorus is expected to be taken up / retained for synthesis, for the purpose of design, it is best to assume that the Total Phosphorus (TP) entering the anaerobic pond is equal to the TP leaving the pond.

4.2.5 Pathogenic organism removal

Coliform removal

Mara (2002) developed a faecal coliform removal model for anaerobic ponds based on data derived from full scale WSPs in Northeast Brazil. Satisfactory faecal coliform removal was demonstrated *at temperatures of 25°C*, and hydraulic retention times of 0.8 to 6.8 days.

$$N_e = \frac{N_i}{1 + k_{B(T)} \emptyset^{(T-20)} \theta_a} \tag{4.5}$$

Where N_e = effluent faecal coliform count (CFU per 100mL)

N_i = influent faecal coliform count (CFU per 100mL)

$k_{B(T)}$ = first order rate constant for faecal coliform removal (day^{-1}) (= 2.0 at 20°C)

$\emptyset$ = temperature coefficient for faecal coliform removal = 1.07

θ_a = hydraulic retention time in anaerobic pond (days)

T = mean water temperature (°C)

This empirical equation (4.5) may be used to estimate faecal coliform removal for anaerobic ponds in warmer climates or where anaerobic pond temperatures are ≥ 20°C.

Alternative:

For most applications faecal coliform removal is calculated by the Marais method being (see more in Paragraph 4.5):

$$N_e = \frac{N_i}{1 + k_T \theta_a} \tag{4.6}$$

$$k_T = 2.6(1.19)^{T-20} \qquad \textit{Marais Equation}$$

Helminth egg removal

Note: The Water Corporation does not have approval from Department of Health to use equation 4.7 to determine the removal of helminth eggs. Always refer to the latest WA health guideline for minimum detention in ponds and storage dams.

Helminth eggs (intestinal nematode eggs) are removed by sedimentation, and sedimentation occurs largely in anaerobic and facultative ponds. Sedimentation continues through the maturation ponds if removal is not completed in the anaerobic and facultative ponds. The rate of removal is constant and is based on the hydraulic retention time in each pond in series. Ayers *et al* (1992) analysed data for Helminth egg removal in Kenya, Brazil, and India. Removal efficiency according to the lower confidence limit of 95% should be used for design is given by:

$$E_a = 100\left[1 - 0.41e^{(-0.49\theta_a + 0.0085\theta_a^2)}\right] \tag{4.7}$$

where E_a = removal efficiency of helminth eggs (%) in each successive pond

Note: This removal rate does not apply to aerated ponds, as these maintain the eggs in suspension

4.2.6 Sludge accumulation

Sludge accumulates in anaerobic ponds at a rate of between 0.03 and 0.05m^3/EP/year. The lower value applies to warmer climates.

Anaerobic ponds should be desludged before the sludge layer reaches one third of the pond volume.

A regular program of desludging is best, and the sludge should not be removed entirely as this would lead to a loss of biomass.

In warmer climates (assuming an accumulation rate of 0.03m^3/EP/year), the sludge build-up per month can be estimated from:

$$V_s = \frac{EP \times 0.03}{12} \tag{4.8}$$

where: V_s = Volume of sludge accumulated per month

EP = The Population Equivalent

The maximum accumulation period (n - years) for desludging can be estimated from:

$$n = \frac{0.33 \times V_a}{EP \times 0.03} \tag{4.9}$$

4.3 Facultative Ponds

4.3.1 Surface BOD loading

Facultative ponds are designed on the basis of *Flow, BOD and Temperature*

This is an **area** determination. Note: The ***surface area*** calculated is taken ***at the mid-water depth***.

The **surface** BOD loading (λ_S, kg/ha.d) is given by:

$$\lambda_s = \frac{10 L_i Q}{A_f} \tag{4.10}$$

where λ_s = Surface BOD loading (from equation 4.10) (kg/ha/day)

L_i = Influent BOD (g/m^3) to that pond

Q = Flow (m^3/d)

A_f = Facultative pond area, at mid water depth (m^2)

[The factor 10 in equation 4.10 arises from the units used: $L_i Q$ is the mass of BOD entering the pond, g/day; so, $10^{-3} L_i Q$ is in kg/day and the area in ha is $10^{-4} A_f$].

Surface loading is used for facultative ponds, and NOT volumetric loading, because the light needed for algal photosynthesis arrives from the sun at the pond's surface. Thus, algal oxygen production is a function of area, so the BOD loading (which is an oxygen demand) must also be a function of area.

The permissible design value of λ_s increases with temperature (T, °C) which is essentially a proxy for climate. Mara (1987) gives the following global design equation:

$$\lambda_s = 350(1.107 - 0.002T)^{T-25} \tag{4.11}$$

Note: The reference to water temperature (T) in equation (4.11) is 25°C.

With a value of λ_s selected, the pond area is calculated from equation (4.10) and its retention time (θ_f, days) from:

$$\theta_f = \frac{A_f D_f}{Q_m} = \text{facultative pond retention time} \tag{4.12}$$

where D_f = The pond working (i.e., liquid) depth, (m) (usually 1.5 m) and

Q_m = Mean flow (m^3/d) = the mean of the influent and effluent flows (Q_i and Q_e)

A_f = the facultative pond area at mid-water depth (m^2)

The mean outflow is the inflow less net evaporation and seepage. Thus, equation (4.12) becomes:

$$\theta_f = \frac{A_f D_f}{0.5(Q_i + Q_e)} \tag{4.13}$$

If seepage is negligible, Q_e is given by:

$$Q_e = Q_i - 0.001eA_f \tag{4.14}$$

where e = The net evaporation rate, mm/day.

Thus, equation (4.13) becomes:

$$\theta_f = \frac{2A_f D_f}{(2Q_i - 0.001eA_f)} \tag{4.15}$$

Tests

Carry out the following tests:

- Ensure that the retention time complies with the following:
  - θ_f > 5 days, for temperatures below 20° C
  - θ_f > 4 days, for temperatures above 20° C
- Ensure that 100 < λ_S < 350 kg/ha/day
- Ensure that λ_v > 100 kg/ha/d at the anticipated commissioning stage flows

Rearranging equation (4.15) can be written as:

$$A_f = \frac{2Q_i\theta_f}{(2D_f + 0.001e\theta_f)} \tag{4.15a}$$

4.3.2 Pond failure envelope

Equation (4.1) above incorporates a factor of safety. This means that the BOD loading limit calculated could be exceeded for short durations without the pond failing. Both facultative and maturation ponds will fail if the pond failure envelope is exceeded - given by:

$$\lambda_S = 60(1.099)^T \qquad (Failure\ envelope\ loading)$$

4.3.3 BOD removal

Facultative pond effluent BOD can be estimated from:

$$L_e = \frac{L_i}{1 + k_{1(\mathrm{T})}\theta_f} \tag{4.16}$$

where $k_{1(T)}$ = first-order rate constant for BOD removal, day^{-1} and is given by:

$$k_{1(T)} = k_{1(20)}\alpha^{(T-20)} \tag{4.17}$$

where α = the Arrhenius constant, whose value is usually between 1.01 - 1.09.

Values for α and $k_{1(20)}$

Typical values of α for waste stabilization ponds range between 1.05 - 1.09. In general, it is acceptable to use a value of 1.07, but 1.05 will give a conservative result.

Note that $k_{1(20)}$= 0.3 day-1 for primary facultative ponds and $k_{1(20)}$= 0.1 day^{-1} for secondary facultative ponds.

The term L_e refers to the unfiltered BOD which includes the BOD of the algae in the pond effluent. This 'algal BOD' accounts for approximately 70 to 90% of the total (i.e., unfiltered) BOD of the effluent. Thus, the relationship between filtered and unfiltered BOD is:

$$L_e(filtered) = F_{na}[L_e(unfiltered)] \tag{4.18}$$

where F_{na} = the non-algal fraction of the total BOD (around 0.1 to 0.3, depending on the efficiency of the filtration system installed).

4.3.4 Suspended Solids

CAUTIONARY NOTE:

Algal biomass concentrations in facultative ponds make up 60 to 90% of the suspended solids (SS) in the ponds. The concentration is dependent on several factors including the BOD loading (λ_s), temperature, the un-ionized ammonia (NH_3), the un-ionized H_2S concentration, pond depth and settled sludge build-up. ***As a result, it has not been possible to develop any reliable method for estimating the effluent SS from ponds.*** Despite this, clients and regulators often insist that this information be provided. Whilst effluent suspended solids would normally not be expected to exceed 150mg/L where there is a series of at least 3 ponds operated close to their design loading, this is not always true.

To put some level of science into estimating the effluent suspended solids, Equation (4.19) below offers a rough indication of what might be expected. The equation includes a "fudge factor" F which the designer should apply based on regional knowledge and / or plant specific factors.

Equation 4.19 is continuously under review and applies to secondary facultative and maturation ponds only:

$$SS_e = F\left(SS_i + (L_i - L_e)1.05^{(T-20)}\right) \tag{4.19}$$

where F = 0.6 to 0.9

Note that this equation is only likely to be useful for ponds that are operating close to the design loading given by equation 4.10. Under loaded ponds are likely to produce more algae.

4.3.5 Nutrient removal

Total Nitrogen

The equations (4.20a) and (4.20b) are for total nitrogen removal in facultative and maturation ponds:

1. Ponds with a hydraulic regime closer to plug flow (more common, and preferred):

$$C_e = C_i e^{\{-k(\theta+60.6(pH-6.6))\}} \tag{4.20a}$$

2. Ponds with a hydraulic regime closer to complete mix (less common – not preferred):

$$C_e = \frac{C_i}{1 + [\theta(0.000576T - 0.00028)e^{(1.08-0.042T)(pH-6.6)}]} \tag{4.20b}$$

where $k = 0.0064 \times 1.039^{(T-20)}$ (d^{-1})

C_e = effluent total nitrogen concentration (mgN/L)

C_i = influent total nitrogen concentration (mgN/L)

If the pH is unknown, it can be estimated from

$$pH = 7.3e^{(0.0005Alk)} \tag{4.21}$$

where Alk = Alkalinity in the influent sewage (mg $CaCO_3$/L)

Ammonia

Pano and Middlebrooks (1982) provided the following equations for ammonia removal in facultative and maturation ponds

For T < 20°C

$$C_e = \frac{C_i}{1 + \left[\left(\frac{A}{Q}\right)(0.0038 + 0.000134T)e^{(1.041+0.044T)(pH-6.6)}\right]} \tag{4.22}$$

For T ≥ 20°C

$$C_e = \frac{C_i}{1 + \left[5.035 \times 10^{-3}\left(\frac{A}{Q}\right)e^{(1.540 \times (pH-6.6))}\right]} \tag{4.23}$$

Phosphorus

The main mechanisms for phosphorus removal in WSPs are:

- Removal of *organic phosphorus* taken up by the algae and bacteria through synthesis, and exiting with the final effluent;
- Precipitation of *phosphate* under high pH conditions (pH > 9.0).

