Operation of Municipal Wastewater Treatment Plants

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This report is part of the Norwegian - Polish joint project "Upgrading Wastewater Treatment Plants in Poland", where the transfer of knowledge concerning operational procedures and problems are considered to be an important task.

The manual covers most unit processes involved in municipal wastewater treatment plants in Central Europe, and for each process the following items are presented:

- Introduction
- Description of process
- Process control
- Troubleshooting

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3. Driftsovervåking	3. Process control
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Troubleshooting	Problemløsning

Preface

The operation manual in hand is part of a project called **"Upgrading of Wastewater Treatment Plants in Poland"** which is included in the Programme of Cooperation between the Norwegian Ministry of Environment and the Ministry of Environmental Protection, National Resources and Forestry in the Republic of Poland.

The project has been executed by the Norwegian Institute for Water Research, NIVA in cooperation with Aquateam - Norwegian Water Technology Centre A/S. As Polish cooperation partner, the Polish Ministry appointed Centrum Techniki Budownictwa Komunalnego.

The intention of the Programme was to transfer know how and experience on wastewater treatment plant operation to the Polish counterpartners and to get started a movement or a plan of action in order to improve the operation of existing wastewater treatment plants in Poland.

At the plants included in the Programme very good results with regard to operation performance have been achieved so far, and our Polish counterpartners have been trained to evaluate operation performance and treatment efficiency in order to continue the work started.

An important part of the programme is the training of wastewater treatment plant operators. Our aim was to conduct workshops for the operators by preparing teaching materials, lectures, etc. Our Polish colleagues have in stead asked us to prepare a handbook for Operation of Municipal Wastewater Treatment Plants. The operation manual in hand is prepared by Aquateam - Norwegian Water Technology Centre A/S who is subcontracted to NIVA on this project.

April, 1995

Norwegian Institute for Water Research
Svein Stene-Johansen
Project Manager

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1 Introduction to Wastewater Treatment

The primary purpose of wastewater treatment is to protect the health and well-being of our community. Wastewater treatment is the process by which solids in the wastewater are partially removed and changed from complex, organic solids to minerals or relatively stable organic solids. Wastewater treatment may also include removal of nutrients (phosphorus and nitrogen) and disinfection.

By providing a wastewater treatment facility, we are allowing the self-purification ability of water to occur under controlled conditions. If the wastewater treatment facility was not provided, this purification process would occur naturally in a pond, lake or several kilometres of a rapidly flowing stream.

1.1 Objectives of Wastewater Treatment

The primary objective of wastewater treatment plants (WWTP) operation is to meet the permit requirements or, if the facility is nondischarging, the applicable requirements of the regulatory agencies for groundwater protection. At the same, the operation must protect the safety, health, and well-being of the plant's employees and neighbours. In establishing the requirements for wastewater treatment, the regulatory agencies may consider the following, as well as compliance with minimum regulatory requirements:

- Prevention of disease,
- Prevention of nuisances,
- Avoidance of water supply contamination,
- Elimination of all pollutant discharges to navigable waters,
- Maintaining clean waters for the propagation and survival of fish and other aquatic life,
- Protection of waters for personal bathing and recreational use,
- Preservation of pristine waters for ecosystem protection, and
- Conservation of water.

Although the effluent quality is usually limited for a specific discharge and watershed, the volume and the physical, chemical and biological characteristics of the treatment plant influent continually change. Some changes are short-term due to seasonal, monthly, weekly, or daily fluctuations in the quantity and composition of the community's wastewater. Other changes are long-term, arising from alterations of the service area's population, social quality of the receiving water and the health and well-being of the people affected may depend on the ability of the operator to recognise and respond to potential treatment problems. These responsibilities demand a thorough knowledge of the treatment facility and wastewater treatment technology. The following sections of this manual will discuss wastewater treatment processes and sludge management.

1.2 Wastewater Collection System

The wastewater collection system is composed of a network of sewer pipes, conduits, tunnels, structures, devices, equipment, and appurtenances for the collection, transportation, and pumping of wastewater. Generally, flow through such a system is by gravity or a combination of gravity and pumping where topography does not allow gravity flow alone.

Three principal types of municipal sewers exist. Sewers that receive wastewater from residential, commercial, institutional, or industrial sources, together with small quantities of groundwater infiltration and stormwater inflow are called sanitary sewers. Systems that convey stormwater runoff and other drainage while excluding sanitary wastes are called storm sewers. Sewers that convey both sanitary wastes and stormwater are called combined sewers. Stormwater conveyance and treatment are outside the scope of this manual.

The type and characteristics of the wastewater collection system may profoundly affect operation of the WWTP. For example, the high flows and heavy sediment loads entering the plant from a combined system during and after a storm may adversely affect operation of the treatment processes. Similar adverse effects may result from excessive infiltration and inflow in a poorly maintained sanitary system. Very extensive collection systems may contribute foul-smelling septic wastewater to the plant headworks unless odours in the system are controlled.

1.3 Preliminary Treatment

Preliminary treatment of wastewater at the headworks of the WWTP removes materials that might harm the headworks or impair operation of downstream processes. Substances typically removed include hydrogen sulphide, wood, cardboard, rags, plastic, grit, grease, and scum. Methods and equipment used to remove these materials may include chemical addition, preaeration, bar racks, screens and shredding devices, and grit chambers.

1.4 Primary Treatment

Following preliminary treatment, primary treatment removes suspended and floating material. Well-designed and operated primary treatment facilities may remove as much as 60 to 75% of the influent suspended solids and up to 20 to 35% of the total BOD_5. However, colloidal solids, dissolved solids, and soluble BOD_5 will not be removed. Therefore, primary treatment must be supplemented by secondary and possibly other advanced wastewater treatment (AWT) to attain the required effluent quality.

1.5 Secondary Treatment

Secondary treatment reduces the concentrations of dissolved and colloidal organic substances and suspended matter in the wastewater. Generally, secondary treatment reduces SS and BOD_5 to a level between 10 and 30 mg/l.

Most secondary treatment processes involve biological treatment. Most of the many biological treatment processes for secondary treatment can be classified as suspended growth or attached growth (biofilm) systems. Each system relies on an established mixed population of micro-organisms in the presence of oxygen and trace amounts of nutrients. The micro-organisms consume organic material in the waste to sustain their life processes and to produce new micro-organisms. In attached growth systems, the mass of micro-organisms affecting treatment are attached to supporting media. Examples of attached growth systems include trickling filters, moving bed biofilm systems and rotating biological contactors. Suspended growth systems have reactors containing microorganisms suspended in the wastewater. These systems include lagoons and the many variations of the activated sludge process.

The effluent from suspended growth and attached growth reactors contains high concentrations of biological solids in suspension. Before further treatment or final discharge, the solids must be removed by means of a liquid-solids separation process. Most systems rely on final settling (sedimentation), although other methods such as flotation may be used.

1.6 Physical-Chemical Treatment

Physical-chemical unit processes may be used to remove pollutants from wastewater although few municipal WWTPs outside Scandinavia employ physical-chemical processes alone. Physical processes for removing solids include screening, sedimentation, and filtration. Chemical processes used as an aid to sedimentation include chemical coagulation and flocculation. Activated carbon adsorption is a physical-chemical process for removing organic pollutants. Chemical processes for removing nutrients include breakpoint chlorination for nitrogen reduction and lime/iron or aluminium addition for phosphorus reduction.

1.7 Advanced Wastewater Treatment (AWT)

Advanced wastewater treatment may be used to reduce the concentrations of nutrients, nitrogen or phosphorus, and soluble organic substances to levels below those normally attained through secondary treatment. The AWT processes may include physical, chemical, or biological processes, or a combination of these. The treatment requirements for compliance with the effluent limitations in the permit usually influence the types of AWT processes, if any, selected for a specific plant.

1.8 Disinfection

Disinfection of WWTP effluent inactivates or destroys pathogenic bacteria, viruses, and amoebic cysts commonly found in wastewater. Pathogens may cause outbreaks of waterborne diseases such as typhoid, cholera, paratyphoid, bacillary dysentery, poliomyelitis, and infectious hepatitis. The demand for reuse of plant effluent for irrigation and recreation purposes and the requirements to control toxins, including chlorine, to protect aquatic life are resulting in a change in the policies regulating disinfection. These changes, in turn, are leading to changes in disinfection practices.

Generally, disinfection processes may be classified as natural, chemical, physical, or radiation. Historically, chemical treatment using halogens, particularly chlorine, has dominated wastewater disinfection practices. Because of the impacts of chemical disinfection by-products (chlorinated hydrocarbons) on water supplies and the requirement for dechlorination of some discharges, other means of wastewater disinfection, such as ozonation and radiation, are gaining acceptance. In Europe, disinfection of the effluent is not common.

1.9 Effluent Discharge

Effluent discharge or reuse is a significant part of a wastewater treatment program. The method of effluent discharge or reuse and the required effluent quality influence the selection and operation of the various unit processes. Possible discharge methods include direct discharge to receiving waters, percolation, wetlands, land application, deep-well injection, and groundwater recharge. The country or community may have a number of alternatives for effluent discharge or reuse. Often, treated effluent is a resource that can be beneficially used in more than one way. Such use options might include agricultural reuse by the discharging community or direct discharge to support the aquatic environment of the receiving water.

Typical influent and effluent quality ranges for several treatment levels are shown in Table 1.1.

Table 1.1 Typical effluent quality for various treatment levels.

Water quality parameter	Raw influent	Primary effl.	Secondary effl.	ATW*-effluent
BOD_5 (mg/l)	100 - 250	100 - 200	10 - 30	5 - 10
SS (mg/l)	100 - 250	100 - 150	10 - 30	5 - 10
Total nitrogen (mg/l)	10 - 40	10 - 40	10 - 30	3 - 5
Total phosphorus (mg/l)	3 - 15	3 - 15	3 - 12	0.2 - 2

* Sophisticated processes can reduce ATW pollutant levels well below those listed.

2 Characteristics of Municipal Wastewater

Wastewater is basically the water supply of a community after it has been contaminated by various uses. There are many sources of wastewater including human and animal wastes, household wastes, infiltration of ground water, inflow of surface waters (storm water), and industrial wastes of various kinds. Physical characteristics can tell you a lot about the type of waste you are receiving at your facility as well as warn you of possible problems that may be developing.

For example, the colour of muncipal wastewater is grey, resembling dirty dishwater. A black colour, on the other hand, may indicate septic conditions or certain types of industrial loading. Many colour variations can occur and if the source and nature of the wastewater is known, you can use this as an operational tool to run your facility efficiently.

Odours of fresh municipal wastewater are usually described as musty, whereas a rotten egg odour would indicate septic condition in the collection system. Anaerobic decomposition of wastewater produces hydrogen sulphide, which calls for measures to increase the oxygen content of the liquid stream. In addition to indicating process problems, the presence of hydrogen sulphide raises concerns for other reasons; it is poisonous at relatively low levels, corrosive to concrete and copper, and potentially explosive. Also methane gas, which is even more explosive, may accompany hydrogen sulphide. The conditions that produce methane and hydrogen sulphide demand oxygen, often resulting in an oxygen deficient atmosphere. Therefore, the presence of hydrogen sulphide and methane demand extreme caution and rigorous application of the established safety procedures.

The normal influent temperature of municipal wastewater is a few degrees warmer than the water supply of the community. Higher temperatures may indicate industrial cooling water discharges, whereas cooler water may indicate infiltration or inflow water.

The flow variation in a community may be great, especially for smaller cities. The impact of this variation can cause severe problems in your plant. Combined sewer system upstream the treatment plant will generate great variation both in flow and contents. The flow in rainy days can be many times the normal flow in such a system. Daily flow variation in dry weather depends largely on the size of the collection system. In general, the smaller the collection system, the greater the daily variation. Other influences on flow variation include the number and type of pump stations, the types of industries served, and the population characteristics.

Total solids in wastewater includes all solids and can be broken down into suspended solids, dissolved solids and floating solids. Suspended solids are solids which are visible and in suspension in the water and normally removed by processes of sedimentation, flotation or filtration.

Suspended solids can be divided into settleable solids and colloidal solids. Settleable solids are those solids which are of sufficient weight and size to settle in a period of a few hours in primary settling tank. Colloidal solids are the solids that remain uniformly dispersed throughout the water and do not settle in a given period of time unless flocculants are added. Dissolved solids are solids which are in solution (dissolved) in the wastewater. Floating solids are normally made up of oils, greases and fats and can usually be removed by a simple skimming process.

Turbidity, measured with a device called a turbidimeter, generally indicates the quantity of suspended material in the flow stream, particularly at low solids concentrations. Turbidity does not directly correlate with suspended solids concentrations, since colour can interfere with the turbidity measurement; nonetheless, it offers a relative indication of suspended solids level, especially in secondary effluents.

The different types of solids can be divided into organic and inorganic compounds. Organic compounds can be considered as solids that are made from animal or vegetable matter, for example food, wood fibres or beer. Organic compounds are combustible, subject to bacterial decay, and constitute about 70% of the total solids in a municipal wastewater stream. Inorganic compounds are solids that are inert, that is, do not originate from once living substances; some examples are sand, glass and metal particles. Inorganic compounds can generally not be burned and constitute about 30% of the total solids in municipal wastewater.

One commonly used parameter for determining the wastewater characteristics is pH. The pH is a measure of the hydrogen ion concentration of a liquid and is measured on a scale from 0 to 14 with 7.0 considered as neutral. Readings below 7 indicate an acid condition, whereas readings above 7 indicate a basic condition. The pH concentration does have an effect both on biological activity and the control of chemical dosage by chemical precipitation. If the pH is within the range of 6.5 to 8, normal biological activity of the bacteria should not be affected.

The alkalinity of the wastewater is a measure of the capability to resist pH changes. Alkalinity is normally determined as meq/l, but other measures are also used, i.e. mg/l calcium carbonate or ml 0.1 N HCl. To determine the alkalinity, we add acid up to pH 4.5, and measure the acid consumption: meq/l = ml 0.1 N HCL/10 = mg $CaCO_3$/50. Alkalinity is an important parameter for determining chemical dosages by chemical precipitation.

The conductivity of the wastewater indicates the quantity of dissolved material present. All wastewaters have a normal range of conductivity associated with the dissolved solids concentrations in the water supply. A significant increase in the conductivity of a wastewater indicates an abnormal discharge, likely from an industrial source, or from seawater infiltrating the sewer system.

The amount of oxygen used for aerobic biological oxidation of organic solids is determined by BOD testing. BOD stands for biochemical oxygen demand, and is a measure of the amount of oxygen required by bacteria to decompose organic matter under aerobic conditions. BOD_5 is the amount of oxygen required during five days. Normal municipal wastewater contains about 100 to 250 mg/l of BOD_5 (see Table 1.1) with each person contributing about 60 g per day to the waste stream. The BOD-test, which is normally conducted for a period of five days (in Norway and Sweden for seven days, BOD_7 = 1.15 - 1.20 BOD_5) in an incubator at 20°C, can be used to determine the efficiencies of your treatment facility to remove organic matter.

COD (chemical oxygen demand) gives the total amount of organic matter in the wastewater. This is determined by 2 hours of boiling with potassium dichromate. In wastewater that has not been biologically treated, $BOD_5 = 0.35 - 0.60 \cdot COD$. In biologically treated wastewater $BOD_5 = (0.15 - 0.35) \cdot COD$.

TOC (total organic carbon) gives the total amount of carbon in the wastewater. Normally, $TOC = (0.3 - 0.4) \cdot COD$. This parameter is less useful in determining the efficiency of the biological process in a treatment plant.

The nutrients phosphorus and nitrogen may be of great importance to the recipient of the effluent from the treatment plant. The biological processes in water are normally limited by the lack of one of these nutrients. The growth of plants in fresh water and brackish water is usually limited by the lack of phosphorus, and by removing the phosphorus from the wastewater, the biological growth in these water courses will be inhibited. In seawater, nitrogen is normally the limiting factor, and thus, removal of nitrogen from the wastewater can limit algae growth in the sea recipient. The total phosphorus is a measure of the total phosphorus content in the wastewater. Each person contributes with about 2.0 - 4.0 g per day to the wastewater stream, depending on the water hardness and the use of phosphate free detergents. The dissolved phosphorus is measured as orthophosphate, and measuring the content of orthophosphate in the effluent is the best way of controlling a chemical precipitation process.

In wastewater, nitrogen occurs in four basic forms: organic nitrogen, ammonia (both ionised and free ammonia), nitrite, and nitrate. The forms of nitrogen present in a wastewater indicate the level of organic stabilisation. As the organic nitrogen is metabolised, it changes first to ammonia, and then, if conditions are suitable, to nitrite and nitrate. In addition, the biological mass assimilates and, thus, removes some of the nitrogen.

Analysis of nitrogen in wastewater involves several procedures. Several techniques measure ammonia. Total nitrogen is the total amount of the nutrient. Normal contribution of total nitrogen to the wastewater is 12 g per person per day. The organic nitrogen level is determined by Kjeldahl nitrogen analysis. This procedure measures organic nitrogen and ammonia and then subtracts the ammonia value. Nitrite nitrogen is measured directly. The nitrate concentration is determined by a procedure that measures total nitrate and nitrite and then subtracts nitrite.

3 Sampling and Flow Measurement of Municipal Wastewater

To get a good documentation of the plant operation, we need to measure the flow and the mass transport through the different parts of the treatment plant. An accurate flow measurement and an effective sampling program are the cornerstones of a good process monitoring and control program. If the flow measurement is not accurate and the samples taken are not representative, we will know very little about what is happening in the treatment plant. In this chapter we will show how to perform the sampling and flow measurement correctly.

3.1 Sampling in Wastewater Treatment Plants

Grab and Composite Samples

A sample is a part or piece taken from a larger entity and presented as being representative of the whole. It is important that the sample is representative, otherwise the conclusions we draw may be wrong. A grab sample is a discrete sample that is collected manually. A grab sample presents the situation at the actual time the sample is collected. The situation in the stream can be quite different the moment before and the moment after the sampling. Therefore, composite samples are taken to show the situation over time. A composite sample is a single sample prepared by combining a number of grab samples during a specific period. A composite sample, collected either manually or with automatic sampling equipment, provides information on the average characteristics of the sample over the specified period.

Composite samples include two general categories: time composite (fixed volume or simple composite) and flow proportional. To create a time composite sample, the operator collects a fixed-volume sample at specific time intervals and combines them. Such sampling is appropriate for process streams with a composition which is not strongly flow-dependant, such as contents of activated sludge aeration tanks. Time composition samples can be collected either manually or automatically.

A flow proportional composite sample requires that either the volumes of the grab samples, or the sampling frequency, are varied to weigh the final sample in proportion to the various flow rates measured during the composite sampling period. Such a composite contains wastewater reasonably equivalent to the flow-weighted average composition of the process stream during the composite sampling period. In effect, the total composite volume collected is proportional to the total flow volume passing the sampling point. Accordingly, flow proportional composite sampling demands accurate measurements of the varying flow rates of the process stream.

A flow-proportioned composite is a more representative sample than a fixed time volume composite for a varying process stream. A variable volume technique for forming the flow proportional composite is the most practical with manual sampling. In this technique, samples are taken at equal time intervals, but the sample volume, named an aliquot, varies directly with the flow rate at the instant of sampling.

Sample Collection

Reliable information depends on correct measurements. At wastewater treatment plants, the many possibilities for sampling and measurements are often hardly taken into account. The wastewater or sludge is often transported in closed pipes from one stage to another, or in slowly flowing channels with high solids concentration near the bottom. The access to a good sampling point must be easy and the flow turbulent. Sampling points for which the possibility of biological fouling from the wall or solids from the bottom is present, should not be used. Best suited sampling points are places where the wastewater passes weirs or flumes. The best sampling points in treatment plants with several parallels are channels where wastewater from the parallels is mixed well. If such a place is not available, samples from all the parallels should be taken and mixed. In special cases, separate samples from each parallel are preferred.

When selecting the appropriate sample locations, it is important to be aware of return flows, e.g.:

- Return sludge from the secondary settling tank
- Supernatant from sludge thickening
- Filtrate or centrate from dewatering equipment

Especially influent samples should not be contaminated by return flows. Raw wastewater consists of many matters, also large solids, and the influent sample location should therefore be downstream screens, and also occasionally downstream grit chambers to prevent clogging of the automatic sample collector or to ensure representative samples. At the same time, the sampling point must be upstream return flows, as mentioned.

To ensure reliable documentation from the treatment plant operation, systematic sampling is necessary. The documentation is necessary both for your own plant process control and for documentation demanded by the regulatory authorities. Table 3.1 serves as an excellent starting point for developing the sampling and analysis program. It includes only suggestions to be adjusted as necessary based on each plant's operational and regulatory requirements.

Table 3.1 Suggested sample locations, typical analysis performed, sampling frequencies, and type of sample required for wastewater treatment unit processes.

Sampling location	**Analysis**[1]	**Sample**		
		Purpose[2]	**Frequency**[3]	**Type**[4]
Plant influent	BOD	PP	W/M	C
	TSS	PP	W/K	C
	TP	PP	W/M	C
	TN	PP	W/M	C
	NH_4-N	PP	W/M	C
	NO_2-N	PP	W/M	C
	NO_3-N	PP	W/M	C
	pH	PC	D	G
	Heavy metals[5]	PP	M	C

***Table 3.1** Suggested sample locations, typical analysis performed, sampling frequencies, and type of sample required for wastewater treatment unit processes (continued).*

Sampling location	**Analysis**[1]	**Sample**		
		Purpose[2]	**Frequency**[3]	**Type**[4]
Primary effluent	BOD	PP	W/M	C
	TSS	PP	W/M	C
	TP	PP	W/M	C
Activated sludge, mixed liquor	DO	PC	D	G
	Temperature	PC	D	C
	TSS/SV	PC	D	C
	VSS	PC	D	C
	NO_3-N	PC	W	G
Return sludge	TSS/SV	PC	D/W/M	C
Chemical precipitation influent	Alkalinity	PC	D	G
	pH	PC	Continuous	G
	Ortho-P	PC	D	C
Plant effluent	BOD	PP	D/W/M	C
	TSS	PP	D/W/M	C
	TP	PP	D/W/M	C
	Ortho-P	PC	D	C
	TN	PP	W/M	C
	NH_4-N	PP	W/M	C
	NO_2-N	PP	W/M	C
	NO_3-N	PP	W/M	C
	Heavy metals[5]	PP	M	C
Sludge thickener, feed sludge	pH	PC	D	G
	TS	PC	D/W/M	C
	VS	PC	D/W/M	C
Sludge thickener, thickened sludge	TS	PC	D/W/M	C
	VS	PC	D/W/M	C
Sludge thickener, supernatant	TSS	PC	D/W/M	C
	BOD	PC	W/M	C
	COD	PC	W/M	C
Sludge dewatering, dewatered sludge	TS	PP	D/W/M	G
	Heavy metals[5]	PP	W/M	C

Table 3.1 Suggested sample locations, typical analysis performed, sampling frequencies, and type of sample required for wastewater treatment unit processes (continued).

Sampling location	**Analysis**[1]	**Sample**		
		Purpose[2]	**Frequency**[3]	**Type**[4]
Sludge dewatering, sludge liquor	TSS	PC	D	C
	BOD	PP	W/M	C
	COD	PP	W/M	C
	TN	PP	W/M	C
	NH_4-N	PP	W/M	C

[1] TSS = Total suspended solids
VSS = Volatile suspended solids
TP = Total phosphorus
TN = Total nitrogen
DO = Dissolved oxygen
SV = Sludge volume
TS = Total solids
VS = Volatile solids

[2] PC = Process control
PP = Plant performance

[3] D = Daily
W = Weekly
M = Monthly
W/M, D/W/M: depending on the size of the plant

[4] C = Composite sample
G = Grab sample

[5] Pb, Hg, Cd, Ni, Cr, Cu, Zn.

Automatic sampling should as far as possible be used by composite sampling. Flow proportional samples are preferred. Samples are collected in a container. The container must be shaken well before collecting samples for analysis.

Small time intervals between grab samples are important in a composite sample. The interval between two grab samples should not exceed 5-10 minutes. The wastewater should not be sieved, settled or in other ways altered by the sampling equipment.

Sample Handling

Regular cleaning of the samplers and the sample lines leading to the samplers is necessary to prevent biological fouling and alteration of the sample. Samplers should be equipped with built-in refrigerators, or the sample container should be stored in a fridge with a hole for the sample line. The temperature in the fridge should be monitored regularly to ensure the proper temperature for sample storage. Samplers should be kept inside buildings. Samplers that are located outside during freezing weather may require heating to prevent freezing of the sample and sample lines. In general, the samples should be collected and transported in plastic devices, and certainly not in metal devices.

Following collection, samples must be handled properly to ensure that they remain unchanged. This requires proper conservation before analysis, proper labelling, and proper transportation to the testing site. Necessary information should be labelled on the bottle or marked on the bottle with a waterproof marker. The information provided on each bottle should include:

- Sample source
- Sample location
- Sample date and time
- Preservative(s) added

- Required analyses
- Signature of the person collecting the sample

Table 3.2 lists the appropriate preservation techniques for each analytic method. Composite samples should be refrigerated at 4°C until they are tested at the laboratory. Immediate analysis is best, when possible.

Table 3.2 Required containers, preservation techniques, and holding times.

Parameter	**Volume required (ml)**	**Preservation**	**Maximum holding time**
Alkalinity	200	Cool, 4°C	14 days
Ammonia (NH_4-N)	400	Cool, 4°C, H_2SO_4 to pH<2	28 days
BOD	1000	Cool, 4°C	48 hours
COD	100	Cool, 4°C, H_2SO_4 to pH<2	28 days
Mercury	500	HNO_3 to pH<2	28 days
Metals, except mercury	200	HNO_3 to pH<2	6 months
Nitrate (NO_3-N)	100	Cool, 4°C	48 hours
Nitrite (NO_2-N)	100	Cool, 4°C	48 hours
Ortho-P	50	Filter immediately, cool, 4°C	48 hours
pH	25	None required	Analyse immediately
TN	100	Cool, 4°C, H_2SO_4 to pH<2	28 days
TSS	250	Cool, 4°C (no acids or freezing)	48 hours
TP	50	Cool, 4°C, H_2SO_4 to pH<2	28 days
Turbidity	100	Cool, 4°C	48 hours
VSS	250	Cool, 4°C (no acids or freezing)	48 hours

3.2 Flow Measurement

Accurate flow metering is of great importance both for plant process control, and control of plant performance and pollutant discharges. We will here describe the most common flow measurement devices and how to calibrate and maintain them.

The flow through a wastewater treatment plant can either be measured in an open channel or in a closed pipe. The flow should either be measured at the plant inlet before any return streams, or at the outlet. The latter is recommended in order to avoid interference from gross solids in the wastewater.

In addition, it should be possible to measure the return sludge flow, the surplus sludge flow and the thickened sludge flow.

Open Channel Flow Meters

Flow in an open channel is normally measured by using a calibrated restriction inserted into the channel, which changes the level of the liquid in or near the restriction. By selecting the shape and dimensions of the restricting structure, the rate of flow through or over the restriction will be related to the liquid level by a known function. There are two categories of restricting structures: weirs and flumes.

A weir is essentially a dam built across an open channel over which the water flows through some type of an opening or notch. The most common types of notches are the sharp-crested rectangular weir, the trapezoidal (or Cipoletti) weir and the triangular (or V-notch) weir.

A flume is a specially shaped open channel flow section providing a change in the channel area and/or slope which results in an increased velocity and change in the level of the water flowing through the flume. The most commonly used types of flumes are the Parshall flume, the Pawler-Bowlus flume and the Khafagi (Venturi) flume.

The installation instructions for the weir or flume given in the ISO-standards are not always followed. It is important to check whether the regulations have been followed in your plant. It is of great importance that e.g. the specification of a disturbance free approach section to the weir or flume is followed, and that a flume is directed the right way. It is also of great importance that the weir is cleaned regularly. Sand and clay sedimentation in the weir will cause errors in the flow measurement, as will biological or chemical growth on the walls.

In open channels, the water level is usually measured, and the flow determined from the level-flow rate relationship for the actual weir or flume. The level can be measured in many different ways, e.g. by floats, dipping probes, changes in electrical circuits, pressure meters, ultrasonic meters or bubblers.

Calibration of Open Channel Flow Meters

To aid in the zero adjustment of the flow meter, it is strongly recommended that a flow meter installation in an open channel includes a staff gauge A staff gauge is simply a fixed scale, on which the level of water in the open channel can be read. The staff gauge must be solidly and accurately attached to the channel or stilling well. Enamelled iron or aluminium gauges are preferred. Installation of the gauge must be done without water running through the channel or over the weir, to find the exact zero point. When filling the channel, you can control the accuracy of your flow meter by reading the water level on the gauge, find the corresponding flow level from the level-flow rate relationship (this must be the correct relationship for your weir or flume).

The best way to control your flow meter equipment is to simulate a water level in your channel, and control that both the visual, recording and counting devices give the correct water flow by different levels in the channel. By controlling the counting devices, the level must be kept stable for some time, and the flow must be measured over a time interval, e.g. 5 minutes. If the simulated water flow is 120 m^3/h, then the counting devices should show

$$\frac{120 \bullet 5}{60} = 10 \text{ m}^3$$

after five minutes. Another possibility of controlling the counting devices if you are measuring the inlet flow of the plant is to fill up an empty settling tank or other tank with exact volume determined. The difference in volume measured by the counting devices before and after the filling should correspond to the volume of the settlement tank. Remember to stop return flows during the measurement!

Closed Pipe Flow Meters

Flow in closed pipes is normally measured either by pressure changes, by propellers (turbine meters), by orifice plates or nozzles, or by electromagnetic, ultrasonic or radioactive measurement. Most of them are described in ISO standards. It is of great importance that the specification of a disturbance free approach section to the meter is fulfilled. The flow is determined by measuring the pressure or velocity of the water, by using a pressure-flow rate relationship, or a velocity-flow rate relationship for the actual closed pipe.

Calibration of Closed Pipe Flow Meters

It is more difficult to calibrate flow meters in closed pipes. If you are able to determine the flow, e.g. by measuring the time needed to fill a known volume, it is possible to calibrate the flow meter. Otherwise, one must rely on that the meter is accurate, which they usually are.

4 Preliminary Treatment

4.1 Introduction

Preliminary treatment of wastewater means removal of gross solids, sand, grit and/or grease and oil from wastewater. Preliminary treatment includes screening, grit removal, odour control (where appropriate), and flow measurement. The removal of debris in the screening area and the removal of sand, rocks, gravel and other inorganics in the grit removal system protect downstream treatment processes.

Coarse screens are classified as either bar racks (clear spacing between 50 and 100 mm) and bar screens (clear spacing between 6 and 50 mm). In the last years step screens and rotating screens have been common in the Western Europe instead of bar screens. Fine screens with clear spacing of less than 6 mm are occasionally used in wastewater treatment plants instead of other primary treatment processes. In this manual, we will concentrate on bar screens.

Other types of preliminary treatment used in small treatment plants are disintegrators, macerators or comminutors that grind or chop solids that could interfere with downstream treatment processes. These devices, usually preceded by a bar screen, return the chopped solids to the wastewater flow for removal by subsequent treatment units. These devices are less common, and will not be discussed in this manual.

Grit chambers are square, rectangular or circular settling chambers, or cyclone degritters. Grit chambers are of two basic types: aerated tanks and flow-through sedimentation tanks. Either manual cleaning or mechanical methods remove grit from the tanks. Grit disposal methods include landfills, landspreading (if washed), or incineration with the sludge.

4.2 Screening

4.2.1 Description of Process

Coarse screening removes large solids and trash that could interfere with the operation of a treatment plant. As the plant's first treatment unit, coarse screens protect plant equipment against damage such as clogging of pipes, pumps and grit aeration diffusers.

Normally, bar screens consist of a vertical arrangement of equally spaced parallel bars designed to trap coarse debris. At appropriate intervals, the debris accumulated on the screens is removed for disposal, either by manual or motor-operated rakes. This is showed in Figure 4.1.