Organic phosphorus corresponds with about 1% of the algae mass in the effluent.

In shallow ponds such as maturation ponds, with low hydraulic loading rates and pH > 9.0, phosphorus removal is likely to be between 60 and 80%. In facultative ponds, phosphorus removal is unlikely to exceed 35%.

4.3.7 Pathogenic organism removal

Coliform removal

For facultative ponds, faecal coliform removal is calculated by the Marais method being:

$$N_e = \frac{N_i}{1 + k_T\theta_f} \qquad \text{see (4.6)}$$

$$k_T = 2.6(1.19)^{T-20}$$

Helminth egg removal

Note: The Water Corporation does not have approval from Department of Health to use equation (4.7) to determine the removal of helminth eggs. Always refer to the latest health guideline for minimum detention in ponds and storage dams.

Helminth eggs (intestinal nematode eggs) are removed by sedimentation, and sedimentation occurs largely in anaerobic and facultative ponds. Sedimentation continues through the maturation ponds if removal is not completed in the anaerobic and facultative ponds. The rate of removal is constant and is based on the hydraulic retention time in each pond in series. Ayers et al (1992) analysed data for Helminth egg removal in Kenya, Brazil, and India. Removal efficiency according to the lower confidence limit of 95% should be used for design and is given by:

$$E_f = 100\left[1 - 0.41e^{(-0.49\theta_f + 0.0085\theta_f^2)}\right] \qquad \text{see (4.7)}$$

where E_f = removal efficiency of helminth eggs (%) in the facultative pond

4.3.8 Sludge accumulation

Sludge accumulation in facultative ponds depends to some degree on the upstream process.

Sludge accumulates in facultative ponds at a rate of between 0.026 and 0.064m^3/EP/year. The lower value is more typical of secondary facultative ponds, whilst the higher rate is more typical of primary facultative ponds.

$$Sludge\ Depth\ (mm) = \frac{1000 \times rate \times EP}{Pond\ Area}$$

Facultative ponds should be desludged when the sludge layer reaches to 1m below the average top water level in the pond. This means that if the facultative pond is 1.5m deep (as recommended), it should be desludged before the sludge layer reaches 1/3 of its depth (or 500mm). If, however, the facultative pond is shallower (say 1.2m deep) the pond should be desludged when the sludge layer is only 200mm thick.

4.4 Integrated Anaerobic / Facultative ponds

Design of these ponds is identical to the design of anaerobic and facultative ponds in series as set out above.

Key points to be made are:

- The full volume of the anaerobic pond as calculated must fit ***below*** the facultative pond. I.e., if the facultative pond is 1.5m deep, then the volume of the anaerobic pond (pit) is taken to be that volume below 1.5m deep;

- A berm around the anaerobic pit about half the depth of the facultative pond is a useful inclusion in the design because the facultative part of the pond needs to be commissioned first to mitigate odours during commissioning;
- The anaerobic pit should be near the inlet end of the Integrated Anaerobic / Facultative Pond system, and if possible, should be positioned so that the prevailing winds blow across the pond towards the anaerobic pond end to ensure that algae rich water is carried over the anaerobic pot for odour mitigation.

4.5 Maturation ponds

Design approach:

1. The approach to sizing a series of maturation ponds is to determine the pond area (and retention times) requirements to achieve a particular CFU reduction. See 4.5.4.
2. Determination of BOD and nutrient removal is secondary to CFU removal.
3. If there is a requirement to achieve a particular BOD removal, the retention time required for both CFU and BOD removal should be determined, and the optimum applied.

Follow the following design approach

- With the desired effluent CFU known, determine the optimum number and retention time of maturation ponds from 4.5.4;
- Calculate the pond for pond area from 4.5.1 (equation 4.15) and check that the surface BOD loading is not exceeded from equation 4.10;
- Calculate the anticipated effluent BOD pond for pond from 4.5.2;
- If a particular effluent BOD is required, determine the required retention time from 4.5.2 (equation 4.16) using the surface loading rate determined in 4.5.1;
- Nutrient removal is determined pond for pond from 4.3.5 in the same manner as for facultative ponds.

4.5.1 Surface BOD loading

Maturation ponds are designed on the basis of **Flow, influent CFU**, desired effluent CFU and Temperature. *The aim is normally to optimise CFU removal*.

This is a retention time determination - translated into a surface area (for a particular pond depth).

As with facultative ponds the ***surface area*** calculated is the area ***at mid-water depth***.

BOD removal in maturation ponds is calculated in the same manner as for facultative ponds.

The influent BOD loading on maturation ponds is equal to the unfiltered effluent BOD from the upstream facultative pond (or the upstream maturation pond in the case of a series of maturation ponds).

From equation 4.10:

$$\lambda_s = \frac{10L_iQ}{A_f} \qquad see \quad (4.10)$$

Single maturation pond:

Where there is a single maturation pond, the surface loading rate is limited to 0.75 of the loading rate of the upstream facultative pond.

Applying to a single maturation pond e.g., 4.10 can be rewritten as:

$$0.75\lambda_{s(m1)} = \frac{10L_{e(fac)}Q}{A_{m1}} = \frac{10L_{e(fac)}D}{\theta_{m1}} \qquad \left(as\ \frac{Q}{A} = \frac{D}{\theta}\right)$$

and rearranging

$$\theta_{m1} = \frac{10L_{e(fac)}D_{m1}}{0.75\lambda_{s(fac)}} \tag{4.24}$$

where $L_{e(fac)}$ = the unfiltered BOD of the facultative pond effluent (mg/L) (= g/m^3)

D_{m1} = the depth of the first maturation pond (m)

$\lambda_{s(fac)}$ = the surface BOD loading of the facultative pond, kg/ha/day

Series of maturation ponds:

If there is more than one or ***n*** maturation ponds, the reduced surface loading rate restriction of 0.75 is no longer applied, and equation (4.10) is re-written and applied to each maturation pond in series:

$$\lambda_{s(mn)} = \frac{10L_{e(n-1)}Q}{A_{mn}}$$

and

$$\theta_{mn} = \frac{10L_{e(n-1)}D_{mn}}{\lambda_{s(fac)}} \tag{4.24a}$$

where θ_{mn} = the retention time in the **n**th maturation pond.

As with facultative ponds, and taking evaporation into account, for each maturation pond

$$A_{mn} = \frac{2Q_i\theta_{mn}}{(2D_{mn} + 0.001e\theta_{mn})} \qquad \text{see (4.15a)}$$

where Q_i is the effluent flow from preceding pond, D_{mn} is the depth of the maturation pond, m and e is the net evaporation rate, mm/day.

4.5.2 BOD removal

Maturation ponds are not normally designed for BOD removal, yet it is often necessary to estimate the BOD of the final effluent. BOD removal in maturation ponds is very much slower than in anaerobic and primary facultative ponds.

***Note** the difference in the first order rate constant vs primary and secondary facultative ponds though.*

The BOD of the maturation pond effluent can be estimated from:

$$L_{e(mn)} = \frac{L_{i(mn)}}{1 + k_{1(\mathrm{T})}\theta_{mn}} \qquad \text{see (4.16)}$$

where $L_{i(m)} = L_e$ from the facultative or previous or n^th^ maturation pond

$k_{1(T)}$ = first-order rate constant for BOD removal, day^{-1} and is given by:

$$k_{1(T)} = k_{1(20)}\alpha^{(T-20)} \quad \text{see (4.17)}$$

where α = the Arrhenius constant, whose value is usually between 1.01 - 1.09.

Values for α and $\boldsymbol{k_{1(20)}}$

- Typical values of α for waste stabilization ponds range between 1.05 - 1.09. In general, it is acceptable to use a value of 1.07, but 1.05 will give a conservative result.

Note that $k_{1(20)}$ = 0.05 day^{-1} for maturation ponds

The term L_e refers to the unfiltered BOD which includes the BOD of the algae in the pond effluent. This 'algal BOD' accounts for approximately 70 to 90% of the total (i.e., unfiltered) BOD of the effluent. Thus, the relationship between filtered and unfiltered BOD is:

$$L_e(filtered) = F_{na}[L_e(unfiltered)] \quad \text{see (4.18)}$$

where F_{na} is the non-algal fraction of the total BOD (around 0.1 to 0.3, with a usual design value of 0.3).

4.5.3 Nutrient removal

Estimating nutrient removal in maturation ponds is carried out pond for pond in the exact same manner as for facultative ponds. The approach is not repeated here.

See paragraph 4.3.5 above and follow the same procedure pond for pond.

4.5.4 Pathogenic organism removal

Coliform removal

Notes:

1. Note the differences in calculation of CFU decay for a single (eq 4.6) vs a series (eq 4.25) of maturation ponds (see below);
2. Usually, a series of maturation ponds is selected to achieve a low CFU level in the effluent. If this is not a requirement, then a single pond is usually adequate;
3. Improved CFU removal is achieved by baffling maturation ponds (see paragraph 5.5.3).

The effluent CFU for a ***single maturation pond*** is given by:

$$N_e = \frac{N_i}{1 + k_T\theta_m} \quad \text{see (4.6)}$$

where N_e = number of CFU in effluent (counts per 100mL)

N_i = number of CFU in influent (counts per 100mL)

k_T = first order rate constant for CFU removal (days) at temperature T

θ_m = maturation pond retention time (days)

$$k_T = 2.6(1.19)^{T-20} \quad \textit{Marais equation}$$

The Marais equation is a measure of the rate of pathogen decay due to the action of natural UV radiation from sunlight. The equation (originally developed in 1961) has been tested in various parts of the world and found to be consistently reliable.