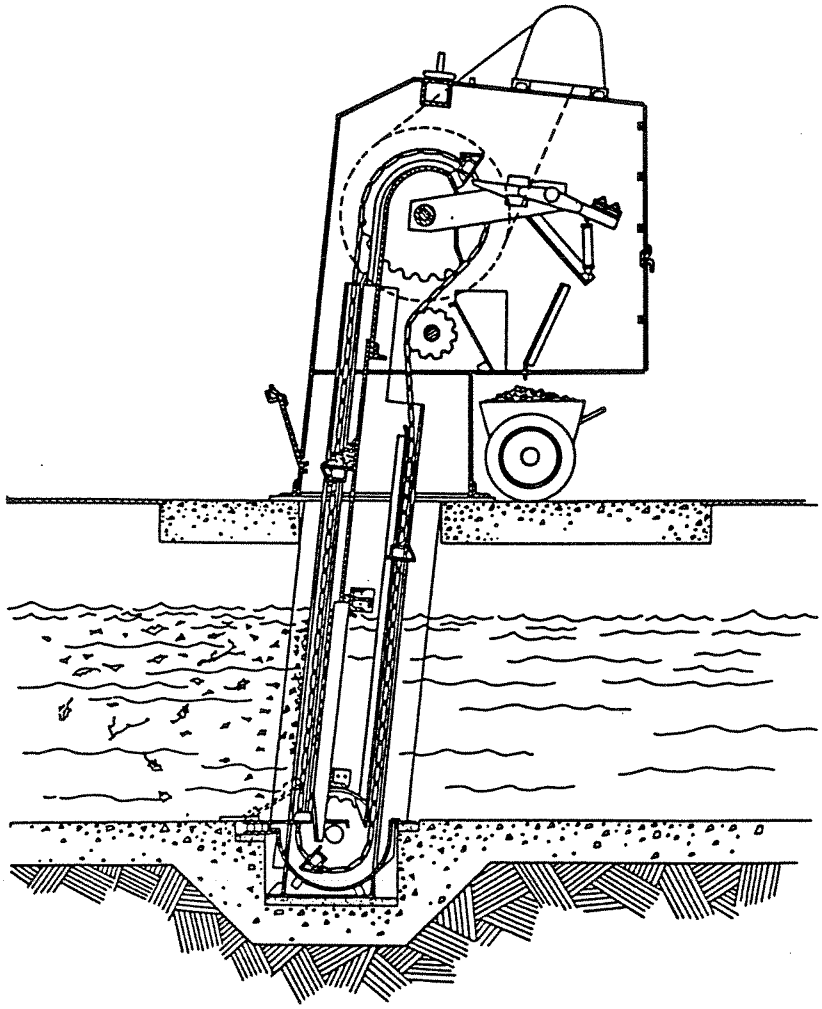


Figure 4.1 Mechanically cleaned bar screen.

Most plants include two screening channels, especially when one or two channels are equipped with mechanically cleaned bar screens. Thus, if one unit malfunctions, the other unit and channel may remain in service during repairs. Stop gates for the channels allow diversion of flow to clean, maintain, and repair the screening equipment.

Usually mechanically cleaned screens are operated by a timer to start and stop the rake. In addition, pressure-operated switches may control that the head loss through the bar screen does not exceed a set value, due to clogging of the bars.

The screenings are transported to a container, either directly by a screw transporter, or by a compaction press that dewater the screenings.

4.2.2 Process Control

The head loss through the bar screen should not exceed 50-150 mm. If the head loss is too large, the cleaning frequencies should be increased. This may happen during rainstorms, when a combined collection system is connected.

The operator should several times during a shift visually check the equipment to ensure that it functions properly. Use of local manual controls allows immediate observation of the operation of the mechanically cleaned bar screen or other equipment, even though it may be inactive when the inspection begins. While the equipment is running, check for unusual noises, scraping of the screen, jerking of the drive mechanisms, need to dispose of screenings, and lubrication of the chain or drive mechanism.

Avoid any overflows in screening storage areas to prevent possible decay of organic matter and resultant offensive odours. Screenings are a major source of odours. Cleanliness also is a health consideration because wastewater-soaked debris harbours pathogenic organisms. Good management practice dictates daily, or even more frequent, disposal of screenings by either incineration or landfill. The weight or volume of disposed screenings should be registered.

4.2.3 Troubleshooting

Problems related to the following categories may occur:

- Unusual operational conditions (sudden loads of debris that clog or physically jam the screening equipment),
- Equipment breakdown (component failure), and
- Control failure.

Mechanically cleaned screens without head loss control systems sometimes receive sudden large loads of debris that jam their raking mechanisms.

Table 4.1 provides a troubleshooting guide for screening equipment which enables the operator to recognise problems and quickly determine their solutions.

Table 4.1 Troubleshooting guide for mechanical bar screens.

Problems	Probable causes	Solutions
Obnoxious odours, flies and other insects	• Accumulation of rags and debris	• Increase frequency of removal and disposal to an approved disposal facility
Excessive grit in bar screen chamber	• Flow velocity too low	• Remove bottom irregularity, or reslope the bottom. Increase flow velocity in a chamber or flush regularly with a hose
Excessive screen clogging	• Unusual amount of debris in wastewater. Check industrial wastes	• Use a coarser screen or identify source of waste causing the problem so its discharge into the system can be stopped.
Screenings downstream screens	• Overflow, damaged bars	• Repair or replace screen

4.3 Grit Removal

4.3.1 Description of Process

Wastewater grit materials include sand, cinders, rocks, coffee grounds, cigarette filter tips, and other large-sized, relatively non-putrescible organic and inorganic substances.

Grit removal, which is an essential element of preliminary treatment, protects equipment by

- Reducing clogging in pipes
- Protecting moving mechanical equipment and pumps from abrasion and accompanying abnormal wear
- Preventing accumulations of materials in aeration tanks and sludge digesters that result in loss of usable volume

A velocity of 0.3 m/s is used in most grit removal systems to separate grits from the organics. Velocities that are too low allow organics to contaminate the grit; excessive velocities carry grit to the downstream processes.

Three types of grit settling chambers - hand-cleaned, mechanically cleaned, and aerated - and cyclone degritting are briefly described below.

Hand-Cleaned Chambers

Hand-cleaned grit chambers, used only in the smallest plants, consist of at least two elongated channels with control devices at their outlets to regulate the wastewater velocity at about 0.3 m/s. Some have hopper-shaped bottoms for grit storage and all have floor drains to empty the the tanks for manual grit removal by shovelling.

Mechanically Cleaned Chambers

Mechanically cleaned grit chambers are usually rectangular tanks. Most have velocity control devices (Parshall flume, parabolic flume, or proportional weir) and a chain-and-flight mechanism (Figure 4.2) or an air lift pump to move the grit to a sump.

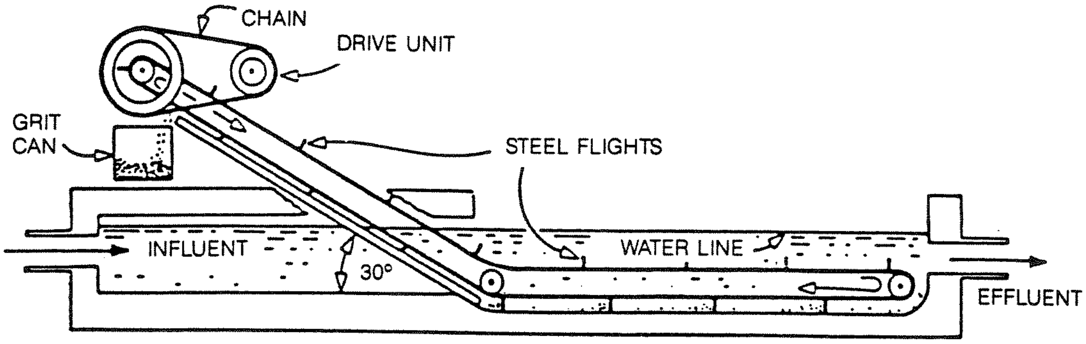


Figure 4.2 Chain-and-flight grit collector section.

Typically, a bucket elevator or inclined screw collector removes the grit from the sump and drops it into a "classifier" or washer that removes most of the organic matter collected with the grit. Also, pumps may remove grit. In larger plants with wider channels, chain-and-flight or screw collectors move the grit to a central sump.

The classifier may be either a reciprocating rake device or screw collector. Either type may include water sprays for grit washing. Both the reciprocating rake and the screw collector are inclined upward at an optimum angle for draining water and organic matter. The grit, washed or unwashed, falls into a wheelbarrow, container, or elevated storage hopper for disposal.

The longitudinal collectors should operate at a low speed and the bucket elevator should move fast enough to accommodate the grit as it collects. Otherwise the collectors will pack the grit and damage will result. The period of collector operation depends on the rate of grit accumulation. Although automatic operation reduces equipment wear, manual control may nonetheless be necessary during periods of surface runoff and other periods with unexpectedly high grit loads.

During operation, frequent inspection of the buckets is needed to check the volume of grit collected. If the buckets are filled, the elevator speed should be increased or an additional grit channel should be placed into service. If the grit volume is decreasing, the opposite actions may be appropriate.

The removal of grit by air lift pumps offers the advantage of eliminating the need for a bucket elevator. However, blockage might occur in the pump suction line or in the intake area of the sump. If this occurs, removal of the blockage may require reversing the pumping direction to blow air into the sump.

Aerated Grit Chambers.

Aerated grit chambers (Figure 4.3) provide a period of wastewater detention to trap grit through air-inducted rotation of the wastewater at approximately 0.3 m/s. Compressed air, usually from the plant's process air supply, enters the grit chamber through diffusers at a controlled rate to induce the correct wastewater velocity near the floor for settling the grit.

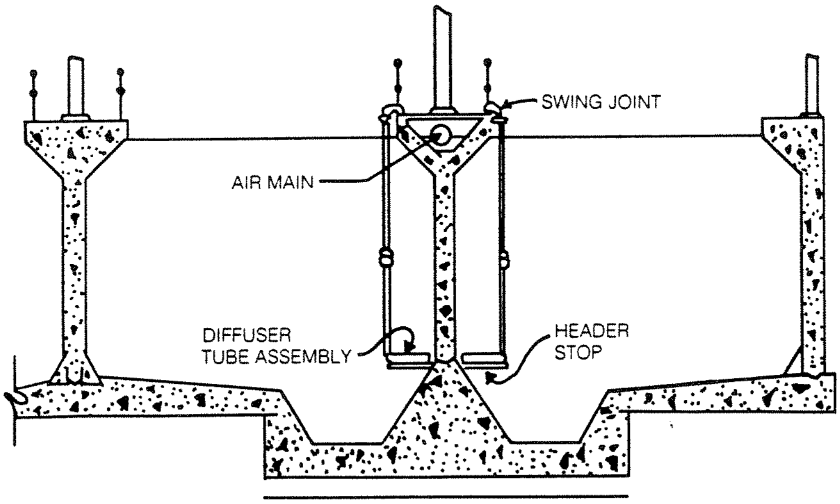


Figure 4.3 Typical cross section of an aerated grit chamber.

If diffusers are used as aerators, the grit should not be allowed to accumulate above the diffusers. Such grit accumulation complicates diffuser inspection and could possibly result in diffuser damage.

If short-circuiting of the aerated grit chambers occurs, it sometimes may be reduced by installing submerged transverse baffles or longitudinal baffles adjacent to the diffusers or along the wall opposite the diffusers. Table 4.2 presents typical performance and loading data for aerated grit chambers.

Table 4.2 Typical design and operation data for aerated grit chambers.

Parameter	Typical values
Transverse velocity at surface	0.6 to 0.8 m/s
Depth: width ratio	1.5:1 to 2:1
Air supply	4.6 to 7.7 l/m length ·s 0.3 to 0.4 m^3/m^3 wastewater
Detention time	3 to 5 min (peak)
Quantity of grit	7.5 to 75 ml/m^3
Quantity of scum (skimmings)	7.5 to 45 ml/m^3

Cyclone Degritters

Cyclone degritters use centrifugal force in a cone-shaped unit to separate grit from the wastewater. At a controlled rate, a pump discharges a slurry of grit and organic into the degritter. The slurry enters the degritter tangentially near its upper perimeter as illustrated in Figure 4.4. This feed velocity creates a vortex that produces a grit slurry at the lower, narrower opening and a larger volume of slurry containing mostly volatile material at the upper port. The grit streams fall into a rake screen washer, and the degritted flow leaves the cyclone through the opening near the top of the unit, moving downstream for further treatment. In some systems, a mechanical mixer induces the centrifugal effect.

The cyclone degritting process includes a pump as an integral part of the process because the cyclone has no moving parts and depends on a steady supply of liquid. The volume of pumped slurry and the resultant pressure at the degritter are critical requirements that are specified by the cyclone manufacturers. The temperature, solids concentration, and other characteristics of the slurry may require changes in the sizes of the upper and lower orifices after installation and some initial operating experience. In some designs, these orifices are manually adjustable.

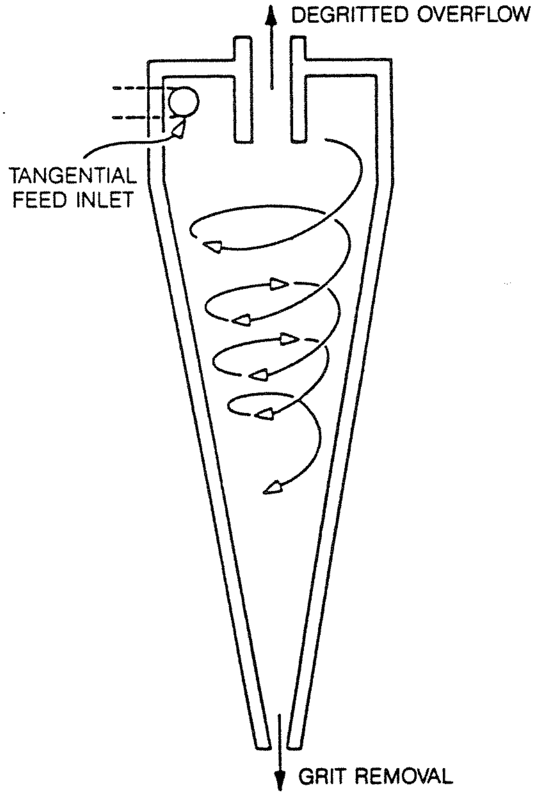


Figure 4.4 Cyclone degritter.

The pressures at the degritter and the pump inlet need to be checked regularly. Operation should be routine, provided the proper volume of slurry is delivered to the degritter at the proper pressure. Thus, the equipment needs regular, frequent observation. Excessive vibration of the degritter may be caused by an obstruction in the lower port, and excessive flow at the lower end may result from a damaged liner or an obstruction of the upper port.

Grit Washing

Grit washers effectively remove organics from the grit. Screw and rake grit washers, proved to be reliable, usually produce a product low in organics. To ensure a low volatile content, however, ample dilution water is required. Pumps normally provide sufficient dilution water, but bucket elevators may not, especially during periods of peak grit capture. Consequently, they may require supplementary liquid.

Grit washing occurs at the low end of the classifier in a well that has an adjustable weir to govern the depth of liquid above the settled grit. Most washers are equipped with spray nozzles to remove stray putrescribles. For optimum washing, some types of cyclone degritters use an adjustable lower orifice to change the volume of liquid discharges with the grit.

4.3.2 Process Control

Grit, if not removed early in the treatment process, will inevitably cause problems later. Therefore, a high level of early grit removal is a sound objective. However, the operator must consider grit removal equipment wear versus increased removal efficiency and resultant downstream equipment longevity. Aerated chambers and cyclone degritter systems are easy on equipment and hand-cleaned channels avoid equipment wear;

increased wear for chain-and-flight collectors and the bucket elevator will result from a high level of grit removal.

The volatile content of the captured grit is another control consideration. Increased removal of grit may cause more organic material to be removed, an undesirable result. Proper operation of grit washers can, however, reduce this potential problem. A plant without grit washers may encounter adverse public reactions, particularly if the unwashed, odorous grit must be trucked along city streets. When an unwashed grit must be transported, the operator may have to choose between a less efficient collection system and odour control during transportation. Furthermore, as landfill regulations continually become more strict, grit with a high organic content may be prohibited at the landfill site.

4.3.3 Troubleshooting

Because grit removal equipment has many moving parts, many possible types of malfunctions and breakdowns can adversely affect the operation and efficiency of the system. The operator needs to learn the equipment and how to assess problems that will occur.

Table 4.3 presents a troubleshooting guide that will enable the operator to quickly identify problems and determine their solutions.

Table 4.3 Troubleshooting guide for grit removal systems.

Problems	Probable causes	Solutions
Grit packed on collectors	• Collector operating at excessive speeds.	• Reduce collector speed Increase speed of grit removal from collector
	• Bucket elevator or removal equipment operating at slow speeds	• Removal system speed
Rotten egg odour in grit chamber	• Hydrogen sulphide formation	• Wash chamber and dose with hypochlorite
Accumulated grit in chamber	• Submerged debris	• Wash chamber daily. Remove debris
	• Flow velocity too low or broken chain or flight	• Repair equipment
Corrosion of metal and concrete	• Inadequate ventilation	• Increase ventilation and perform annual repair and repainting
Removed grit is grey in colour, smells, and feels greasy	• Inadequate air flow rate	• Increase velocity in grit chamber. (0.3 m/s usually optimum unless operating strategy calls for lower velocity with washing)
Surface turbulence in aerated grit chamber is reduced	• Diffusers covered by rags or grit	• Clean diffusers and correct screens or other pretreatment steps to prevent
Low recovery rate of grit	• Bottom scour at excessive speeds	• Maintain velocity near 0.3 m/s.
	• Too much aeration	• Reduce aeration Increase retention time by using more units or reducing flow to unit
Overflowing grit chamber	• Pump surge problem	• Adjust pump controls or control infiltration and inflow
Septic waste with grease and gas bubbles rising in grit chamber	• Sludge on bottom of chamber	• Wash chamber daily.

5 Primary Treatment

5.1 Introduction

Primary treatment, normally associated with sedimentation, occasionally uses fine screens instead of sedimentation. Fine screens remove less suspended solids than does sedimentation, and will not be discussed further in this manual. Primary sedimentation separates the readily settleable and floatable solids from the wastewater for subsequent wastewater treatment. Other benefits of primary sedimentation include equalisation of sidestream flows and removal of the BOD associated with settleable solids.

To improve settling performance, pre-aeration or mechanical flocculation sometimes precedes the sedimentation basins. With flocculation, chemicals are often added to the wastewater influent flow to enhance floc formation. This is described in Chapter 8.

Many treatment plants use primary sedimentation tanks for thickening primary sludge as well as solids separation from wastewater. In these cases, the primary tanks are sized to accommodate both sedimentation and thickening.

5.2 Description of Process

Removal

Suspended particles may be classified as granular or flocculent. Granular particles (sand and silt) settle at a constant velocity, with no change in size, shape, or weight. Ideally, most such particles are removed in upstream grit chambers. Flocculent particles (organic matter, flocs formed by coagulants, or biological growths) tend to cluster during settling, with attendant changes in size, shape, and relative density. The cluster ordinarily settle more rapidly than their individual particles.

Settleable solids, including portions of the granular and flocculent material, settle under quiescent conditions, within a reasonable time. Non-settleable solids, finely divided and colloidal materials, are too fine to settle within usual settling times.

Removal of floatables, including grease and scum, helps protect downstream plant unit processes, reduces the pollutants discharged, and improves the aesthetics of the plant effluent. Under quiescent conditions, part of the grease and scum settles with the sludge, while the reminder floats to the surface for removal by suitable skimming devices.

Primary settling tanks (also referred to as sedimentation tanks or clarifiers) can be rectangular, circular, or square. In the rectangular type, wastewater flows from one end of the tank to the other and scrapers move the settled sludge to the inlet end. In the circular or square type, the wastewater usually enters at the center and flows radially to the outside edge (Figure 5.2), with the settled sludge scraped or otherwise transported to the centre. In some circular tank designs (called peripheral feed clarifiers), wastewater enters at the outer edge and flows inward. Imhoff tanks perform the dual function of settling and anaerobic digestion.

Sludge collection entails moving settled sludge to a point in the sedimentation tank where it is drawn off. Primary sedimentation tanks, except for those in a few small plants relying on manual sludge removal, use mechanical sludge collection. One of the following three types of equipment may be used:

- Flight and chain
- Travelling bridge with screens
- Rotating sludge scrapers

Usually pumping, but sometimes gravity flow, removes sludge from the sedimentation tanks. For gravity removal, sludge flows from the collection point to a separate vault with a lower liquid surface level than the primary sedimentation tank.

Rectangular Tanks

A rectangular tank, either a single unit of one of several adjacent units, has a length usually several times its width (Figure 5.1). Either wood flights mounted on parallel strands of steel conveyor chain (Figure 5.2) or a single bottom scraper mounted on a bridge moving on rails fastened to the tank walls (Figure 5.3) moves settled sludge to a hopper at one end of the tank, usually the inlet end. Other scraper equipment in use is scraper wagon pulled by wires or hydraulic scraper systems.

Chains and flights, guided by submerged sprockets, shafts, and bearings, are driven by a motor through a speed reducer. The flights are usually arranged to remove floating surface material from the tank on their return path. Plastic chain, flights, and sprockets are now often being used in new installations and in retrofits of existing tanks. In some tanks, flat water sprays, declined toward the liquid surface, move the scum and grease to the skimming device.

A travelling bridge, towed by a cable or powered by a traction drive, skims grease and scum with its surface scraper and moves sludge to the sump with its lower scraper blade. Normally, as the bridge travels with the flow, the sludge scraper is up while the grease skimmer is down. When the bridge is moving opposite to the flow, the sludge scraper is down to move the sludge to the hopper while the skimmer blade is raised.

The settled solids may collect in a single hopper, multiple hoppers, or a transverse trench with a hopper at one end. The transverse trench may be equipped with a flight-and-chain collector or a screw conveyor to carry the sludge to the hopper.

A slotted pipe, rotating helical wiper, or other skimming device removes floating material. It is then pumped or conveyed to another process, such as a thickener or a holding basin.

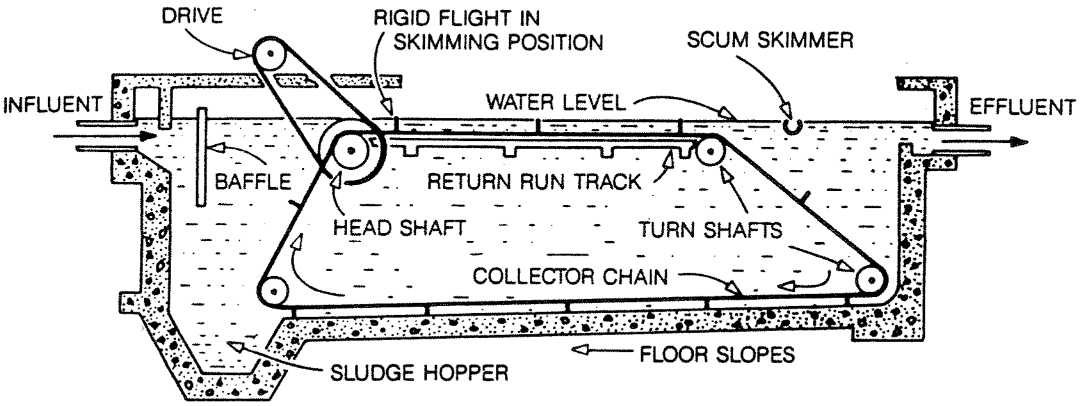


Figure 5.1 Section of a single rectangular tank for primary sedimentation.

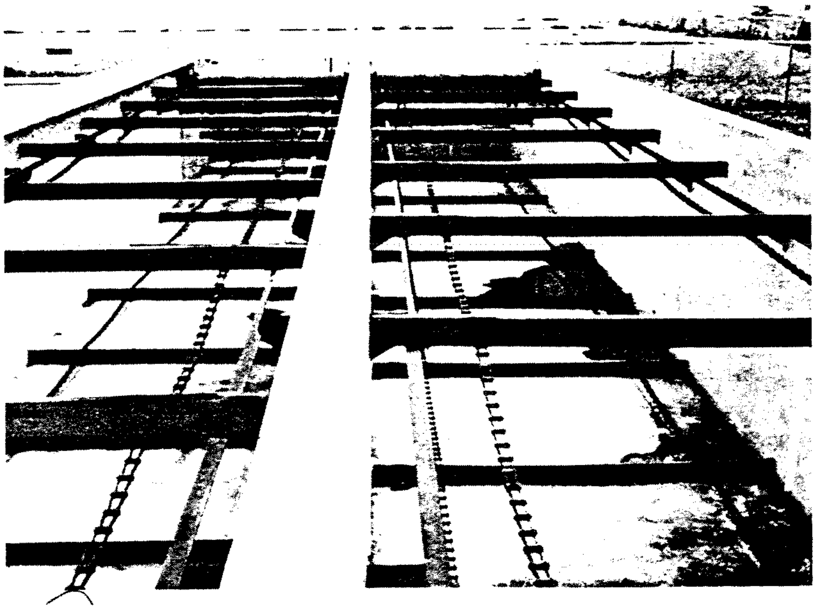


Figure 5.2 Rectangular primary sedimentation tank with wood flights mounted on parallel chains.

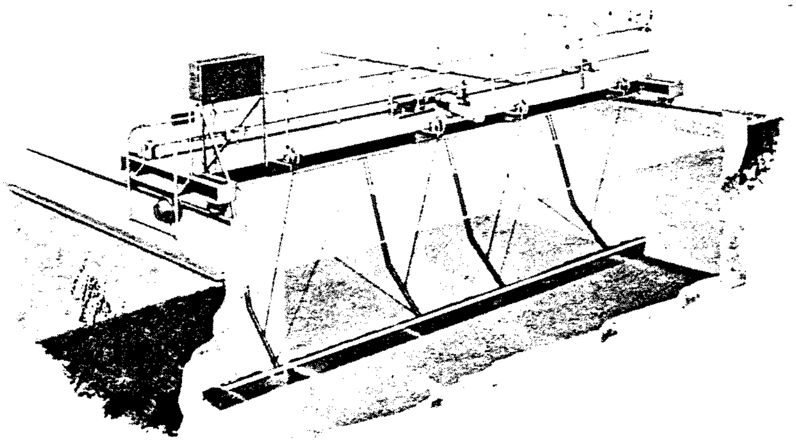


Figure 5.3 Rectangular primary sedimentation tank with travelling bridge sludge collector.

Circular Tanks

In the most common type of circular tank, a centre feed well introduces the flow in a manner that dissipates inlet velocities and distributes the flow evenly around the tank. (Figure 5.4). The wastewater moves radially from the feed well to the weir and overflow at the perimeter of the tank. Settled solids are raked to a hopper near the tank centre by arms attached to a drive unit at the centre of the tank or by a traction unit operating on the tank wall. A surface blade attached to the sludge collector moves floating material to a scum hopper.

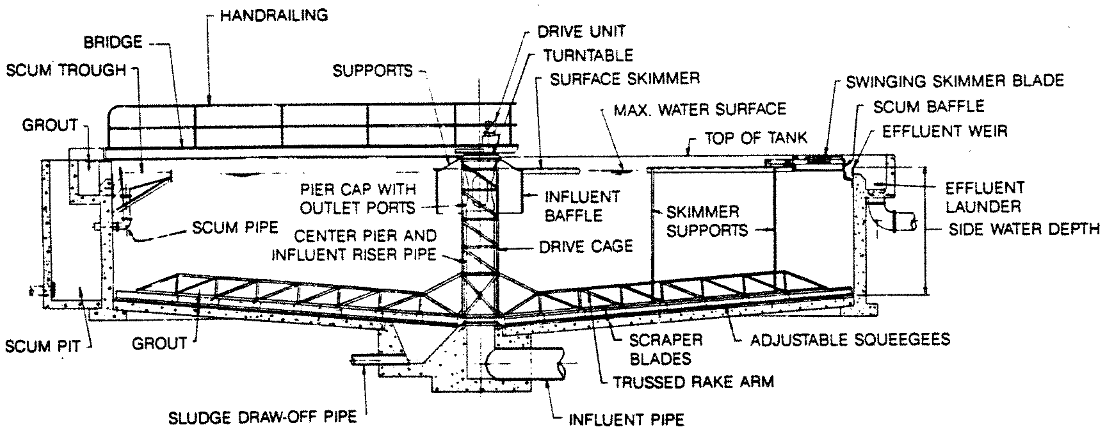


Figure 5.4 Section of a circular sedimentation tank.

Square Tanks

These tanks remove settleable solids and floatables in a manner similar to that of circular tanks. Corner sweeps are added to the scum and sludge collectors arms to remove sludge and scum from the corners.

Imhoff Tanks

These tanks, which are seldom used in modern treatment plants, remove settleable solids from a slot in the bottom of the upper chamber and digest these solids in a lower compartment of the same unit.

5.3 Process Control

The efficiency of a settling tank depends largely on hydraulic loading parameters, namely surface loading rate and detention time, tank configuration, wastewater characteristics, particle characteristics, temperature, and industrial wastewater contributions. Each of the above variables is discussed below and the operational considerations affecting settling efficiency are presented in the subsequent section on process control.

For any influent flow, tank area governs the surface loading rate and tank volume determines the detention time. Although the rate of removal of granular particles settling at uniform velocities depends almost entirely on the tank surface area, the rate of removal of flocculant particles settling at variable velocities depends on the tank volume, including depths as well as surface area. The detention time should be sufficient to allow nearly

complete removal of settleable solids. A longer detention time would not improve removal and might actually impair removal by allowing the wastewater to become septic.
Because the dimensions of settling tanks are fixed, the overflow rate and detention time will vary with flow, resulting in variable removal efficiencies. Consequently, the number of units required for sedimentation may vary if the flow changes significantly.

Inlet and outlet structures, including appropriate baffles, play an important role in sedimentation. Inlets reduce entrance velocity and distribute flow uniformly through the cross section of the basin by use of suitable openings, baffles, or other means. Proper inlet design also allows for an even distribution of flow between multiple tanks. Outlets for removal of clarified wastewater are usually multiple weirs. Baffles normally precede outlet weirs to prevent loss of floating solids. Proper outlet conditions will allow uniform flow withdrawal and avoid localised high-velocity areas.

In Table 5.1 we have summarised the most important information to be collected for process control of primary sedimentation.

Table 5.1 Process control of primary sedimentation.

Item	**Observations**	**Measurements and analyses**	**Calculated values**
Influent	• Appearance	• Settleable solids • Suspended solids	
Settling tank	• Hydraulic conditions • Floatable solids • Sludge blanket		• Detention time • Surface loading rate
Effluent	• Appearance	• Settleable solids • Suspended solids	• Removal Rate

5.3.1 Influent

Appearance

Changes in colour, odour and wastewater quality should be observed on a regular basis.

Settleable Solids

Settleable solids are measured in a 1-liter Imhoff cone.

This parameter gives information about expected removal of settleable solids in the settling tank.

Suspended Solids

Influent suspended solids analysis is needed to find the suspended solids removal rate of the settling tank.

5.3.2 Settling Tank

Hydraulic Conditions

Ensuring equal flow distribution among multiple tanks and uniform flow distribution across the width of each rectangular tank (or round the perimeter of circular tanks) are important hydraulic objectives.

Floatable Solids

Some floatable solids will always occur. The volume depends on the effectiveness of pretreatment and preliminary treatment, and also on water temperature. Odours may occur from floatables, and the removal frequency depends on the amount of floatables in settling tanks.

Sludge Blanket

Sludge scraping and pumping routines should be monitored by sludge blanket observation. The sludge blanket in flat-bottomed tanks should be less than 10 cm. The upper layer of too thick sludge blankets in settling tanks and hoppers may be carried away to the effluent. Thick sludge blanket indicates high sludge age in the tank, and this can easily cause anaerobic sludge conditions with odours and extreme volumes of floatables as a result.