The effluent CFU for a series of ponds including anaerobic, facultative and a series of n equal maturation ponds is given by:

$$N_e = \frac{N_i}{(1 + k_T\theta_a)(1 + k_T\theta_f)(1 + k_T\theta_m)^n} \tag{4.25}$$

Notes:

- Subscripts a, f and m refer to anaerobic, facultative and maturation ponds respectively;
- There could be n (number) of maturation ponds;
- Equation 4.25 assumes that maturation ponds are of equal size;
- If maturation ponds are not of equal size, the denominator of equation 4.25 becomes:
- $(1 + k_T\theta_a)(1 + k_T\theta_f)(1 + k_T\theta_{m1})(1 + k_T\theta_{m2}) \dots (1 + k_T\theta_{mn})$

Solution of these equations is as follows:

- Examining equation 4.25 reveals 2 unknowns:
    - θ_m and n
- Solve as follows:
    - θ_a and θ_f have been calculated and N_i and N_e should be known
    - Calculate k_T from the *Marais equation*
    - Calculate θ_m for n = 1,2,3 etc (usually not more than 3 M-ponds in series)
- Adopt the following rules to select most appropriate combination of θ_m and n
    1. $\theta_m \le \theta_f$
    2. $\theta_m \ge 3$ days if T ≥ 20°C
    3. $\theta_m \ge 4$ days if T < 20°C for more than half of the year
    4. $\lambda_{m1} \le 0.75\lambda_f$ if there is only 1 maturation pond

Helminth egg removal

> **Note:**
> The Water Corporation does not have approval from WA Department of Health to use equation 4.7 to determine the removal of helminth eggs. Always refer to the latest health guideline for minimum detention in ponds and storage dams.

The design equation for percentage egg removal is given by:

$$E_m = 100[1 - 0.41\exp(-0.49\theta_m + 0.0085\theta_m^2)] \quad \text{see (4.7)}$$

where E_m = the percentage (%) egg removal in respective maturation ponds

θ_m = the retention time in the respective maturation ponds, (days)

4.6 Aerated ponds

4.6.1 Loading rates

Partial mix aerated ponds provide only sufficient aeration to substitute for the oxygen that would have been provided by natural (algae (photosynthetic) and wind) causes in conventional facultative ponds.

Complete mix aerated ponds supply sufficient aeration to completely degrade all BOD, ***PLUS*** the additional aeration required to ensure that all suspended solids in the pond remain in suspension.

From previously

$$L_e = \frac{L_i}{1 + k_1\theta_b} \qquad (see\ \ 4.13)$$

As with facultative ponds, aerated pond effluent consists of both soluble BOD and particulate BOD, but the effluent particulate BOD is no longer mostly associated algae. Therefore:

$$S_e = L_e - BOD_{part}$$

where S_e = soluble BOD_5 in the effluent (mg/L)

L_e = total BOD_5 in the effluent (mg/L)

BOD_{part} = particulate (suspended) BOD_5 in the effluent (mg/L)

For design purposes, in aerated ponds, we therefore apply first order kinetics only to the soluble portion of the influent BOD, so equation 4.13 becomes:

$$S_e = \frac{L_i}{1 + \kappa_{1(T)}\theta_b} \qquad (4.26)$$

All the influent BOD is therefore assumed to be soluble (i.e., $S_i = L_i$).

where S_e and L_i are the effluent and influent BOD concentrations respectively

$\kappa_{1(T)}$ = first-order rate constant for BOD removal day^{-1} and is given by:

$$\kappa_{1(T)} = \kappa_{1(20)}(\alpha)^{T-20}\ (per\ day\ @\ 20°C) \qquad (4.27)$$

$\kappa_{1(20)}$ = 2.5 (/day @ 20°C)

θ_b = hydraulic retention time: - normally 2 < θ_b < 6 days. (> 4 days preferred)

α = temperature coefficient for BOD removal

= 1.056 for α > 20°C

= 1.035 for 4°C < α < 20°C

Note: The retention time in an aerated lagoon is 2 to 6 days with 4 days being typical (Mara). Von Sperling (and others) prefer a minimum of 5 days for partly aerated ponds. For the purpose of this design manual and because of our generally warmer climate in WA, a minimum of 4 days should be adopted for partly aerated, and 2 days for the first in a series of aerated ponds.

For aerated ponds in series:

$$\frac{L_e}{L_i} = \left(\frac{1}{1+\kappa_1\theta_{b1}}\right)\left(\frac{1}{1+\kappa_2\theta_{b2}}\right)\cdots\left(\frac{1}{1+\kappa_n\theta_{bn}}\right)$$

where $\kappa_1, \kappa_2 \ldots \kappa_n$ are the reaction rates in cells 1, 2,…n and normally taken to be equal

$\theta_{b1},\ \theta_{b2},\ and\ \theta_{bn}$ are the hydraulic retention times in the respective ponds

4.6.2 Suspended solids and BOD

The suspended solids (cell concentration) in aerated ponds can be estimated from:

$$X = \frac{Y(L_i - S_e)}{1 + b_T\theta_b} \tag{4.28}$$

where X = cell concentration in the pond (mg/L)

Y = 0.67 = organism yield coefficient (mg SS/mgBOD)

b_{20} = 0.07 = autolysis rate (/day @20°C)

$b_T = 1.029^{T-20}$

Conversion of BOD to the quantity of cells (X) is given by:

$$C_5H_7NO_2 + 5O_2 \xrightarrow{yields} 5CO_2 + 2H_2O + NH_3$$

Thus, 1g of cells has an ultimate $BOD = \frac{5\times32}{113} = 1.42$g.

Since $\frac{BOD_5}{BOD_u} \approx \frac{2}{3}$, 1g of cells has a BOD of 0.947g.

The total effluent BOD is therefore given by:

$$L_e = S_e + 0.947X \tag{4.29}$$

In this case, L_e is a good indication of the effluent BOD in a *complete mix aerated pond*.

The value of X can vary downwards depending on the level of aeration in the pond.

Near the outlet, X depends on the level of aeration in the vicinity of the outlet.

4.6.3 Nitrification

Nitrification may occur in aerated ponds in warm climates subject to:

- Sufficient aeration being provided (4.57mg O_2 per mg of ammonia-N nitrified) [Brink *et al* (2007) showed that this can be reduced to 4.42 as about 1% of the FSA is used for synthesis]
- Sufficient alkalinity available (7.14 mg of alkalinity as $CaCO_3$ per mg of ammonia-N nitrified) and minimum alkalinity in the pond of 60mg/L
- Minimum pH of 7.0
- Adequate retention time (minimum sludge age) for nitrification

- Minimum sludge age: θ > 3 days at temperatures above 20°C
- Good (roughly equal) aeration distribution throughout the pond

The kinetic equation is:

$$\theta_s = \frac{S_f P_f}{\mu_T - b_{hT}} \tag{4.30}$$

where $\theta_s = sludge\ age = \theta_b = \frac{V_b}{Q}$ = hydraulic retention time due to no recycle.

S_f = Design safety factor applied to nitrification (recommended S_f > 1.3)

[Note: Setting S_f =1.0 will give the minimum sludge age for nitrification]

P_f = the ratio between daily peak and average ammonia mass in the influent

≈ daily peak dry weather flow **:** average dry weather flow ratio

≈ 1.5 to 2.0 for most WWTPs in Western Australia (see 3.6)

b_{hT} = endogenous decay rate at temperature T. b_{hT} = 0.24 @ 20°C

$b_{hT} = b_{h20} 1.029^{T-20}$

$$\mu_T = 0.8(1.086)^{T-20}\left(\frac{C_e}{K_{s(N)T} + C_e}\right)\left(\frac{DO}{K_{s(DO)} + DO}\right) \tag{4.31}$$

where C_e = the required effluent ammonia concentration

DO = the “in lagoon” dissolved oxygen concentration (normally 1 to 2 mg/L)

$K_{s(N)T}$ = Monod half saturation constant for NH_3. $K_{s(N)T}$ = 0.733 @ 20°C

$K_{s(N)T} = K_{s(N)20} 1.1246^{T-20}$

$K_{s(DO)}$ = Monod half saturation constant for DO (typically 0.5mg/L)

Notes:

1. **In aiming to achieve nitrification when sizing an aerated pond, the hydraulic retention time for BOD removal and for nitrification must both be calculated, and the higher of the 2 values used for determining the pond volume.**
2. **In ponds where $S_f < 1.0$, nitrification will not occur, and the aeration system is only sized for Oxygen required for BOD removal**

4.6.4 Phosphorus removal

Phosphorus removal is only 15 to 25%, and mostly associated with synthesis.

4.6.5 Aeration requirements

Aeration requirement for ponds can be divided into three components, being:

1. Oxygen required for BOD removal (minimum requirement for aerated ponds)
2. Oxygen required for nitrification (additional to BOD removal oxygen requirement)
3. Aeration required for mixing the pond if a complete mix aeration system is required (represented by the energy required to keep pond solids in suspension)

Oxygen required for BOD removal

The oxygen required for biological oxidation (R_{O2BOD}) is based on the ultimate BOD removed

Given that $BOD_U \approx \frac{3}{2} BOD_5$

$$R_{O2BOD} = \frac{1.5\,(L_i - L_e)Q \times 10^{-3}}{24} \tag{4.32}$$

From equation (4.29), equation (4.32) can also be written as:

$$R_{O2BOD} = \frac{(1.5(L_i - \mathrm{S}_e)\mathrm{Q} - 1.42XQ) \times 10^{-3}}{24} \tag{4.33}$$

where R_{O2BOD} = the oxygen required for biological oxidation (kg/h)

Oxygen required for nitrification

Applies if $S_f \geq 1.0$. If $S_f < 1.0$ then $R_{O2NH3} = 0$

The oxygen required for nitrification is given by:

$$R_{O2NH3} = \frac{4.42(C_i - C_e)Q \times 10^{-3}}{24} \tag{4.34}$$

where R_{O2NH3} = the oxygen requirement for nitrification (kg/h)

C_i *and* C_e = influent and effluent ammonia–N concentrations respectively (mg/L)

Note:

The theoretical oxygen requirement for nitrification is 4.57kgO$_2$/kgFSA. Mara (2003) has argued that this should be 3.1, but as previously indicated, Brink et al (2007) have demonstrated experimentally a figure of 4.42 with the difference between the 4.42 and 4.57 being attributed to assimilation.

Aeration required for complete mixing

Where complete mixing is required, the power input will depend on the type of aeration system in the pond and the mixing energy (W/m^3) that the particular aeration system is able to supply.

The Power input is defined as:

$$P_V = \frac{P}{V} \tag{4.35}$$

where P_V = required power input (W/m^3)

P = Power required for mixing aeration (W)

V = Volume of pond (m^3)

To ensure complete dispersion (mixing) of suspended solids:

$$P_V \geq 3.0\ W/m^3$$

Mara quotes Horan (1990) as providing the following equation to estimate the power requirements for complete mix systems:

$$P_V \geq 5 + 0004X \tag{4.36}$$

With X, as previously, being the suspended solids (cell concentration)(mg/L) in the pond.