The detention time represents the time required for a unit volume of the wastewater to pass entirely through the tank at a given rate of flow. The formula for detention is:

$D = \frac{V}{Q}$, where

D = Detention time (hours)
V = Volume of the tank (m^3)
Q = Wastewater flow (m^3/h)

Surface Loading Rate

The surface loading rate is the most common design parameter of settling tanks.

$F = \frac{Q}{A}$, where

F = Surface loading rate ($m^3/m^2 \cdot h$)
Q = Wastewater flow (m^3/h)
A = Surface area of the tank (m^2)

Normally, the surface loading rate of primary settling tanks should not exceed 3.8 $m^3/m^2 \cdot h$ (water depth less than 2.5 m) to 4.8 $m^3/m^2 \cdot h$ (water depth more than 2.5 m) by design peak flow.

According to Norwegian design guidelines, correction factors depending on the tank design should be used while determining the surface loading rate:

Rectangular tanks: $A_{tot} = A + B$

Circular and quadratic tanks: $A_{tot} = A + ½\sqrt{A}$

A_{tot} = Total tank surface area
A = Necessary tank surface area
B = Tank width

5.3.3 Effluent

Appearance

Disturbances in the primary settling tanks may affect the effluent appearance, and this should therefore be observed regularly.

Settleable Solids

The main task of primary settling tanks is to move settleable solids. Regularly based control routines are necessary. Assessment can be made from these criteria:

Settleable solids (ml/l) in primary effluent	Scale
< 0,3	Good
0,3-0,5	Acceptable
>0,5	Bad

Suspended Solids

The suspended solids removal rate can be calculated in this way:

$$E = \frac{SS_{in} - SS_{out}}{C_{in}} \bullet 100$$

E = Removal rate (%)
SS_{in} = Influent concentration of suspended solids (mg/l)
SS_{out} = Effluent concentration of suspended solids (mg/l)

5.4 Troubleshooting

Table 5.2 presents a troubleshooting guide that identifies problems and presents possible solutions.

Table 5.2 Troubleshooting guide for primary sedimentation.

Problems	Probable causes	Solutions
Floating sludge	• Sludge decomposing in tank	• Remove sludge more frequently or at a higher rate
	• Return of well nitrified waste activated sludge	• Vary age of returned sludge, or move point of waste sludge recycle
	• Sludge withdrawal line plugged	• Flush or clean line
Black and odorous septic wastewater or sludge	• Improper sludge removal pumping cycles	• Increase frequency and duration of pumping cycles until sludge density decreases to undesirably low value
	• Inadequate pretreatment of organic industrial wastes	• Pre-aerate wastewater. Have wastewater pretreated by industry
	• Sewage decomposing in collection system	• Add chemicals or aerate in collector system
	• Recycle of excessively strong digester supernatant	• Improve sludge digestion to obtain better quality supernatant. Reduce or delay withdrawal until quality improves. Select better quality supernatant from another digester zone. Discharge supernatant to lagoon, aeration tank or sludge drying bed
	• Sludge withdrawal line plugged	• Clean line
	• Septic dumpers	• Regulate or curtail dumping
	• Insufficient run time for sludge collectors	• Increase run time or run continuously
Scum overflow	• Frequency of removal inadequate	• Remove scum more frequently
	• Heavy industrial contributions	• Limit industrial waste contributions
	• Improper alignment of skimmer	• Adjust alignment
	• Inadequate depth of scum baffle	• Increase baffle depth

Table 5.2 Troubleshooting guide for primary sedimentation (continued).

Problems	Probable causes	Solutions
Sludge hard to remove from hopper	• Low velocity in withdrawal lines	• Increase velocity in sludge withdrawal lines. Check pump capacity.
Undesirably low solids contents in sludge	• Hydraulic overload	• Provide more even flow distribution in all tanks, if multiple tanks
	• Over-pumping of sludge	• Reduce frequency and duration of pumping cycles
Short circuiting of flow through tanks	• Uneven weir setting • Damaged or missing inlet line baffles	• Change weir setting • Repair or replace baffles
Surging flow	• Poor influent pump programming	• Modify pumping cycle
Excessive sedimentation in inlet channel	• Velocity too low	• Increase velocity or agitate with air or water to prevent decomposition
Poor suspended solids removal	• Hydraulic overloading	• Use available tankage, shave peak flow, chemical addition
	• Short circuiting	• See short circuiting of flow through tanks
	• Poor sludge removal practices	• Frequent and consisting pumping
	• Industrial waste	• Eliminate industrial wastes that hinder settling
	• Density currents wind or temperature related	• Eliminate storm flows from sewer system. Install wind barrier
Excessive growth on surfaces and weirs	• Accumulation of wastewater solids and resultant growth	• Frequent and thorough cleaning of surfaces

6 Activated Sludge Treatment

6.1 Introduction

The activated sludge process is an aerobic, suspended growth, biological treatment method. It uses the metabolic reactions of microorganisms to produce a high quality effluent by converting and removing substances that have an oxygen demand.

The primary scope of this process is to remove suspended, colloidal and dissolved organic matter. Some plants may be designed for removing ammonium also, by nitrification in aeration tanks.

6.2 Description of Process

In the basic activated sludge process, the wastewater enters an aerated tank where previously developed biological floc particles are brought into contact with the organic matter of the wastewater. The organic matter, a carbon and energy source for cell growth, is converted into cell tissue and oxidized end products (mainly carbon dioxide). The contents of the aeration tank are called mixed liquor. The biological mass, referred to as the mixed liquor suspended solids (MLSS) or mixed liquor volatile suspended solids (MLVSS), consists mostly of microorganisms, inert suspended matter, and nonbiodegradable suspended matter.

After the mixed liquor is discarged from the aeration tank, a clarifier (also referred to as a settling tank or sedimentation tank) separates the suspended solids from the treated wastewater. The concentrated biological solids are then recycled back to the aeration tank to maintain a concentrated population of microorganisms to treat the wastewater. Because microorganisms are continually produced (synthesized) in this process, a way must be provided to waste the excess biological produced. These solids are generally withdrawn from the clarifier, although wasting from the aeration tank is an alternative.

The most common activated sludge processes are presented below.

Conventional Process

Influent and returned sludge are added to the activated sludge suspension in the inlet zone of the aeration tank. This is a common solution for small plants. Bigger plants may also be conventional, but this is usually a costly solution as the volume demand will be relatively high.

Another disadvantage is uneven oxygen demand. This may be high in the inlet zone of the tank. In long tanks there may be zones with insufficient air to satisfy the oxygen demand. This may be prevented by increasing the air supply to the inlet zone. Mixed liquor suspended solids (MLSS) concentration is relatively even in the whole tank. Figure 6.1 shows a conventional activated sludge process.

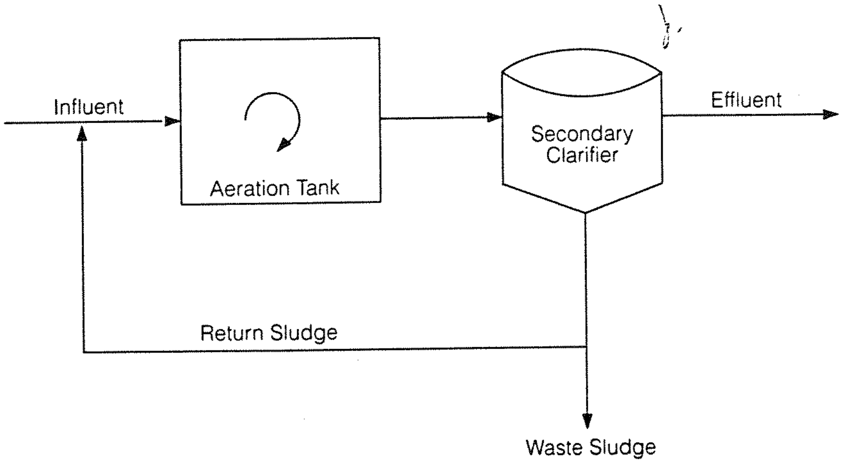


Figure 6.1 Conventional activated sludge process.

Step Feed (Tapered Aeration) Process

Both influent and returned sludge can be added to the aeration tank at different points. Normally, the returned sludge is added to the inlet zone, while the influent flow is distributed to many points towards the outlet zone. In this way the high oxygen demand in the inlet zone is decreased. Mixed-liquor suspended solids (MLSS) is highest in the inlet zone and decreases towards the outlet. A higher biomass can be held in suspension, and the volume demand will be decreased. Figure 6.2 shows the step feed process.

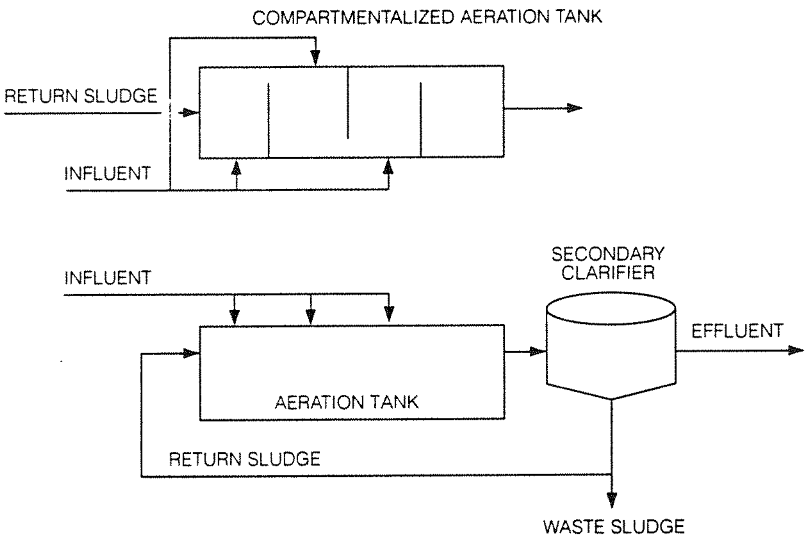


Figure 6.2 Step feed process.

Contact Stabilisation

In this process the return activated sludge is brought to a separate tank for aeration, the stabilisation tank. In this tank there will be a lack of nutrients, and the sludge will absorb organic matter faster when mixed into the contact tank. MLSS in the stabilisation tank is as high as in the return activated sludge. Total aerated sludge mass will be greater than in the other variations of the activated sludge process. Figure 6.3 shows the process.

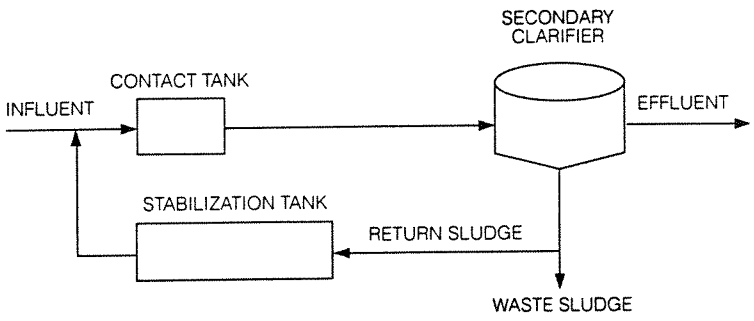


Figure 6.3 Contact stabilisation (biosorption) process

6.3 Process Control

Table 6.1 shows which observations, measurements etc. that are specific to activated sludge treatment.

Table 6.1 Process control of the activated sludge process.

Item	**Observations**	**Measurements and analyses**	**Calculated values**
Aeration tank influent	• Appearance and odour	• Temperature • pH • BOD_5 • COD • Total nitrogen • Total phosphorus • Suspended solids	
Aeration tanks	• Odour • Appearance • Foam formation • Sludge microscopy • Air distribution • Sludge accumulation	• Temperature • pH • Dissolved oxygen • Oxygen uptake rate • Settled sludge volume • MLSS • Nitrate	• Sludge volume index (SVI) • Sludge age • Sludge loading

Table 6.1 Process control of the activated sludge process (continued).

Item	Observations	Measurements and analyses	Calculated values
Settling tanks	• Floatables • Gas generation • Flow conditions • Sludge accumulation	• Secchi depth • Sludge blanket level	• Surface loading rate • Mass surface loading rate
Return activated sludge		• Recycled sludge • Settled sludge flow • MLSS	• Ratio of recycled flow
Surplus (waste) sludge		• Sludge wasting • MLSS	• Sludge production
Settling tank effluent	• Appearance	• Settleable solids • Suspended solids • BOD_5 • COD • Ammonium • Nitrate • Total phosphorus	• Removal rates

6.3.1 Aeration Tank Influent

Appearance and Odour

Appearance and odour should be observed to prevent malfunctioning of the biological step. Undesirable additives to the wastewater are septic wastewater, petrol, oil or solvents and peak loads from dairies, canneries etc.

Temperature

All biological processes are temperature dependant. The process velocity increases with increasing temperature. If there is no dominating industry effluent connected, the temperature will vary little from day to day. It is sufficient to measure temperature once or twice a week.

pH

Normally, the pH is relatively stable through the day. Industry effluents may cause large and rapid variations, however. These variations may disturb the biological process, and to be sure that these variations are observed, continuous pH-registration is recommended.

BOD_5/COD

Organic load is determined by the wastewater flow and the content of organic matter. Information about the organic load is essential in order to calculate the sludge loading or the food to microorganism ratio (FiM). The sludge loading is an important operational parameter at activated sludge treatment plants. BOD_5 and COD should be determined routinely.

COD is often measured instead of BOD_5, since COD is easier to analyse. For all plants, the BOD_5/COD ratio is almost constant. However, the ratio varies from plant to plant, and we therefore have to analyse both parameters for some samples. If there are industrial effluents connected, the BOD_5/COD ratio is seldom constant.

Total Nitrogen

If nitrification and nitrogen removal is part of the biological treatment, the nitrogen content of influent should be determined. The parameter is not necessary for process control.

Total Phosphorus

Normally, total phosphorus is of little value for process monitoring. The parameter may be used to control the BOD_5:N:P ratio and removal rates in different parts of the plant at chemical precipitation plants.

Suspended Solids

Normally, this analysis is of no interest to process control, but the parameter may be useful. Effects of return flows, such as sludge liquor, on influent quality and sludge loading of the biological process may be investigated.

6.3.2 Aeration Tanks

Odour

Bad odour usually occurs due to lack of oxygen or sludge accumulation in aeration tanks.

Appearance

The wastewater quality affects the activated sludge colour. Fresh sludge will always have a brownish colour, often very light brown.

Lack of oxygen may give the sludge a dark, and in serious cases, a black colour. The same is likely to occur if the wastewater has been stored in pumping stations or sewage pipes for so long time that it becomes nearly septic. Especially at process start this may represent a problem. Peak loads of septic sludge may lead to poisoning of activated sludge.

Sludge poisoning will also lead to colour changes, but the final colour of the sludge will vary with the poisoning cause.

Foam Formation

In a well functioning plant, a moderate layer of light brown foam will be formed at the aeration tank surface.

At process start up and during periods of low MLSS content in aeration tanks, large amounts of white scum appear. This scum will decrease with increasing MLSS content. This phenomenon may be an indication of uneven MLSS content in aeration tank. If this occurs during normal surplus sludge removal, solids washout from settling tanks may be the cause.

At low loaded plants with low sludge production and low amounts of surplus sludge, the sludge in the aeration tanks will get old. This often leads to a thick, viscous, dark foam at the aeration tank surface. This kind of foam building is a sign of too low sludge wasting.

Sludge Microscopy

If the sludge settleability is bad, the sludge should be examined using a microscope. In this way, sludge bulking caused by filamentous bacteria can be determined. This is described in the section "Sludge Bulking" (Chapter 6.4.1).

Air Distribution

Air distribution between parallel tanks or in the tanks should be observed and regulated if necessary, by the air supply system.

Sludge Accumulation

Sludge accumulation is normally caused by insufficient mixing. If preliminary and primary treatment are deficient, the risk of sedimentation in the aeration tanks increases.

Sludge accumulation should be monitored regularly by poling or use of a sludge level meter. Exposed tank parts are corners, below and behind aerators and at the tank bottom opposite aerators.