If a complete mix system is required, the best approach is to work with vendors to ensure that a complete dispersion of suspended solids is achieved, always ensuring that $P_V \geq 3.0\ W/m^3$.

4.6.6 Aeration System Design

Several parameters affect the efficiency of aeration systems. These include the oxygen transfer rate, the effect of impurities, surfactants, solids concentration, altitude (atmospheric pressure), temperature, and system losses in the mechanical and electrical systems of the aerators. When specifying aeration systems, the relevant efficiency parameters should be defined by the Engineer, and the system losses provided by the vendor. The most basic of these parameters is the oxygen transfer rate (OTR) which defines the amount of oxygen the system can supply per unit of time.

$$OTR = k_{La}(DO - DO_{sat})V \tag{4.37}$$

where OTR = Oxygen Transfer Rate (kgO_2/h)

k_{La} = liquid side mass transfer coefficient (h^{-1})

DO = Dissolved Oxygen in water (kgO_2/m^3)

DO_{sat}= Dissolved Oxygen in water at saturation (kgO_2/m^3)

V = Volume of water (m^3)

For the purpose of this manual, the following terminology is adopted

Parameter	Definition	Remark
OTR	Oxygen transfer rate in clean water	$= k_{La}(DO - DO_{sat})V$
SOTR	Oxygen transfer rate in standard conditions in clean water	
AE	Aeration Efficiency in clean water	= OTR / P
SAE	Aeration Efficiency in standard conditions in clean water	$= \frac{k_{La} C_{s(std)} V}{P}$
α	Alpha factor: ratio of process to clean water mass transfer (the reduction in O_2 transfer rate caused by impurities)	$= \frac{k_{La\ (process\ water)}}{k_{La\ (clean\ water)}}$
β	Beta factor: reduction in transfer rate caused by salinity	
C_s	Saturation concentration of oxygen in clean water	Impacted by both temperature and pressure (altitude)
$C_{s(std)}$	Saturation concentration (std) of oxygen in clean water	= 9.07mg/L
Standard conditions are defined as 20°C, 1 atm (i.e., 760 mm Hg), zero salinity, zero DO in water		

Standard Aeration Efficiency (SAE) is to be provided by the vendor. Table 4-2: SAE values for different types of aerators

provides some guidance on aeration efficiencies of different types of aeration devices.

1. This information is to be used with caution, as an aid only.
2. Pond systems usually have a short sludge age (SRT), and this reduces aeration efficiency.
3. SAE applies to ponds of at least 3m deep. When aerators are installed in traditional facultative ponds (of about 1.5m deep) aeration efficiency is usually also lost.
4. It is the designer's responsibility to ensure that the correct aeration efficiency information is used in design.

Aerator Type	SAE (kgO_2/kWh)	AE for low SRT (@ $2mgO_2/L$)
High speed surface aeration	0.9 – 1.3	0.4 – 0.8
Low speed surface aeration	1.5 – 2.1	0.7 – 1.5
Submerged Jet aeration	0.9 – 1.4	0.4 – 0.6
Coarse Bubble aeration	0.6 – 1.5	0.3 – 0.7
Brush aeration	1.2 – 1.8	0.6 – 0.9

Table 4-2: SAE values for different types of aerators

To correct the aeration efficiency to field conditions, the following equation is used.

$$AE_f = \mathrm{SAE} \times \alpha \times (1.024)^{T-20} \times \left(\frac{\beta . P . C_s - C_o}{9.07}\right) \quad (4.38)$$

where:

AE_f = the aeration efficiency under field conditions, kgO_2/kWh;

SAE = the standard aeration efficiency in clean water at zero dissolved oxygen, at sea level and 20°C (refer to **Table 4-2**: SAE values for different types of aerators

above or (preferably) use values provided by aerator's manufacturer), kgO_2/kWh;

α = the ratio of oxygen transfer to pond water to the rate of oxygen transfer to clean water (can assume 0.85 to 0.9 for aerated ponds with surface aerators);

T = the temperature of the pond water, °C;

β = the ratio of dissolved oxygen saturation concentration in pond water to that in clean water at the same temperature and pressure (assume 0.95 for aerated ponds unless the pond is unusually saline then a lower value may be appropriate);

C_o = the dissolved oxygen concentration to be maintained in the aerated pond, mg/L;

P = the ratio (correction factor) of barometric pressure at the plant elevation to the barometric pressure at sea level in **Table 4.3**);

C_s is the saturation dissolved oxygen (solubility) concentration for clean water, mg/L see **Table 4-4**.

Elevation (m)	Correction Factor (P)
Sea Level	1.00
300	0.97
600	0.93
900	0.90
1200	0.86
1500	0.83
1800	0.80
2100	0.76
2400	0.73

Table 4-3: Correction factor (P) at various elevations

Temperature (°C)	C_s Solubility (mg/L)
5	12.67
10	11.23
15	10.07
16	9.86
17	9.65
18	9.46
19	9.27
20	9.07
21	8.91
22	8.74
23	8.57
24	8.42
25	8.26
26	8.12
27	7.97
28	7.84
29	7.70
30	7.57

Table 4-4: Solubility of O_2 in clean water at sea level

Design example:

Determine the oxygen transfer rate under field conditions for a slow speed surface aerator (SAE = 1.5kgO_2/kWh) with the following operating conditions:

- temperature of pond water is 20°C
- elevation at sea level
- dissolved oxygen concentration to be maintained in the aerated pond is 2 mg/L

Solution – applying equation (4.38):

$$AE_f = \mathrm{SAE} \times \alpha \times (1.024)^{T-20} \times \left(\frac{\beta.P.C_s - C_o}{9.17}\right)$$

$$= 1.5 \times 0.9 \times (1.024)^{20-20} \times \left(\frac{0.95 \times 1 \times 9.07 - 2}{9.07}\right)$$

$$= 1.01 \text{ kg } O_2/\text{kWh}$$

4.6.8 Pathogen removal

Helminth eggs

As the removal of Helminth eggs is based on them settling in ponds, they will remain in suspension in aerated ponds. It must therefore be assumed, for aerated ponds, that the number of eggs in the effluent is equal to the number of eggs in the influent.

Coliform removal

For coliform removal, normal facultative ponds rely on a combination of temperature, UV radiation through light penetration into the pond, pH, and dissolved oxygen for CFU reduction. In the case of aerated ponds, the amount of algae is significantly reduced, and the suspension of solids prevents the penetration of natural UV radiation to effect disinfection. Aerated ponds therefore have a lower CFU removal than conventional facultative ponds.

4.6.9 Sedimentation pond

Following an aerated pond, a sedimentation pond is required. This pond may take on two forms, being:

1. A dedicated sedimentation pond;
2. The front end of a maturation pond if maturation ponds form part of the process train.

It is preferred that sedimentation ponds be separate and upstream of maturation ponds (i.e., they will be after the aerated pond and before the maturation pond).

Where maturation ponds are used for sedimentation:

1. The first maturation pond must be increased in size to allow for the sedimentation zone;
2. Provision must be made for “on-line” desludging of that zone of the maturation pond on a regular basis.
3. The key risk is odour generation, and therefore, at least the front end of the maturation pond should have an increased depth for sludge storage, and 1m of liquid depth above the sludge layer.
4. Ponds should not be used for settling of Alum sludge. Alum should rather be dosed as a tertiary treatment step and filtered out.

The basic requirements for sedimentation ponds are:

Description	Measure / retention time
Sufficient hydraulic retention time to allow settlement of solids	6 hours minimum
Sufficient volume for sludge storage	Calculated as below
Algal growth must be minimized	2 days maximum (1 day preferred)
Odours must be controlled	1m of liquid depth above the sludge layer (D_L) (m)

The volume of the sedimentation pond is given by

$$V = Q\theta \text{ (m}^3\text{)}$$

and the area by:

$$A = \frac{Q\theta}{D_L} \ (m^2)$$

where D_L = liquid depth above the sludge layer

The annual mass of suspended solids added to the sedimentation pond is:

$$M = 365Q(SS_i - SS_e) \times 10^{-3} \quad (4.39)$$

where M = Mass of SS (kg/year)

SS_i = Influent SS (mg/L)

SS_e = Required effluent SS

$$SS_i = SS_{RW} + X \quad (4.40)$$

where SS_{RW} is the biodegradable SS concentration in the raw wastewater (mg/L)

The mass of volatile SS is assumed to be $M_{VSS} = 0.7M_{TSS}$

Further assuming that the mass of VSS is reduced by 75% over the period of 1 year, then over a period of n years:

$$(M_{VSS})_n = 0.25nM_{VSS} \quad (4.41)$$

And the total solids produced in n years is:

$$(M)_n = 0.3nM + 0.25nM_{VSS} \quad (4.42)$$

Assuming that the sludge can compact to a 15% solids concentration, and that the density of that sludge is 1,060kg/m^3, the depth of the sludge layer can be determined from:

$$D_S = \frac{(M)_n}{A(0.15 \times 1060)} \quad (4.43)$$

The total depth of the sedimentation pond is therefore:

$$D = D_s + D_L$$

5 POND HYDRAULICS

5.1 General

Understanding pond hydraulics requires consideration of the inputs and influences on the flow momentum within a pond. These broadly include:

- flowrate – higher flowrates increase inlet momentum;
- inlet size – smaller inlets increase the inlet velocity and so also the inlet momentum;
- inlet position and orientation – defines the way the inlet momentum is introduced into the main body of the pond and as a result influences the main flow pattern;
- outlet position – sets the distance from the inlet and, therefore the time, for the main flow to reach the outlet;
- pond geometry and baffles – strong influence on flow patterns and defines the degree of 'channelling';
- temperature/density effects – may influence the channelling and circulation of the main flow;
- wind shear – higher wind velocities and greater pond surface areas increase the water column momentum and as a result influence the main flow pattern;
- mechanical aerators – if present constitute significant momentum input and as a result have a strong influence on the main flow pattern.

5.2 Pond Inlets

5.2.1 Inlet depth

The influent to each pond should be discharged at mid depth. This reduces short circuiting across the surface of the pond or stirring up settled sludge.

Anaerobic ponds

Inlet pipe are to discharge downwards, and the discharge point should be in the geographical middle of the pond. This allows screenings and grit to be deposited, light material to more uniformly float to the surface to form the crust and biological gas filter.

Facultative and Maturation ponds

Although the "jet action" of the water released at the inlet is small and only has a localised effect, it does still have an influence on the overall flow pattern and momentum through the pond. Inlets should comply with the following:

- Inlet pipe to discharge horizontally at mid depth;
- Inlet pipe to be turned parallel to the embankment or wall;
- A stub baffle as indicated in Figure 5-1 and Figure 5-2 combined with inlets parallel or facing the wall aid in creating an S-shaped momentum pattern through the pond.