Temperature

The aeration tank temperature affects the purification process. The removal rate (organic matter) decreases with decreasing temperature, but at low or medium loaded plants, the effect on BOD_5 removal is normally insignificant. The temperature strongly affects the nitrification process.

pH

Large variations in influent pH affect the aeration tank pH. Even though the activated sludge process can function over a wide pH-range, rapid changes may affect the process in a negative way. The nitrification process is particularly sensitive. Nitrification also causes a decrease in the water alkalinity.

The optimum pH-value of the process is 7, but there are examples of plants that operate at pH 5, and plants that performs well at pH 9. However, the risk of external disturbance will then be greater. If the plant is operated at low pH, this could result in increased corrosion. pH should be measured at least every week.

Dissolved Oxygen Concentration (DO)

The dissolved oxygen concentration (DO) varies in the aeration tank, depending on the kind of process used (Chapter 6.2). The aeration should be sufficient to maintain necessary mixing (avoid sludge accumulation) and a certain level of dissolved oxygen throughout the tank.

DO should be measured daily at different points in the tank when the oxygen demand is assumed to be highest. To estimate the time of day when the oxygen demand is highest, a series of measurements should be taken once or twice a year to find the DO distribution in the tanks during the day. During the investigations, the air supply must not be changed.

The oxygen supply should be sufficient to maintain DO of at least 2 g O_2/m^3. This DO level is assumed to give excess oxygen into the sludge floc centres. Higher DO is not beneficial for the process and will be costly for the owner. Increasing DO from 2 to 6 g O_2/m^3 may double the energy consumption.

Oxygen Uptake Rate

The oxygen uptake rate of mixed liquor is a biological activity measurement. The measurements are done by pouring the sample into an Erlenmeyer flask. The flask is corked, and the sample stirred continuously. The DO is measured continuously or with short intervals. The results are plotted in a diagram, and the oxygen uptake rate is calculated based on this diagram.

Example: By time T_1 = 2 minutes, C_1 = 3.0 g O_2/m^3. By time T_2 = 6 minutes, C_2 = 1.5 g O_2/m^3. Oxygen uptake rate is:

$$\frac{C_2-C_1}{T_2-T_1} \bullet 60 = \frac{1.6}{4} \bullet 60 = 22.5 \text{ g } O_2/m^3$$

The oxygen uptake rate is MLSS-dependent. The specific oxygen uptake rate is calculated as g O_2/kg SS·h.

The oxygen uptake rate varies during the day; It reaches its maximum when the organic load is highest (normally in the middle of the day and in the evening), and is lowest when the organic load is low. If the air supply is constant, DO will vary with the oxygen uptake, but in the opposite direction. This means that DO decreases with increasing oxygen uptake, and vice versa.

Figure 6.4 shows how the oxygen concentration varies during the day at a plant with dairy contributions. A grab sample measurement at this plant at 8 a.m. will give completely incorrect information about the oxygen conditions.

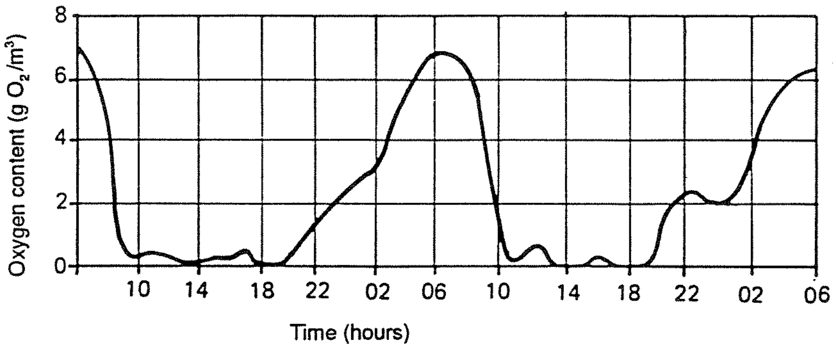


Figure 6.4 Dissolved oxygen variations in the aeration tanks by constant aeration. Continuous registration during a whole day.

The reason for decreased activity, and thereby decreased oxygen uptake, is normally poisoning caused by metals, acids, bases, oils, septic peak loads or similar. The sludge activity will also vary during the year. The changes are due to temperature differences, varying loads etc. The variations may be plant specific. If the oxygen uptake rate is used as a monitoring parameter, the "normal" oxygen uptake rate should be known.

Figure 6.5 shows a "normal" relation between the oxygen uptake rate, sludge loading (BOD loading) and time of the day (day or night). Figure 6.5 may be used for all activated sludge plants, but the deviation from the diagrams should be more than 50% before poisoning can be ascertained. If nitrification is present, the oxygen uptake rate will be higher.

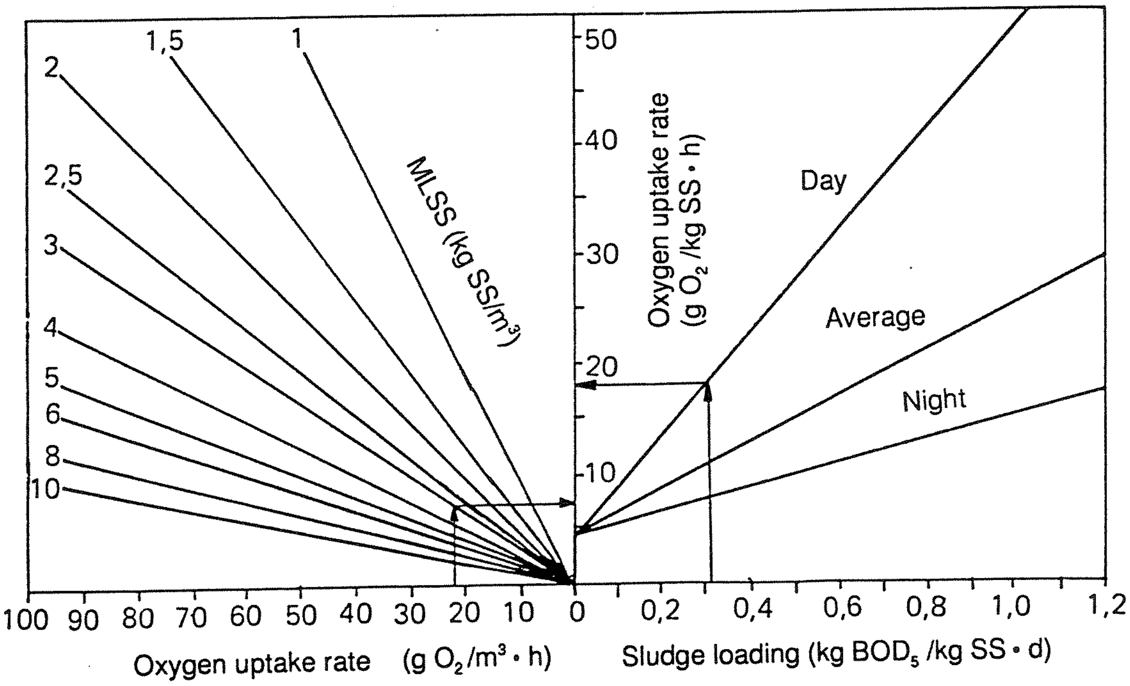


Figure 6.5 Nomogram for controlling measured oxygen uptake rate against "normal values".

To illustrate how to use the nomogram, the previous example will be used.

Example: 1. Measurements:
Oxygen concentration by time T_1 = 2 minutes: 3.0 g O_2/m^3
Oxygen concentration by time T_2 = 6 minutes: 1.5 g O_2/m^3
2. Calculation:
Oxygen uptake rate = $\frac{3.0-1.5}{6-2} \bullet 60$ = 22.5 g $O_2/m^3 \cdot h$
3. Analysis:
MLSS = 3 kg/m^3
4. Nomogram:
Specific oxygen uptake rate = 5.5 g O_2/kg SS·h
5. Influent analysis:
BOD_5-content = 200 g/m^3
6. Measurement:
Water flow = 437 m^3/d
7. Calculation:
Organic load = $\frac{200 \bullet 437}{1000}$ = 87.4 kg BOD_5/d
8. Measurement:
Aeration tank (conventional process) volume = 95 m^3
9. Calculation:
Sludge loading = $\frac{87.4}{3 \bullet 95}$ = 0.31 kg BOD_5/kg SS·d
10. Nomogram:
The measurement is done early afternoon. The "Day" diagram is used.
Expected specific oxygen uptake rate is 18.2.
11. Calculation:
Deviation = 18.2 - 7.5 = 10.7 g O_2/kg SS·h. This is $\frac{10.7}{18.2} \bullet 100\%$ = 59% of expected value from Figure 6.5.
12. Conclusion:
Oxygen uptake rate is low and indicates sludge poisoning. Further investigations should be made.

Settled Sludge Volume

The settled sludge volume indicates the part of the settling tank volume that is occupied by sludge. The settled sludge volume is used to control sludge wasting. The day to day changes in settled sludge volume should be as small as possible, and the changes should be within the range of 200 - 400 ml/l. Normally, higher settled sludge volumes are not preferred, as this may result in too high solids loading on the settling tank.

Settled sludge volume is measured by filling a 1 litre graduated cylinder with sludge (mixed liquor) and determine the volume of settled sludge after 30 minutes quiescent settling.

Nitrate

To determine whether or not nitrification takes place in the aeration tank(s), supenatant from a settled sludge volume sample is analysed for nitrate (NO_3). If the nitrate content is higher than about 5 - 10 g N/m^3, full nitrification can be anticipated.

Mixed Liquor Suspended Solids (MLSS)

Aeration tank sludge content, measured as suspended solids, is called mixed liquor suspended solids (MLSS). MLSS is a measure of the sludge mass in the suspension, given as g/m^3, kg/m^3 or g/l.

Analysis of MLSS and influent organic matter are necessary to calculate sludge loading (F:M-ratio) and sludge age. MLSS is usually between 2 000 and 5 000 g/m^3. If nitrification is preferred, MLSS may be higher. Generally, frequent removal of small amounts of surplus sludge are preferred rather than less frequent removal of large volumes.

Observations and Analyses of Water and Sludge in the Settled Sludge Volume Test

Observations of decant and sludge in the settled sludge volume test may give useful information about operational problems. Figure 6.6 shows four typical results of the settled sludge volume test with reference to possible explanations of the results.

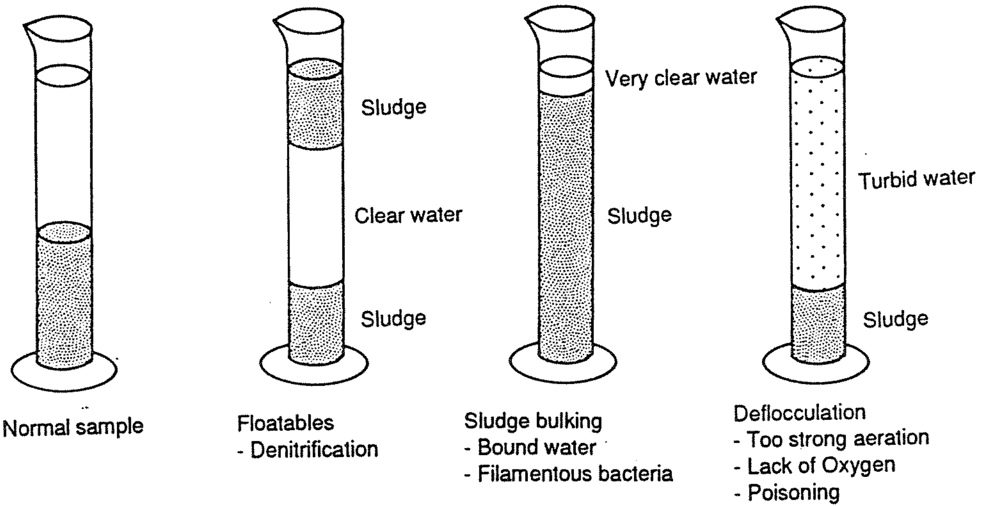


Figure 6.6 Observations of a settled sludge volume sample.

Sludge Volume Index (SVI)

SVI is a measure of sludge settleability and shows the volume 1 g of suspended solids occupy in the measuring cylinder.

When the settled sludge volume is determined to x ml/l and the mixed liquor suspended solids content is y g/l, the sludge volume index is

$$\mathrm{SVI} = \frac{x}{y}\ \mathrm{ml/g}$$

At municipal wastewater treatment plants the following assessment may be relevant:

SVI = 40 - 150 ml/g - Readily settleable sludge
SVI > 200 ml/g - Only partly settleable sludge

If wastewater with large amounts of readily degradable organic matter (e.g. from the food industry) is discharged to the plant, SVI may exceed 200 - 500 ml/g. High SVI indicates sludge bulking or deflocculation.

Sludge Age (Solids Retention Time, SRT)

Sludge age indicates average retention time of sludge particles in aeration tanks. Sludge age is the ratio between total sludge mass and daily removal of surplus sludge and loss with effluent water. Sludge age is calculated as follows:

$$G = \frac{SS_a \bullet V}{SS_s \bullet Q_s + Q \bullet SS_e}$$

G = Sludge age - days
SS_a = MLSS of aeration tank(s) - kg/m^3
V = Volume of aeration tank(s) -m^3
SS_s = MLSS of surplus sludge - kg/m^3
Q_s = Sludge wasting - m^3/d
Q = Effluent flows - m^3/d
SS_e = Effluent suspended solids - kg/m^3

Sludge age is the most important and best parameter for controlling the activated sludge process. The parameter is based on suspended solids analyses only and these analyses are rapidly executed.

Determination of sludge age is also based on sludge wasting measurements. All plants should have the possibility of measuring sludge wasting rate.

Sludge Loading (F:M Ratio)

This parameter shows the amount of organic matter, measured as BOD_5 , that each kg of mixed liquor suspended solids has to "work with" every day. Sludge loading is the ratio between BOD_5 loading per day and the total sludge mass in the aeration tanks. The calculation is as follows:

$$F = \frac{BOD_5 \bullet Q}{SS_a \bullet V}$$

F = Sludge loading - kg BOD_5/kg SS·d
BOD_5 = BOD_5 concentration in influent (daily composite sample) - kg/m^3
SS_a = MLSS in aeration tank(s) - kg/m^3
V = Volume of aeration tank(s) - m^3
Q = Wastewater influent flow - m^3/d

There is a relation between sludge age and sludge loading. There is also a relation between the two parameters and the expected removal rate . Table 6.2 shows normal values of sludge loading and sludge age, dependant of removal rate.

Table 6.2 ***Normal values of sludge loading and sludge age, dependant of expected removal rates.***

Expected results	**Sludge loading, F (kg BOD_5/kg SS·d)**	**Sludge age, G (days)**
Partly removal of readily degradable organic matter (BOD)	> 0.8	1 - 2
Removal of the main part of readily degradable organic matter (BOD)	0.3 - 0.5	3 - 5
Nitrification in addition to BOD-removal	< 0.15	> 10
Nitrification and stabilised surplus sludge	< 0.08	> 20

The temperature in the aeration tanks affects both the nitrification rate and the BOD_5-reduction. Table 6.2 is valid for about 10°C. Lower temperature demands lower sludge loading and higher sludge age to achieve the same results. Higher temperature will, on the contrary, allow higher sludge loading and lower sludge age.

6.3.3 Settling Tanks

Floatables and Gas Generation

Some foam formation in the settling tanks is normal, especially at plants without primary settlement tanks. There are many reasons for this. Sludge gas generation caused by denitrification, anaerobic digestion or too strong aeration in the last part of aeration tanks are often main reasons.

Flow Conditions

Turbulent flow or uneven loading of a settling tank, or uneven distribution between multiple tanks may reduce the sludge separation.

Sludge Accumulation

Sludge blanket monitoring shows how the sludge scrapers function. In flat-bottomed tanks, the sludge blanket thickness should be as small as possible. Continuous sludge scraping is often necessary. Sludge accumulation in tank bottoms or in hoppers increase the risk of gas generation and the formation of foam.