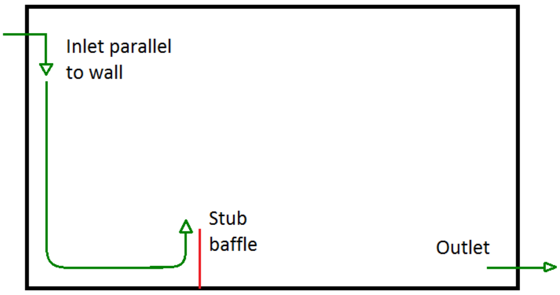


Figure 5-1: Pond with inlet parallel to wall and stub baffle

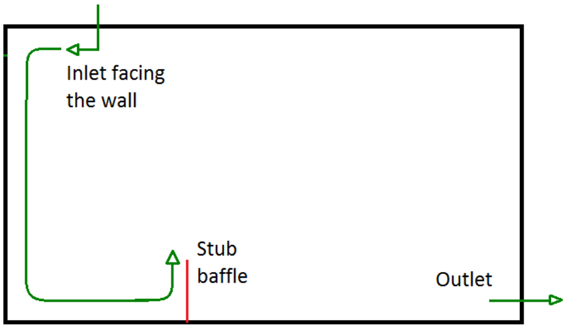


Figure 5-2: Pond with inlet facing the wall and stub baffle

5.3 Pond Outlets

5.3.1 Positioning

Work by Shilton *et al* (2003) demonstrated that water tends to circulate around the pond rather than simply moving from the inlet to the outlet. If, however, it swirls around past the outlet, then short circuiting will occur, and treatment efficiency compromised. The engineer must therefore consider the most likely flow pattern and position the outlet in a "sheltered" spot. This means that the inlet position, its orientation, the pond shape, and the pond flow pattern must be considered first, and an outlet position selected which will provide the best treatment efficiency without affecting the hydraulic flow pattern.

Outlet manifolds tend to expose the outlets over a wide area, trapping scum, causing short circuiting, reducing the effective pond treatment area, ultimately compromising effluent quality, and may not be used.

5.3.2 Outlet configuration

Outlets are to be designed to draw water from a zone below the water surface without drawing water down from the surface. This means that velocities must be low, and some form of underflow baffle or T-pipe are preferred. Depths for the underflow baffles or T-pipes are given below. Straight horizontal pipes, even if they are at the depths indicated below are ***NOT*** acceptable. Horizontal open-ended pipes do not draw water from the target zone, but instead draw water from the zone above the pipe and compromise quality.

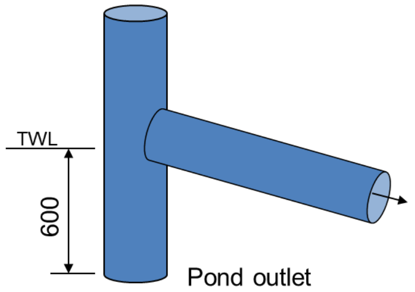
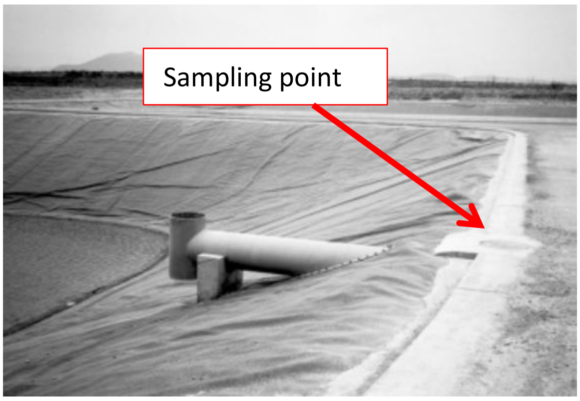


Figure 5-3: Schematic of a T-pipe outlet for a facultative pond.

Key points to consider:

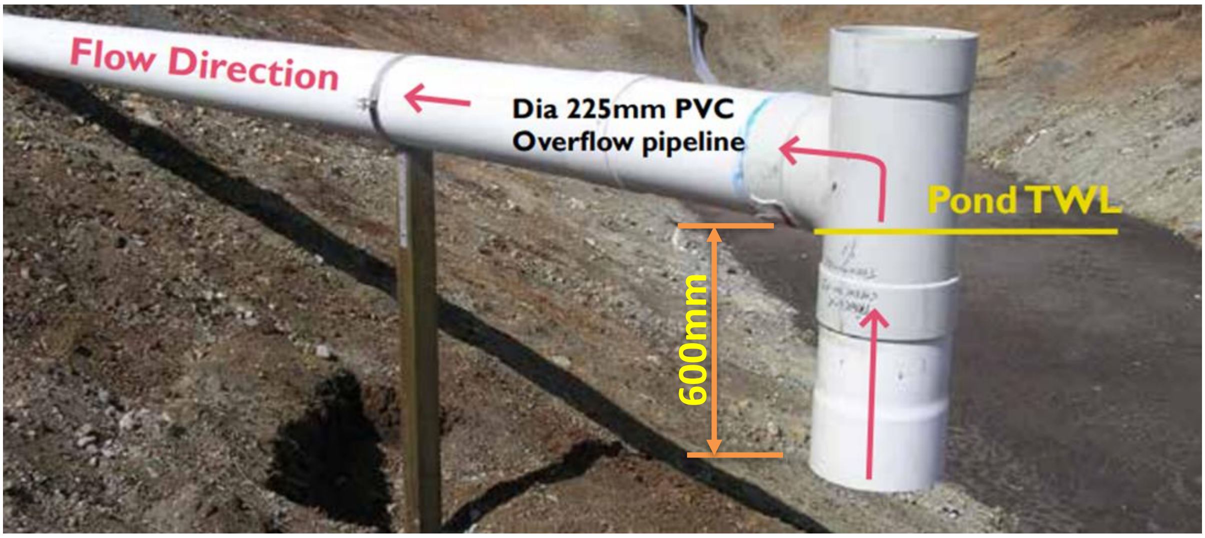
- Keep outlets simple;
- Allow for sampling points in the embankment;



Note: The image to the left shows a convenient location for a sampling point, although it should be 2m from the edge of the embankment. The T-piece at the intake is poorly designed though. It needs to be extended further into the pond, be 600mm deep at the intake, and the intake point should be at least 600mm above the floor / embankment to avoid drawing in settled sludge.

- Size to keep water levels relatively constant, and outlet velocities low (<1.0m/s);
- Bottom to be 600mm above the floor or pond embankment;
- Keep away from turbulent zones – refers particularly to aerated ponds;
- The “big idea” is to draw water from below the scum layer (anaerobic pond), and the algae layer (facultative pond).

An example of a pond outlet



- The image above shows a T-intake for a DN225 overflow pipe under construction. The intake point must also be at least 600mm above the embankment. The pipe support should be made of non-corrosive material and close enough to the T to prevent the pipe from flexing.
- Weir using concrete segment placed within pond, surrounded by underflow baffle - Plan JT14-003-004. (Broome North);
- Concrete outlet built into the embankment with underflow baffle - Plan GE88-003-005-01D (Coral Bay – good example for large system but expensive);
- HDPE duck foot bend pipe in association with an underflow baffle – Plan JT71-5-5 (Exmouth North).

5.3.3 Outlet depth

Use the following depths (to invert of a T-pipe or to the depth of an underflow baffle) as a guide:

- Anaerobic pond: 300 mm below the static water surface level to reduce sludge and the chance of solids from the crust being carried over;
- Facultative pond: 600 mm below the static water surface to minimize the carryover of algae and scum to the maturation pond or receiving water;
- Maturation pond: variable from 500mm in the first maturation pond to minimise algae carryover if there is a series of maturation ponds to 50 mm in the last maturation pond to obtain the highest level of UV disinfected and oxygenated final effluent.

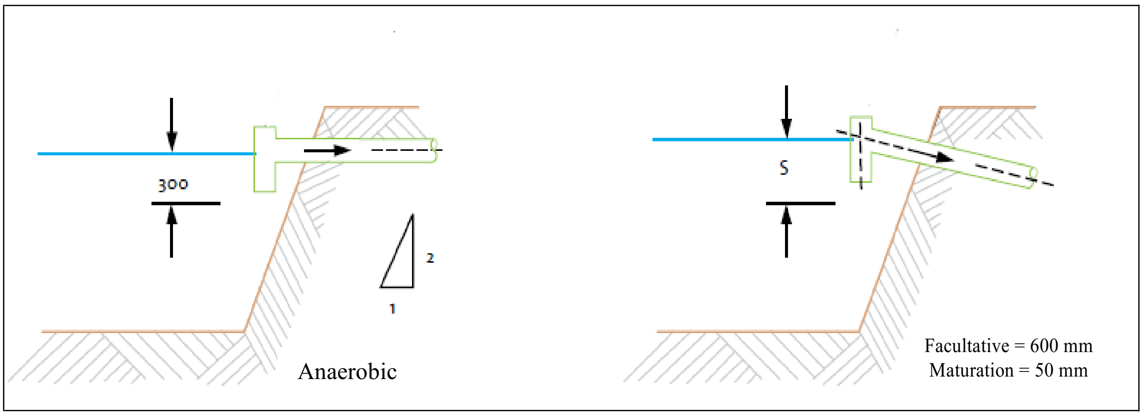


Figure 5-4: Pipe outlet designs (Source: Waste Stabilisation Pond Design Manual – Power and Water Corporation)

5.4 Pond Hydraulic Profile

All pond designs must include a hydraulic grade line calculation at both peak and average inflows. Flow attenuation within ponds should be considered to optimise interconnecting pipework sizing provided a minimum freeboard is maintained. *Note: Hydraulic design is to be completed for average flows, and tested for high and low flows, not the other way around.*

5.5 Baffles

Note: This section on baffles does not apply to aerated ponds or facultative ponds.

5.5.1 Plug flow vs completely mixed flow

The concept of 'plug flow' assumes that there is no mixing or diffusion as the wastewater moves through the pond. Alternatively, 'completely mixed flow' assumes the wastewater is instantaneously fully mixed upon entering the pond. Neither of these 'ideal-flow' conditions exist.

The aim is to maximise plug flow and minimise short circuiting so that treatment efficiency can be maximised.

In facultative ponds approximating plug flow should be achieved by strategic positioning of inlets and outlets and the momentum created through the pond. See 5.2 for approaches to maximising plug flow in facultative and maturation ponds.

NOTE: ***Plug flow must not be initiated with long baffles in facultative ponds.*** Design loading rates essentially give the ratio of BOD to pond area, and although the treatment rate is known to be non-linear, this is averaged across the pond to determine the design area of the pond. Installing baffles across facultative ponds therefore creates sub-zones within the pond and the first zone, in particular, will exceed the design loading limits, and probably also the pond failure envelope leading to poor treatment, front end sludge accumulation and odours.

In maturation ponds where much of the BOD is already burned off, extensive baffling to achieve plug flow can be helpful, with effective width to length ratios of up to 1:10 being quite acceptable.

5.5.2 Sludge Deposition and Wind

Sludge accumulation in WSPs is not evenly distributed throughout the pond. Sludge deposition is influenced by the following factors:

- Inlet location
- Inlet velocity
- Inlet direction
- Prevailing wind direction
- Pond geometry

Sludge accumulation at or near pond inlets is an obvious expectation due to the rapid settling of solids in that zone. In general, less sludge is accumulated immediately at the inlet due to the inlet velocity component with a mound forming a short distance from the inlet and then tapering away in the direction of the dominant momentum vectors through the pond.

Studies by Shilton *et al* (2003) showed that in correctly designed ponds the wind power does not significantly impact the input power of the inlet and therefore the overall momentum in the pond. The impact of wind is generally across the surface and therefore two dimensional in its impact. An exception to this is in conditions of high winds which may have short-term three-dimensional impacts on the pond momentum and could induce pond turnover.

Shilton *et al* (2003) offers three reasons why the inlet power may not be the dominant in some pond systems:

1. Overly large inlets are often used, which means that the inlet velocity (and its power input) is significantly reduced.
2. A significant number of ponds in current use are oversized with larger surface areas than modern designs. This increases the relative influence of the wind.

3. For periods of time the wind speed will be significantly higher than the average value used in the calculations.

An impact of wind across the surface is to drive accumulated surface scum into the pond corners where it settles out as sludge creating a sludge mound in the receiving corners. This has been observed at Northam where settled scum has caused the sludge layer to rise to the pond surface at the down-wind corners. Whilst it is always preferable to design pond systems so that the prevailing winds are in the opposite direction of the general flow through the pond, this is not always possible, potentially leading to sludge accumulation caused by scum near the outlet. To prevent this, an outlet shield (or baffle) should be constructed so that settled scum accumulates behind the shield. See **Figure 5-5**.

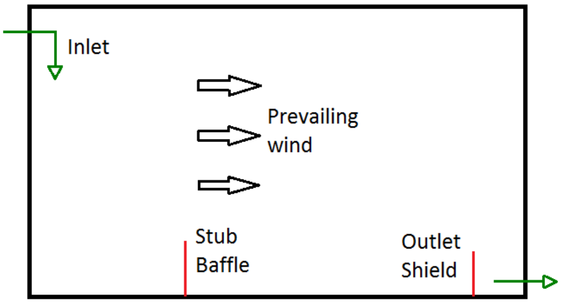


Figure 5-5: Outlet shield keeping settled scum from the outlet

5.5.3 Horizontal (transverse) baffling

Definitions:

Horizontal baffles are baffles that extend from the floor to above the top water level in a pond causing water to flow around the end of the baffle and not over or under the baffle.

Vertical baffles are designed to cause the water to flow over and under sequential baffles. The use of these is not described in this document.

Transverse baffles are positioned across the width of the pond (as in **Figure 5-6**), whilst ***longitudinal baffles*** are positioned along the length of the pond.

Note: All baffles should be designed with due consideration of the various desludging methods that may be applied to plants. They must be sturdy enough to allow desludging around them, and not cause "difficult to reach" spaces

Work by Watters *et al* (1973) demonstrated the following:

- Horizontal baffles are more efficient than vertical (under and overflow) baffles;
- Incorrectly designed baffles can induce short circuiting;
- Long, evenly spaced baffles of about 70% of the width of the pond gave improved performance when compared with 50% which led to short circuiting, and 90% which tend to cause higher velocities.
- Longitudinal baffles are no more efficient than transverse baffles.

Field work by Shilton *et al* (2003) was undertaken on a secondary pond with the key efficiency parameter tested being the influent and effluent CFU. This being the case, it is again emphasised that pond baffling applies to maturation ponds, and not to facultative ponds except for short stub baffles as discussed above.

Work by Shilton was carried out using evenly spaced baffles extending across 70% of the width of the pond. Key findings were as follows:

- A single baffle gave little benefit over an unbaffled pond;
- Two baffles gave a stepped improvement in performance;
- Evenly spaced baffles gave the best performance;
- Four and more baffles continued to give improvements, but with diminishing returns.

If maturation ponds are baffled, no more than 4 baffles should be used in any single pond.

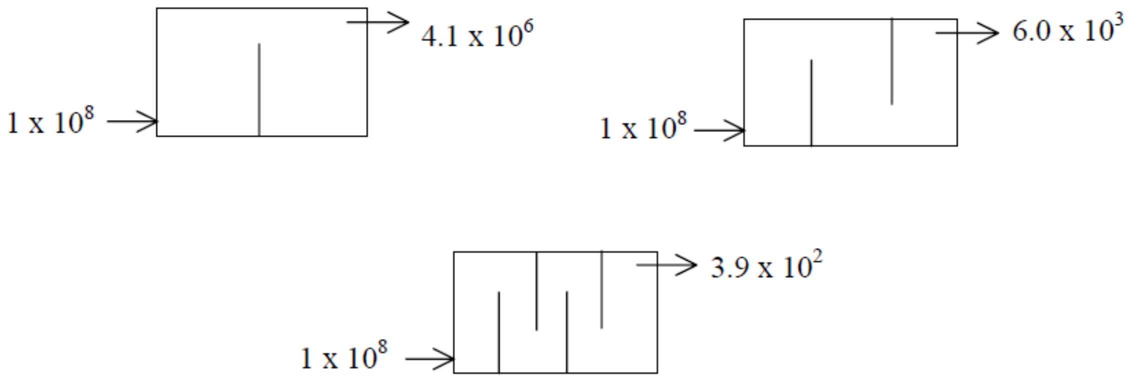


Figure 5-6: Improvements in CFU removal with increasing number of baffles (Shilton et al (2003))

5.6 Emergency overflows

Emergency overflows shall be provided where flooding and rising pond levels (such as caused by blocked outlets) may cause embankment washout. Overflows are provided by shallow, wide depressions in banks, and should generally be as follows:

- placed to minimize external erosion (preferably placed near piped outlets to maintain flow path, and if possible, where the external bank height is lowest);
- overflow invert to be a maximum of 0.3 m below Top of Bank (TOB) for internal (pond to pond) overflows;
- overflow invert to be a maximum of 0.2 m below Top of Bank (TOB) for external overflows;
- minimum (flat) weir length = 2 m; and
- maximum 1:6 (V:H) side slope (parallel to bank axis) to allow for vehicle movement.

Example on drawing JT14-3-6 (Broome WWTP)

5.7 Freeboard

The minimum pond freeboard, measured from the invert of the outlet (tee or weir) to the Top of Bank (TOB) shall:

- be 0.50m (measured vertically);
- consider pond dimension, wind fetch and embankment run-up based on prevailing wind conditions;
- consider the impact of embankment material on the run-up potential.

Note that emergency storm spillways inserted for embankment protection are incorporated within the 0.5m or calculated minimum freeboard.

If an existing pond system has been built with 0.3m freeboard, and includes spillways to prevent overtopping, it should not be necessary to do any work to raise the embankment unless the wind fetch and run-up could cause the pond to overtop.

5.8 Spillage

Drainage back to process ponds shall be provided for all inlet works spillage (generally by sump and gravity pipework). Where practical, drainage back to process units shall be provided for all other maintenance activities (e.g., mechanical process unit repair where spillage cannot be managed by normal maintenance procedures). In the case where it is not practical (excessive distance, inadequate elevation, or low-cost benefit), the designer should consult the operations representative to ensure requirements are met to minimise spillage.

6 CIVIL DESIGN

6.1 General Arrangement

General considerations

The following are general considerations when siting a WSP system:

- Waste stabilisation pond land area should allow for expansion, sludge drying and stockpiling and future services and buffer distances. The total plant area will be considerably greater than the process mid depth area;
- Pond geometry should conform, as far as possible, with the topography. The contours should be used to provide inter-pond hydraulic head to minimise earthworks and the use of pumping;
- Contour plans and Australian Rainfall and Runoff data should be used to identify drainage paths and position drainage to divert runoff around the pond plant and back to natural drainage lines;
- Allowing for access to maintenance and desludging vehicles;
- To minimize earthworks, - the site should be flat or gently sloping.
- Siting in an open area to take advantage of the sun and wind, which assist the efficient operation of the facultative pond and thus improve the quality of the discharge.
- A tree belt may be required if high winds blow significant volumes of sand into the ponds. Trees must not shade ponds, so must be a sufficient distance away;
- Avoiding sites that are likely to flood, have steep slopes that run towards a waterway, spring, or bore hole;
- Fencing of the site is a requirement;
- An office and utilities are to be provided based on project requirements.

Odour Buffer

Ponds should be located to provide a minimum buffer (generally at least 500 m) downwind from the community they serve and away from any likely area of future urban expansion.

Ponds should not be located within 2 km of airports (check with aviation authorities), as birds attracted to the ponds constitute a risk to air navigation.

Physical Layout

The areas calculated by the process design procedures for facultative and maturation ponds are mid-depth areas. Dimensions calculated from them need to be corrected for the slope of the embankment as shown in Figure 6-1. In general, a batter slope of 1V:3H for facultative and maturation ponds is preferred. Steeper slopes may be used for anaerobic ponds, but embankments may need protection, and geotechnical advice is required.