Sludge accumulation in hoppers may be caused by to little slope of hopper walls, too large flat part at the bottom or inadequate sludge pumping routines. Sludge accumulation in hoppers can be reduced by increasing the return sludge flow. Accumulation caused by too little wall slope or too large flat bottom may be removed by regularly poling.

Secchi Depth

Normally, the relationship between the Secchi depth and the removal rates of BOD and suspended solids is reliable. The Secchi depth is easy to measure. The measurement is of great informative value and should be executed daily.

The results may be assessed by this ranking:

Secchi depth (cm)	Scale
> 80	Good
40 - 80	Acceptable
< 40	Bad

Sludge Blanket Thickness

Sludge blanket thickness monitoring in settling tanks can be used to control the return activated sludge flow. Sludge accumulation in the settling tanks should not occur. If the sludge blanket thickness increases, the return sludge flow should also be increased.

Too thick sludge blanket may cause suspended solids erosion from the blanket to the effluent, resulting in poor removal rates. In flat-bottomed tanks, the sludge blanket should be no thicker than about 10 cm. Thick sludge blanket means long sludge retention time in the settling tanks.

Surface Loading Rate

This is the most important parameter for the settling tanks and especially their ability to remove small flocs.

The surface loading rate is calculated as follows:

$$F = \frac{Q}{A}$$

F = Surface loading rate - m/h or $m^3/h \cdot h$
Q = Wastewater flow - m^3/h
A = Necessary tank surface area - m^2

To compensate for disturbances in the inlet and outlet zones of the tanks, the surface loading rate calculation is based upon a reduced surface area:

Rectangular tanks: $A_{tot} = A + B$
Circular or quadratic tanks: $A_{tot} = A + ½\sqrt{A}$

A_{tot} = Total tank surface area
A = Necessary tank surface area
B = Tank width

Solids Loading Rate

This parameter is used in plant design. It may also be useful for plant operation. It expresses the settling tank loading of suspended solids per unit area and unit time. It can be calculated as follows:

$$R_{ss} = \frac{Q \bullet SS_A}{A_{tot}}$$

R_{ss} = Solids loading rate - kg SS/m^2·h
Q = Wastewater flow - m^3/h
SS_A = Aeration tank MLSS - kg SS/m^3
A_{tot} = Total tank surface area - m^2

6.3.4 Return Activated Sludge

Flow

Return activated sludge pumping should, strictly speaking, vary proportional to the influent flow. This possibility does, however, not exist at most plants. By constant return flow, high influent flow will lead to more sludge in the settling tanks. MLSS in aeration tanks decreases as the sludge loading of the settling tanks increases.

The return flow should be as low as possible, but high enough to keep the settling tanks free from sludge. Foam formation, odour and sludge accumulation are signs of too low return activated sludge flow.

Measurement of the return activated sludge flow should be possible at all plants. At small plants it is often difficult to achieve low enough return flows because of pump capacities, self-cleansing of pipes etc.

Recycling Ratio

The recycling ratio flow is defined as:

$$r = \frac{q_r}{Q}$$

r = recycling ratio of recycled flow
q_r = return activated sludge flow - m^3/h
Q = wastewater flow - m^3/h

Settled Sludge Volume

Settled sludge volume of return activated sludge is measured in the same way as for the mixed liquor in aeration tanks. Settled sludge volume of return activated sludge is useful as one of many operation parameters.

Mixed Liquor Suspended Solids (MLSS)

The relation between MLSS in the aeration tanks and MLSS of return activated sludge is unambiguously dependant on the ratio of recycled flow:

$$\frac{SS_R}{SS_A} = \frac{1 + r}{r}$$

SS_R = MLSS of return activated sludge - kg SS/m^3
SS_A = MLSS in aeration tanks - kg SS/m^3
r = Recycling ratio

There is an upper limit of MLSS in return activated sludge. The above equation expresses a mass balance where effluent suspended solids is neglected.

Figure 6.7 shows a nomogram of the relationship between MLSS in aeration tanks, MLSS of return activated sludge, and sludge volume index and recycling ratio. This will facilitate the control of the return activated sludge flow.

Example: Known parameters:

Sludge volume index	SVI = 100
MLSS in the aeration tanks	SS_A = 4 kg/m^3

What is the minimum recycling ratio without decreasing MLSS in the aeration tanks?

The nomogram in Figure 6.7 shows $r \geq 0{,}6$

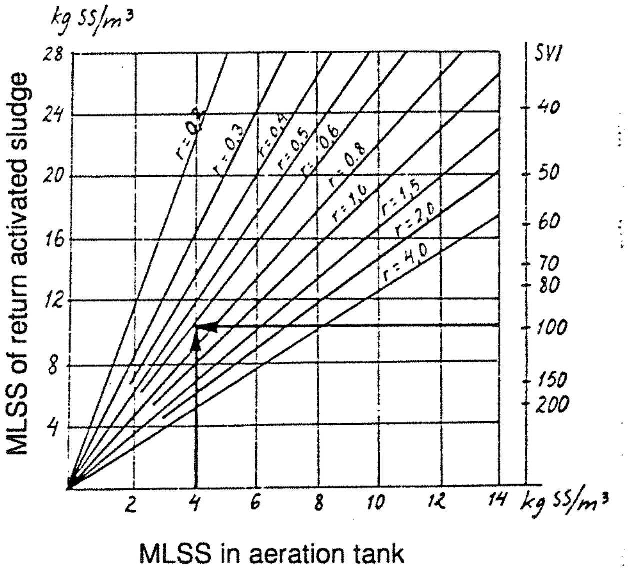


Figure 6.7 ***Relation between sludge volume index, recycling ratio and MLSS of return activated sludge and in aeration tanks.***

If SVI increases, achievable MLSS of the return activated sludge decreases. The recycled flow must be increased to have a constant MLSS in the aeration tanks, or decreasing MLSS in the aeration tanks must be accepted.

6.3.5 Surplus Sludge

Sludge Wasting

The most common strategy for the operation of activated sludge plants is to keep the sludge age constant. This means a sludge wasting rate which corresponds with the rate which corresponds with the rate of sludge production in the plant. This stresses the need for an operational standard which is based on determination of sludge wasting quantities.

Surplus sludge should be removed as even as possible during the day. At some small plants, it may for economic and technical reasons be necessary to store the sludge in the process and remove larger quantities more seldom. Generally, this kind of operation is undesirable and is not recommended.

Figure 6.8 shows how sludge wasting rate can be calculated. The figure does not include sludge loss in the effluent and thereby a decreased sludge age to be compensated by less sludge wasting.

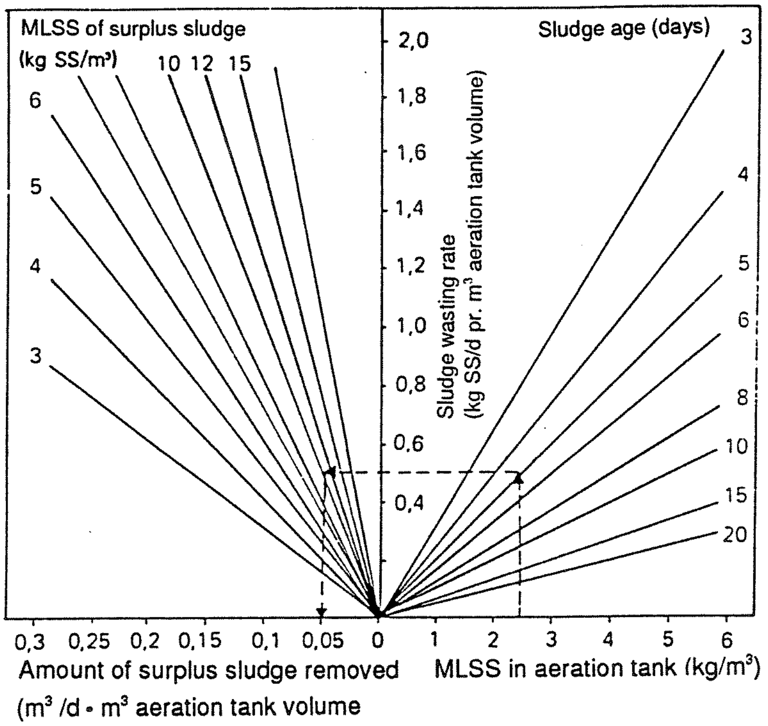


Figure 6.8 Nomogram for calculating sludge wasting.

Example: Use of the nomogram in Figure 6.8.

Known values:

MLSS in aeration tanks	2,5 kg/m^3
MLSS of surplus sludge	10 kg/m^3
Volume of aeration tanks	1800 m^3
Wastewater flow	12000 m^3/d
Effluent suspended solids	15 g/m^3
Necessary sludge age	5 days

How much surplus sludge must/can be removed daily?

1: Necessary surplus sludge removed (from Figure 6.8):
= 0,5 kg SS/d·m^3 · aeration tank volume
= 0,5 · 1800
= <u>900 kg SS/d</u>

2: Necessary sludge wasting rate (from Figure 6.8):
= 0,05 m^3/d·m^3 aeration tank volume
= 0,05 · 1800
= <u>90 m^3/d</u>

Sludge loss by effluent water is not taken into account in Figure 6.8. Sludge loss is:
= 0,015 kg SS/m^3 · 12000 m^3/d
= <u>180 kg SS/d</u>

Necessary surplus sludge removed (corrected):
= 900 kg SS/d - 180 kg SS/d
= <u>720 kg SS/d</u>

MLSS of surplus sludge gives sludge wasting rate (corrected):

$$= \frac{720 \text{ kg SS/d}}{10 \text{ kg SS/m}^3}$$

= <u>72 m^3/d</u>

MLSS

MLSS of surplus sludge is an important operational parameter and is used for controlling sludge wasting. See previous section.

Sludge Production

Accurate information about sludge production rate may be used as a basis for new plant design on upgrading and extensions of existing plants. Sludge production may also be used as a basis for budgeting sludge operation costs.

Sludge production is calculated from surplus sludge data, MLSS and plant loading.

Example:	Population served:	100 000 pe
	Influent flow:	40 000 m^3/d
	BOD_5-loading of biological step	5 000 kg/d
	BOD_5 reduction (80% removal rate)	4 000 kg/d
	Sludge wasting rate	500 kg/d
	MLSS of surplus sludge	8 kg SS/m^3

Sludge production:

Totally: $500 \cdot 8 = 4000$ kg SS/d

Specific sludge production per pe: $\frac{4000 \bullet 10000}{100000} = \underline{40\ g\ SS/pe \cdot d}$

Specific sludge production per BOD_5-production:

$$\frac{4000}{4000} = \underline{1\ g\ SS/g\ BOD_5\ removed}$$

Specific sludge production per m^3 wastewater: $\frac{4000 \bullet 10000}{40000} = \underline{100\ g\ SS/m^3}$

6.3.6 Settling Tank Effluent

Appearance

Visual observation of the effluent appearance may provide useful information about operational conditions.

Settleable Solids

Measurement of settleable solids gives an indication of settling tank performance. The results may be assessed the following way:

Settleable solids (ml/l)	Scale
< 0.1	Good
0.1 - 0.3	Acceptable
> 0.3	Bad

Suspended Solids

Effluent suspended solids should be measured routinely. Effluent quality is assessed the following way by means of suspended solids:

Suspended solids (g/m^3)	Scale
< 20	Good
20 - 30	Acceptable
> 30*	Bad

* By low loading and nitrification: > 60 g/m^3

High suspended solids concentration indicates poor settling tank separation efficiency or poor sludge settleability.

Organic Matter

Effluent quality monitoring should include both BOD_5 and COD analysis. It may also be beneficial to analyse dissolved organic matter to find the ratio dissolved/suspended organic matter.

The effluent concentration depends on loading and wastewater temperature. By temperatures about 10 - 12 °C and sludge loading $F \leq 0.3$ kg BOD_5/kg SS·d, the values may be assessed in this way:

BOD_5 (g O_2/m^3)		COD (g O_2/m^3)		Scale
Dissolved	Total	Dissolved	Total	
< 7	< 20	< 30	< 40	Good
7 - 15	20 - 30	30 - 60	40 - 70	Acceptable
> 15	> 30	> 60	> 70	Bad

These values are valid mainly for domestic wastewater. By large industrial contributions, the values can be much higher.

High dissolved BOD_5 content indicates bad functioning and reduced sludge activity. High total BOD_5 may occur for the same reasons, but if the SS content is high too, poor settling tank performance may be the cause.

Nitrogen Compounds

If the plant is designed for nitrification, effluent ammonium and nitrate should be monitored. Effluent ammonium should be lower than 2 g N/m^3 if full nitrification is achieved. Nitrification strongly increases the oxygen demand and the recipient oxygen demand decreases accordingly.

Total Phosphorus

This parameter is of interest for the process control only if the plant has a chemical treatment step or if the activated sludge process is operated with enhanced biological phosphorus removal. At biological-chemical plants, it can be of some interest to know the phosphorus removal rate of the biological step.

6.4 Troubleshooting

It is important to realize that the reaction time of biological processes is long. Practical experiences have shown that you should wait 1-3 weeks (depending on process SRT) after making an activated sludge process change before deciding the success or failure of the change.

6.4.1 Aeration Tank Problems

Table 6.2 Problems in aeration tanks.

Problems	Probable causes	Solutions
White and fluffy foam	• Low sludge age caused by too low MLSS in aeration tanks (e.g. plant start up)	• Control settled sludge volume. If too low: Increase recycled flow and decrease sludge wasting.
	• Poisoning caused by harmful discharges to the plant	• Control oxygen uptake rate and assess the result using Figure 6.5 • Find the source of the poisoning • Empty the aeration tanks and start once again
Dark, grensy foam	• High sludge age causes growth of bacteria that secretes a fat-material. This material forms the foam.	• Control MLSS and sludge age • Increase sludge wasting rate
Other foaming problems	• Discharge of fat, soap, detergents and various types of industrial waste-water may cause different types of foaming. It is impossible to say if the foaming is harmful to the process without further investigation.	
Sludge accumulation	• Bad mixing	• Increase aeration • Control that aerators are not clogged
	• Bad tank design	• Improve tank design
Aeration system failures		• Improve aeration system
Sludge bulking	• Filamentous bacteria or fungi	• See Tables 6.3 and 6.4

6.4.2 Sludge Bulking

Sludge bulking is, as the name says, a bulking of sludge: It becomes light and voluminous, and has low settleability. Sludge volume index increases.

The causes for sludge bulking may be numerous. The probable cause has a certain influence on how the problem should be solved.

Sludge Bulking Caused the Growing of Filamentous Bacteria

The biomass of an activated sludge plant consists mainly of bacteria, fungi and protozoa. There are many species, of which several are undesirable at a treatment plant. Influent wastewater quality, temperature, oxygen conditions, light conditions etc. determine which species that will be dominating.

To solve a sludge bulking problem, the cause will have to be identified first. Common causes and solutions are listed in Table 6.3.

Table 6.3 Sludge bulking problems caused by filamentous bacteria.