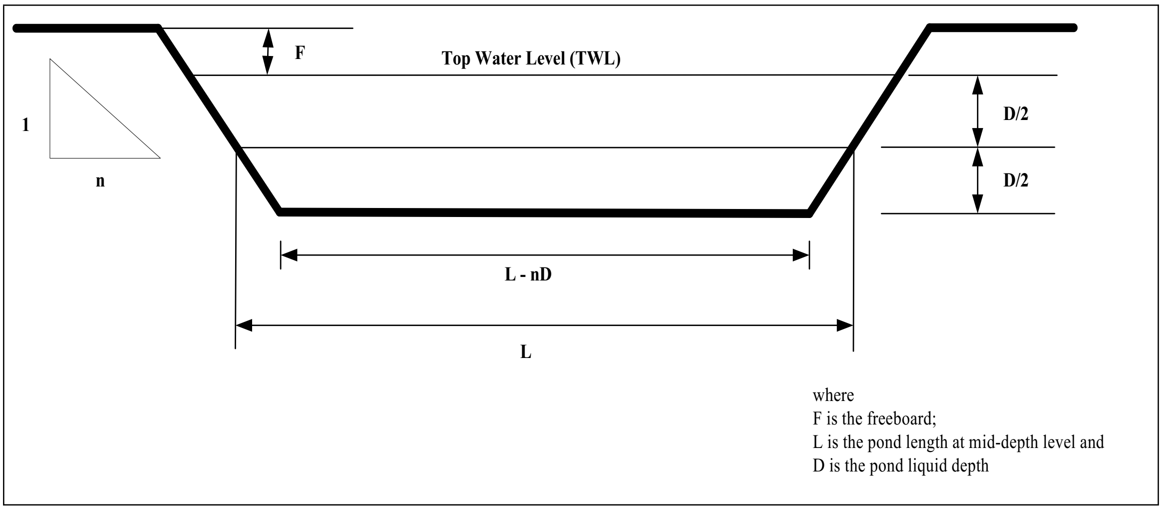


Figure 6-1: Calculation of top and bottom pond dimensions based on mid-depth

Tops of embankments should be a minimum width of 4 m (not including allowance for embankment protection)

Note that lining termination trenches will add 0.8 to 1.0 m to each side that is lined. That is, the intermediate bank of lined ponds will be 6 m wide.

Sharp corners provide hydraulic dead areas and are to be avoided. Pond corners are to be rounded.

6.2 Lining and Embankment Protection

Lining

Ponds should be lined if the soil is too permeable. General guidance on the lining requirements for different soil permeability's (k) is:

- $k > 10^{-6}$ m/s: the soil is too permeable, and the ponds must be lined;
- $k < 10^{-7}$ m/s: some seepage may occur but not sufficient to prevent the ponds from filling;
- $k < 10^{-8}$ m/s: the pond will seal naturally and
- $k < 10^{-9}$ m/s: there is no risk of groundwater contamination

Works Approval conditions may require a permeability of $k < 10^{-9}$ m/s. Conditioned and compacted clay is the preferred lining material, but in-situ clay may not be capable of being processed to meet this requirement

.Order of Preference	Lining	Internal Batter Protection	Pond Floor Protection	Reference Plan
1	Clay, min 500mm thick on internal batter and 300mm thick on floor	Minimum 100mm crushed graded igneous rock over geotextile on liner	Not required	
2	Bituminous based geomembrane (e.g., Coletanche or Siplast Teranap products)	Anchor trenches at bottom and top of embankments.	Anchor trenches across the floor to prevent uplift from wind when empty. Lay over 200mm of compacted fill – no sharp objects	JT71-3-34
3	Geo-synthetic clay liner (GCL)	Minimum 100mm of crushed graded igneous rock over geotextile over 100-200mm of clean sand over GCL over 200mm of clean sand, no sharp fill	100 to 200mm of clean sand over GCL over 200mm of clean sand, no sharp fill	JT14-003-002
4	Polyethylene (PE)	Provide weighting at all changes in direction	100 to 200mm of clean sand over PE over 200mm of clean sand, no sharp fill	

Table 6-1: Pond lining (and protection) in order of preference

Note: All lined ponds must have egress ladders as per the latest version of Plan BN85-007-001.

Embankment protection

A stable and impermeable embankment core shall be formed, whether chosen from an available local or imported soil. The following geotechnical considerations should be taken when constructing the embankment:

- Embankments must be well constructed to prevent seepage, excessive settlement, and erosion over time;
- Embankment slopes are commonly 1 (vertical) to 3 (horizontal) internally and 1(vertical) to 1.5-4.0 (horizontal) externally. A gradient of 1:4 externally if space allows makes for easier maintenance and reduced erosion;
- Slope stability should be ascertained according to standard soil mechanics procedures for small earth dams;
- External embankments should be protected from storm water erosion by providing adequate drainage;
- Internal embankments should be protected from wave action erosion;
- The top of banks shall be finished with a minimum 150 mm thick compacted road base (gravel or limestone);
- Frequently trafficked areas such as for chemical deliveries are usually bituminized (as agreed in user requirement documentation).

Note: Although hydro testing is the best means of proving the pond construction the DWER does allow independent inspection of liners during installation. If the independent part certifies the installation, then no hydro-testing is required.

6.3 Pond Desludging

6.3.1 Background

Provision of appropriate facilities for desludging of ponds forms an integral part of the design process. For background information in understanding desludging and to aid selection of a desludging method, refer to Wastewater Pond Desludging - Selecting a desludging method

Three recognised methods of desludging facultative, maturation and (possibly) storage ponds are available and described in Table 6-2. Desludging of anaerobic ponds remains challenging and pumping from desludging pipes installed during construction is the current approach.

Desludging method	Description
Drain & Dry or long-term Off-line desludging	Desludge after water has been pumped away and sludge is dried in-situ in the pond over several weeks or months then removed by loaders and trucks.
Dredge method or On-line desludging	Desludging a pond while it continues to receive wastewater inflows and continue to perform treatment functions. Floating dredges specially designed to minimise sludge disturbance by mobilising the sludge to a suction point from where it is pumped away - typically to geobags for dewatering.
Slurry method or short-term Off-line desludging	Desludge after water has been pumped off and the residual sludge mixed and pumped to geobags for dewatering. The process typically only takes a couple of weeks.

Table 6-2: Methods of desludging ponds

Desludging method	Key advantages	Key disadvantages
Drain & Dry	Removes 100% of the sludge Normally removed directly from site so no geobag area	Loss of process capacity during desludging Duplication of infrastructure, increases capital costs Access ramps required Potential liner damage
Dredge method	Pond continues normal operation Duplication of infrastructure not required Access ramps are not required	Only removes ±85% of sludge from horizontal surfaces. Requires infrastructure for dewatering in geobags or sludge drying beds Risk of damage to liner
Slurry method	Shorter off-line duration than Drain & Dry method	Removes less sludge than dredge Loss of process capacity during desludging Duplication of infrastructure, increases capital costs Access ramps required Requires infrastructure for dewatering in geobags or sludge drying beds Risks leaving “fluffed up” sludge behind in pond

Table 6-3: Advantages and disadvantages of different desludging methods

Table 6- provides some key advantages and disadvantages of different methods of pond desludging. This must be considered in future designs whether for new plants or plant upgrades.

6.3.2 Process requirements

Drain & Dry

This approach requires that ponds need to be taken off line (normally one at a time), and inflow redistributed to parallel pond(s) in the system i.e., the system must have redundancy built in.

Key considerations are:

- Desludging activities will normally be undertaken during the summer months (especially in the southern half of WA), and the built-in redundancy can therefore be based on water temperatures for the months of November to April.
- During desludging the pond loading rates may be increased temporarily as follows:
    - Anaerobic ponds: 400g/m^3/d for temperatures above 25°C;
    - Facultative ponds: 400kg/ha/d for temperatures above 25°C, ***and*** remain within 90% of the pond failure envelope given in 4.3.2.
- Redundancy means that ponds are never loaded to their design capacity except possibly during desludging operations. Whilst this may not be a problem when inflows are near to the plant hydraulic design capacity, the opposite could be a problem in the early years of the life of the plant when inflows are still low. Ponds could be under loaded, and the designer must check for this. Refer to 4.2.1 and 4.3.1 for minimum loading rates.

On-line dredging

Figure 6-2: Online unmanned dredge

As dredging is undertaken whilst the ponds continue with their normal operations, no special process considerations are required.

The pumping rate is in the order of 4000Litres per minute, and must be considered, both in the context of return flows, and the effect of the pond hydraulics.

One practical issue affecting the process relates to the type of dredge used. Although not a specific problem for the designer, some dredging machines stir up the sludge creating a sludge cloud in the water body above the sludge. This cloudy sludge takes a long time to settle (up to several weeks) and during this time, treatment is compromised as light penetration is inhibited.

Acceptable dredge type

The dredge should be cable driven auger dredge (the auger should be a helical screw type which drives the sludge into the central pump inlet). The dredge should be unmanned. This method allows the sludge to be planed and removed efficiently and effectively with little disturbance. The auger dredge should have wheels to prevent any damage to the liner.

Unacceptable dredge types

- A pump on a float which 'potholes' and has to be lifted, moved and dropped on a regular basis. This system also results in solids shoved onto the sides and banks of the ponds.
- During dredging it is necessary to ensure that mobilised solids do not impact effluent quality. Unacceptable dredge types are those that shovel sludge to one side into windrows and build-up on the banks. These cause a suspended white cloud in the water matrix that takes a long time to settle and allow normal processing to proceed.
- In considering a dredge type, it ***must not*** be "self-propelled" as these models disturb the sludge blanked, and it is difficult to track which areas have been desludged.

Slurry method

Process requirements for the slurry method are similar the Drain & Dry method. In addition though, care must be taken not to damage the pond embankments whilst dragging the long lengths pipe with excavators.

6.3.3 Access requirements

Drain & Dry

- Key considerations are:
- Access to the ponds for removal of sludge. This requires:
  - Adequate access roads and turning circles;
  - Access ramps into the ponds. See example on JT1-2-1.
- Disposal locations certified to receive the removed sludge (including Alum sludge in some cases).

On-line dredging

Dredges are brought to site on specially designed trailers, and they are lifted into the ponds with a crane. This means that access ramps into the ponds are not required, but the following must be considered:

- Road access for the truck and trailer – this applies particularly to turning circles, and gate widths;
- Dredges are approximately maximum 7-8m long x 2.4m wide;
- A crane pad for lifting the dredge into the pond is useful but can also be lifted with a Franna;
- Flat area on site for a 6m (20ft) container which contractors would need to store their pipework and other dredge/dewatering equipment, near to the geobag area;
- Flat area (approximately 3 × 6m) for laydown of dosing equipment, also near the geobag area;
- Vehicle access is required around the geobag laydown area for the removal of geobags once they have sufficiently dewatered.
- Minimising the number of dredge hoses that can get in the way of the dredges or cause damage to embankments or get in the way of dredges.

Slurry method

Access requirements for the slurry method are the same as for the Drain & Dry method.

6.3.4 Dewatering and drainage

Drain & Dry

As sludge is left to dry in the ponds and then trucked directly from the site, no other dewatering infrastructure is required on the site. Note that ponds are offline for a long time (months).

On-line dredging

Reference drawings: JT71-8-1 and JT71-8-2.

Dredge equipment collects the sludge from an operational pond and pumps it to geobags. The sludge is dosed with a diluted polymer to provide greatly improved sludge-water separation, and this allows dewatering once in the geobag.

Geobags are a large (typically 7 metres by 20 metre) woven polypropylene bags designed to retain solids and pass liquid. Geobags of this size have a maximum filling height of up to 1.8 m but some suppliers recommend operating them to 1.5 m to maintain safety margins unless a full containment bund exists.

Although other dewatering methods such as sludge drying beds, centrifuges etc. may be available, geobags are the preferred dewatering method. In some circumstances drying beds and other methods may be preferred due to cost and / or space constraints.

The geobag laydown area is required to comply with the following:

- A hardstand area at least 30m long and 8m wide for each geobag placed. The 30m dimension includes the 4m wide drainage channel referred to below;
- The hardstand may be concrete or compacted gravel:
    - Concrete should be used where anaerobic ponds require desludging on a regular basis, or where regular backwashing of filters occurs on the site;
    - Gravel hardstands are appropriate where ponds are only desludged every several years. During desludging operations, the gravel hardstand is covered with HDPE sheeting to contain and drain the water to a specially designed drainage channel.
- Gradient to be 1:200 in the lengthwise direction of the geobags. All flow **must** be parallel with the geobags. The gradient and flow direction are both safety issues that must be adhered to. Failure to adhere to this could cause the geobags to slew without warning with the potential for significant damage and / or personal injury.
- A 4m wide concrete drainage channel is required at the downstream end of the hardstand area which will drain the water to central sump from where it can either flow back to the ponds under gravity, or be pumped back to the ponds if gravity flow is not possible;
- The Geobags require the sludge to be dosed with polymer so allowance should be made for a water supply. An area must be provided for contractors to place their temporary polymer dosing equipment;
- A wash-down hose point is useful for cleaning down the hardstand area following the removal of geobags.

Slurry method

The slurry method requires a geobag area similar to that required for the dredge method.

6.3.5 Anaerobic ponds

Varying degrees of success have been reported on the desludging of anaerobic ponds. Historically they have been fitted with one or more pipes for sludge withdrawal, but the actual pumping of sludge has proved to be problematic because the sludge tends to rat-hole sucking water from above the sludge blanket instead of from the blanket itself. Solutions being trialled are the following:

- Providing several extraction pipes into the bottom of the anaerobic zone linked to an external manifold where sludge can be pumped out zone for zone.
- In addition to the first point, sludge must be pumped for short durations from each zone, with the operator visually monitoring the withdrawn sludge for signs of rat-holing, and shutting down the pump as soon as it becomes evident.
- If direct pumping proves to be problematic, then on-line dredging is probably the best alternative. Modern dredges can reach down to a depth of 5m and more which is the recommended depth of anaerobic ponds. Dredging would normally not proceed to that full depth due to existing extraction pipework close to the floor.

7 APPENDIX A

Referenced Resources and Documents

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8 APPENDIX B

Preferred Terminology

The table below contains preferred terms for use by the Designer in Corporation mechanical designs.

Preferred Terminology Units	**Non-preferred**
Bend	Elbow
Discharge (pump)	Delivery, outlet
Drinking water	Potable water
Ejector	Injector
GRP	FRP
Impeller	Impellor
L/s	l/s
MLD	ML/d, Ml/d
Nominal diameter - DN	ND
Non return valve	Check valve
Pumpset	Pump unit, pumping unit
Pump station	Pumping station
Sewage pump station	Wastewater pump station
Suction (pump)	Inlet, intake

9 APPENDIX C

Abbreviations, acronyms and symbols

The table below contains terms and symbols used in this Guideline and more generally in the water industry.

Term	Description
AADF	Average Annual Daily Flow = the total annual flow reaching the WWTP in a calendar year divided by 365. It is useful for understanding annual plant throughput but should not be used as a basis for process design as it includes flows from wet weather events. For process design purposes ADWF and PDWF should be used.
ADWF	Average Dry Weather Flow = the average flow of incoming used water measured in the three driest (non-rainfall) months of the year.
ABS	Acrylonitrile – Butadiene – Styrene (pipe and fittings)
AHD	Australian Height Datum
AISI	American Iron and Steel Institute
ANSI	American national Standards Institute
API	American Petroleum Institute
AS	Australian Standards
ASM	American Society of Metals
ASME	American Society of Mechanical Engineers
ASTM	American Society for testing and Materials
AWS	American Welding Society
BEP	Best Efficiency Point
BFJ	Butt-fusion joint
BJ	Butt joint (plain ends)
BOD	Biochemical oxygen demand
BS	British Standard
BSP	British Standard Pipe
BSI	British Standards Institute
BWL	Bottom Water Level
CI	cast Iron
CIP	Clean-in-place
CML	Cement mortar lined
COD	Chemical oxygen demand
CS	Carbon steel (pipe)
CSA	Canadian Standards Association

Term	Description
©	Copyrighted
Cv	Flow coefficient, flow factor or valve coefficient (imperial)
dBA	Decibel – A weighted scale
DI	Ductile Iron (pipe and fittings)
DICL	Ductile iron cement lined
DIN	Deutsches Institut fur Normung (Germany)
°C	Degrees Celsius
DN	Nominal diameter
EAS	Excess activated sludge
EFJ	Electro-fusion joint
EPDM	Ethylene propylene diene monomer rubber
ESJ	Elastomeric seal joint
FAD	Free air delivered
FBE	Fusion bonded epoxy
FJ	Flange joint (bolted)
FRP	Fibreglass reinforced plastic
g	Acceleration due to gravity – 9.81 m/s^2
GDA	Geocentric datum of Australia
GL	Gigalitres
GRP	Glass reinforced plastic (pipe)
HBW	Brinell hardness number
HDPE	High density polyethylene
HGL	Hydraulic Grade Line
H	Head of water in m
Hz	Hertz (cycles per second)
h	Hour
HRB	Rockwell B (hardness)
HRC	Rockwell C (hardness)
IEC	International Electrotechnical Commission
IFJ	Flush joint
I/O	Input/Output
IRHD	International rubber hardness degree

Term	Description
ISO	International Standards Organisation
JIS	Japanese Industrial Standard
k	Absolute pipe roughness in mm
K	Resistance coefficient
kg	Kilogram
kL	Kilolitre
kN	Kilonewton
kPa	Kilopascal
Kv	Flow coefficient, flow factor or valve coefficient (metric)
kW	Kilowatt
L	Litre
L/s	Litres per second
m	Metre
m^2	Square metres
m^3	Cubic metres
mm	Millimetre
m/s	metres per second
MDPE	Medium density polyethylene
ML	Megalitre
MLD	Megalitres per day
MLSS	Mixed liquor suspended solids
MSCL	Mild steel cement lined (pipe and fittings)
N	Speed in revolutions per minute
NACE	National Association of Corrosion Engineers
NATA	National Association of Testing Authorities
NDT	Non-destructive testing
NEMA	National Electrical Manufacturers Association
NFPA	National Fire Protection Association
Nm	Newton metres
NPSH	Net positive suction head
NPSHa	Net positive suction head available
NPSHr	Net positive suction head required

Term	Description
NZS	New Zealand Standards
OEM	Original equipment manufacturer
OH&S	Occupational health and safety
O&M	Operations and maintenance
PE	Polyethylene (pipe)
PDWF	Peak Dry Weather Flow applies to the daily diurnal flow pattern. As a factor, it is the ratio of the peak hourly flow (usually late morning) to the ADWF measured in the three driest (non-rainfall) months of the year (Usual range is 1.5 to 2.0).
P&ID	Piping & Instrumentation Diagram
PFD	Process Flow Diagram
pH	Measure of acidity/alkalinity (from German *potenz* = power, and *H*; the symbol for hydrogen). A logarithmic index for the hydrogen ion concentration in an aqueous solution.
PLC	Programmable logic controller
PN	Nominal pressure
ppm	Parts per million
PU	Polyurethane
PVC	Polyvinyl chloride
PWWF	Peak Wet Weather Flow is usually caused by infiltration of water into the collector system during rainfall events. It can be of short (one hour) or long (days) duration. As a design factor it is the ratio of the peak hour flow reaching the plant to the ADWF. (Usual range is 1.9 to 2.2 for Metro plants). To be used for hydraulic design.
Q	Flowrate, capacity or discharge rate
®	Registered
RCD	Residual current joint
Re	Reynolds number
rpm	Revolutions per minute
RPS	Raw primary sludge
RPZD	Reduced pressure zone device
RRJ	Rubber ring joint
s	Second
RST	Rotary screw thickener
SANZ	Standards New Zealand
SCADA	Supervisory control and automated data acquisition
SI	Systems International d' Unites

Term	Description
SLR	Solids loading rate
SPS	Strategic Product Specification
SS	Stainless steel
SSJ	Spherical slip-in welded joint
SWJ	Solvent welded joint
TDH	Total developed head in metres
TEAS	Thickened excess activated sludge (= TWAS)
™	Trademark
TOC	Total organic carbon
TSS	Total suspended solids
TWL	Top Water Level
uPVC	Unplasticized Polyvinyl Chloride (pipe and fittings)
UV	Ultraviolet
V	Volts
VSD	Variable speed drive
VVVF	Variable voltage variable frequency drive (= VSD)
WLL	Working load limit (replaces SWL)
WAS	Waste activated sludge (= EAS)
WSAA	Water Services Association of Australia
WTIA	Welding Technology Institute of Australia
WRRF	Water Resource Recovery Facility
WWTP	Wastewater Treatment Plant

10 APPENDIX D

The table below contains standard units and relationships used by the Corporation.

Quantity	Unit	Relationship
Flow	L/s	Rate of flow
	MLD	L/s x 86.4
Volume	L	Amount of volume
	kL	L / 10^3
	ML	L / 10^6
	GL	L / 10^9
Length	mm	Linear dimension
	m	mm / 10^3
Area	m^2	Areal measure
	ha	m^2 / 10^4

The table below lists SI unit prefixes and symbols for reference.

Fraction or Multiple	Prefix	Symbol
10^{-1}	Deci	d
10^{-2}	Centi	c
10^{-3}	Milli	m
10^{-6}	Micro	µ
10^{-9}	Nano	n
10^{-12}	Pico	p
10	Deca	da
10^2	Hecta	h
10^3	Kilo	k
10^6	Mega	M
10^9	Giga	G
10^{12}	Terra	T

END OF DOCUMENT