Problem	**Possible causes**	**Solutions**
Sludge bulking caused by filamentous bacteria	• Low oxygen content of aeration tank (< 0.5 g O_2/m^3)	• Increase total air supply or regulate distribution in tanks or between multiple tanks to bring oxygen content at all places above the minimum level.
	• Discharge of wastewater with high carbohydrate content	• Usual by food industry effluents. Equalize the discharges throughout the day at the source or at the treatment plant. Distribute the influent all over the aeration tank - step aeration.
	• Nutrient deficiency	• This may be the case at plants with industrial discharge. This is out of question when influent nutrients are higher than: Tot-N 5% of BOD_5 Tot-P 1% of BOD_5 Fe 0.5% of BOD_5 If there are shortage of any of these, they may be added
	• Flushing of accumulated materials from the sewer system	• More frequent flushing of pipes with little slope

After having ascertained causes and carried out necessary means, sludge bulking can be stopped by:

- Simultaneous precipitation
- Chlorination
- Adding of hydrogen peroxide (H_2O_2)

By chlorination, the following should be attended:

- Chlorination should be done in a controlled way by a dosing pump.
- Add chlorine to the return activated sludge

- Start with low dosages (≤ 2 g Cl_2/kg SS·d)
- Increase dosage gradually (maximum 1 g Cl_2/kg SS·d)
- Control dosage by effluent turbidity, alternatively Secchi depth, measurements. Increased turbidity or decreased Sechhi depth indicates too high dosage. If effluent turns white, the dosage is far too high.
- Maximum dosage should not exceed 9 - 10 g Cl_2/kg SS·d
- Monitor effects by sludge volume index measurements. Continue the treatment till SVI ≤ 100 ml/g
-

Necessary dosage of sodium hypochlorite (containing 15% chlorine):

- 2 g Cl_2/kg SS·d = 13.3 ml/kg SS·d
- 1 g Cl_2/kg SS·d = 6.7 ml/kg SS·d
- MLSS = 2,2 kg SS/m^3
- Aeration tank volume 800 m^3
- Dosage:
    Start by $13.3 \cdot 2.2 \cdot 800 = 23.4$ l/d
    Increase by $6.7 \cdot 2.2 \cdot 800 = 11.8$ l/d

By adding hydrogen peroxide, one should notice:

- Adding to the return activated sludge
- Normal dosage 100 - 200 g/m^3 return activated sludge

Sludge Bulking from Other Reasons

This type of sludge bulking is not primarily caused by the biomass composition, but rather by sign of illness of the "normal" biomass. Here too, the causes may be many and will have to be identified in each case. Chlorination will be useless. It may be worthwhile to try to settle the sludge by precipitation. About 10 g Fe/m^3 iron salts may be tried.

Sludge volume index will in these cases be increased for some days and be stabilised at a higher level.

Table 6.4 lists common causes where filamentous bacteria are not involved.

Table 6.4 Sludge bulking problems not caused by filamentous bacteria.

Problem	Possible causes	Solutions
Sludge bulking caused by other means than filamentous bacteria	• High and even sludge loading	• If sludge loading exceeds 0.5 kg BOD_5/kg SS·d, it should be decreased. The loading may be decreased, or the total biomass of the system increased (by decreased sludge wasting).
	• Low pH caused by nitrification	• Decrease nitrification by decreasing the biomass of the system (by increased sludge wasting), if nitrification is not intended • Increase alkalinity by sodium bicarbonate adding
Sludge bulking caused by other means than filamentious bacteria	• Low influent pH	• Find the source and neutralise before discharging
	• Temperature changes	• Experience shows that some plants are exposed to temporarily bulking every spring and autumn

6.4.3 Settling Tank Problems

Table 6.5 Settlement tank problems, possible causes and solutions.

Problems	Possible causes	Solutions
Floating sludge	• Thick, scummy, dark brown foam caused by denitrification	• Decrease MLSS of aeration tank if nitrification is not intended, that is: Increase sludge wasting • Decrease sludge detention time by increasing return sludge flow
	• Blackish sudsy foam formed by digestion of accumulated sludge	• Increase running time of sludge collectors • Increase return sludge flow • Pole and clean regularly
	• Flotation of sludge particles caused by air from aeration tanks	• Shelter aeration tank effluent
	• Grease and oil	• Increase running time of sludge collectors • Increase return sludge flow

Table 6.5 Settlement tank problems, possible causes and solutions (continued).

Problems	**Possible causes**	**Solutions**
Sludge losses	- Sludge bulking	- See Chapter 6.4.2
	- Peak loads	- Regulate inlet pump capacities - Equalize wastewater flow
	- High sludge level	- Increase running time of sludge collectors - Increase return sludge flow
	- Floc deformation	- Decrease air supply to aeration tank - Decrease sludge age

6.4.4 Plant Effluent

Poor effluent quality may be due to insufficient conversion of organic matter in the aeration tanks, poor sludge separation in the settling tanks or a combination of these. Assessment of the final result is based on this information:

Results	**Cause**
Dissolved $BOD_5 > 20\ g/m^3$	Unsatisfactory decomposition
Settleable solids < 0.3 ml/l Suspended solids $> 30\ g/m^3$	Unsatisfactory settleability of activated sludge
Settleable solids > 0.3 ml/l	Unsatisfactory separation

This table may be used for further investigation and assessment:

Table 6.6 Effluent problems, possible causes and their solutions.

Problems	**Possible causes**	**Solutions**
Unsatisfactory decomposition	- Peak loads of organic matter (BOD_5). Even if the average loading during a day is not higher than normal, peak loads at short time intervals will cause lack of oxygen	- Increase total air supply - Regulate air supply by oxygen content or time - Equalize BOD_5-discharges
	- Too low MLSS content in aeration tank	- Increase MLSS by reducing sludge wasting
	- General organic overload	- Improve preliminary treatment - Introduce pre-precipitation - Increase aeration tank volume

Table 6.6 Effluent problems, possible causes and their solutions (continued).

Problems	Possible causes	Solutions
	• Sludge bulking	• See Chapter 6.4.2
	• Poisoning	• Find the source and stop the discharges
Unsatisfactory settleability of activated sludge	• Deflocculation caused by too strong turbulence	• Decrease air supply to the outlet part of the aeration tank • Remove hydraulic jumps or weirs if there are any between aeration tanks and settling tanks
	• Poisoning	• See "Unsatisfactory decomposition"
	• Sludge bulking	• See Chapter 6.4.2
	• High sludge age may cause "pin point flocs", which means a turbid effluent	• Increase sludge wasting
	• Low sludge age may lead to small, nearly transparent flocs in the effluent, even though the effluent is otherwise clear	• Decrease sludge wasting
Unsatisfactory separation	• Peak loads caused by uneven pumping	• Decrease pump time • Differentiate pump sizes • Overflow a part of the wastewater back to pumping sump
	• Hydraulic overload	• Build retention basin • Overflow some wastewater if the total removal rate then improve • Upgrade the sewer system • Try to improve sludge settleability. Use polymer or other flocculants
	• Too high solids loading rate	• Adjust MLSS and return sludge flow to the solids loading capacity of the settling tanks
	• Sludge bulking	• See Chapter 6.4.2
	• Improper hydraulic regime	• Check loading of multiple tanks • Check that effluent weirs are horizontal and distribute flow equal over the weir length

7 Activated Sludge with Chemical Precipitation (Simultaneous Precipitation)

7.1 Introduction

Simultaneous precipitation is a process where precipitants (flocculants) are added directly to the aeration tank or the return activated sludge to increase the phosphorus or suspended solids removal of the plant. Biological and chemical treatment take place at the same time (simultaneously) in the same treatment process.

Simultaneous precipitation can be implemented in every kind of activated sludge treatment plant. A flocculation step between aeration tanks and settling tanks will have a positive effect on the removal rates.

7.2 Description of Process

By phosphorus precipitation phosphates react with trivalent metal salts. The most common precipitants are iron and aluminium salts, ferric chloride, aluminium sulphate (PIX and PAX). Iron (II) sulphate (bivalent) is also used, and oxidised to trivalent in the process. Simultaneous precipitation is favourable for use of iron sulphate if it is added in such a way that bivalent iron of the precipitant can be oxidised to trivalent in the aeration tanks. Iron (II) sulphate should therefore be added to aerated grit chambers or the inlet zone of aeration tanks. By use of PIX, PAX, ferric chloride or aluminium sulphate the dosing point can be more flexible.

Adding precipitants to activated sludge usually improves the sludge settleability, and iron salts addition can be a solution to sludge bulking (see Chapter 6.4.2). This means that the sludge concentration in aeration tanks normally increases by simultaneous precipitation. This must be taken into account by plant operation because solids loading rate of the settling tanks increases. Sludge production will also increase, normally by 20-30% on a dry solids basis if the sludge loading is constant.

7.3 Process Control

Table 7.1 shows the observations, measurements etc. that are specific to simultaneous precipitation and are additional to the parameters outlined in Chapter 6 (Table 6.1).

Table 7.1. Process control of simultaneous precipitation.

Item	Observations	Measurements and analyses	Calculated values
Aeration tank influent		•Alkalinity •Phosphorus	
Dosing equipment	•Mechanical function •Dissolver •Clogging •Corrosion	•Stock of precipitant •Dosing rate •Concentration of solutions	•Molar ratio
Aeration tank		•pH	
Plant effluent		•Alkalinity •Total phosphorus •Ortho phosphate •Residual precipitant •Suspended solids	•Removal rates

In addition to the parameters listed in this table, further consideration should be taken concerning operation of the biological process (Chapter 6.3) and chemical considerations (Chapter 8.3).

7.3.1 Aeration Tank Influent

Alkalinity

Influent alkalinity is an important parameter when dealing with chemical precipitation. Variations may occur from day to day and during the year. Rain storms, snow melting and industrial discharge affect the alkalinity in a combined sewer system.

Influent alkalinity and pH are less important in simultaneous precipitation plants than in chemical treatment plants, due to the buffering capacity of the activated sludge process. At biological-chemical plants, nitrification also has a great influence. The nitrification process decreases wastewater alkalinity. For every g/m^3 nitrogen that is oxidised from ammonium to nitrate, alkalinity will decrease by 0.1 - 0.14 meq/l.

In addition, precipitant dosing itself decreases alkalinity. This is shown in Figure 8.8.

Total Phosphorus and Orthophosphate

Influent total phosphorus and orthophosphate content are important parameters to determine chemical dosing rate and removal rate of the treatment plant.

7.3.2 Dosing Equipment

Mechanical Function

Daily control of the dosing equipment function and the precipitant transport to the dosing point is essential. Function control depends on type of equipment. Follow carefully the specifications in the operation and maintenance manual from the supplier.

Dissolver

Granular or powdered chemicals should be dissolved before dosing. Non-soluble or nearly non-soluble precipitant contaminants will occur. These contaminants may sediment in the dissolver. Routine cleaning is therefore essential.

Clogging

The possibility of pipe clogging between the dosing equipment and the dosing point is always present. The possibility is greatest when aluminium sulphate or lime are used.

To reduce the possibility as much as possible, the pipes should have no sharp bends or low points. The precipitant velocity should be higher than 0.5 m/s. Precipitant concentration should not be too low, as precipitant impurities precipitate easier by low concentration.

To further reduce clogging, the dosing equipment should be placed near the dosing point, pipes should be multiple, or the precipitant should be transported in an open V-shaped channel.

Corrosion

Precipitants based upon iron- and aluminium salts are extremely corrosive to iron and concrete when concentrated. Also iron sulphate solutions can be corrosive.

Those parts of the plant that have been in contact with concentrated precipitants should be closely observed.

Stock of Precipitant

Amount of precipitant in silos or tanks and the precipitant consumption (kg/day) should be controlled regularly to ensure uninterrupted operation and that new orders can be made in time.

Dosing Rate

Precipitant dosing rate and feeding equipment should be controlled weekly. This control must be done manually, since a dosing screw timer or revolution counter will not show whether the screw turns with or without precipitants. Frequent monitoring of flow meters and other automatic equipment is essential.

Generally, the precipitant dosing rate should be as low as possible without unsatisfactory removal rates. Low dosing rates reduce precipitant costs, residual precipitant in the effluent and sludge production. A decrease in sludge production will reduce the sludge treatment costs accordingly.

The dosing rate depends on the influent phosphorus concentration. Simultaneous precipitation experiences show satisfactory results by a precipitant dosing rate according to molar ratios (see next section) of **Al/P or Fe/P >1.5**. This is lower than by chemical precipitation alone, and the reason for this is believed to be the flocculation of activated sludge and partly return of precipitant by the return activated sludge.

By simultaneous precipitation, precipitant dosing can be made very simple due to long detention times in aeration tanks. With a combined sewer system, water flow proportional dosing is not recommended at simultaneous precipitation plants, since storm water flows may cause too high dosing rates, too little alkalinity, and too low pH in the aeration tanks. The dosing rate should be proportional to normal influent total phosphorus load at dry weather conditions. The total phosphorus content should be measured at different times of the day and week. This information should be put into a table which presents expected values. This table should be used for setting the dosing rate. An example is presented below.

Time	Monday - Friday		Saturday		Sunday	
	Total phosphorus load (kg P/h)	Expected dosing rate (kg/h)	Total phosphorus load (kg P/h)	Expected dosing rate (kg/h)	Total phosphorus load (kg P/h)	Expected dosing rate (kg/h)
00 - 06 a.m.	5	115	5	115	5	115
06 -12 a.m.	20	461	15	346	10	230
00 - 06 p.m.	15	346	10	230	10	230
06 - 12 p.m.	10	230	10	230	10	230

The dosing rate could be set this way and adjusted if effluent quality does not comply with standards.

The activated sludge process operates best with a pH around 7.0. Low pH-values (< 6.0) may impair the biological process, and pH by simultaneous precipitation should therefore be around 6.5 - 7.5.

The low pH risk is small when the biological step is operated without nitrification. If nitrification is present or pH for other reasons falls below 6.5, alkali (e.g. bicarbonate, lime, soda) should be added.

Molar Ratio

Molar ratio shows the ratio between the active part of the precipitant (Al^{3+}, Fe^{3+}) and the wastewater phosphorus content, Al : P or Fe : P. This parameter is essential for calculating the necessary precipitant dosing rate.

By simultaneous precipitation, the molar ratio is calculated from wastewater **total phosphorus** content.

Precipitant Concentration

If precipitants are supplied in a liquid form, the precipitant concentration should be checked using a hydrometer (see Figure 7.1) for each supply. If precipitants are dissolved at the plant, the concentration should be monitored the same way.

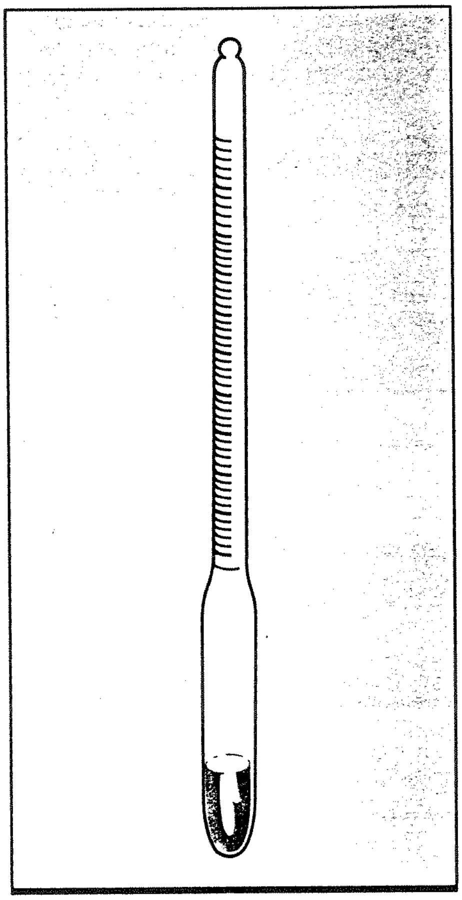


Figure 7.1 ***Hydrometer.***

If deviation in the concentration occurs, the dosing rate must be adjusted to compensate for this.

7.3.3 Aeration Tanks

pH

The pH-value after dosing of precipitant is of great importance both for the biological process and for the phosphorus removal rate. pH must be measured at least once a day.

7.3.4 Plant Effluent

The biological function of the treatment plant is assessed in Chapter 6.3.

Alkalinity

Desirable effluent alkalinity is > 0.5 meq/l. If alkalinity often falls below this value, alkalinity should be measured regularly.

Total Phosphorus

The main objective of precipitant addition is to reduce total phosphorus content. Total phosphorus should therefore be measured regularly. This scale can be used to assess the effluent quality for simultaneous precipitation plants:

Total phosphorus (g P/m^3)	Scale
< 0.6	Good
0.6 - 1.5	Acceptable
> 1.5	Bad

Orthophosphate

Wastewater orthophosphate should be analysed on **filtered samples**. The results give information about shows the precipitation efficiency, and the parameter should therefore be measured daily. The results can be assessed as follows:

Orthophosphate (g P/m^3)	Scale
< 0.1	Good
0.1 - 0.3	Acceptable
> 0.3	Bad

Unsatisfactory precipitation is generally caused by too low dosing rate, or in some cases by wrong pH-range.

Good precipitation is no guarantee for good phosphorus removal. If the sludge separation is bad, the effluent total phosphorus concentration can be high despite of low ortho phosphate concentrations.

Residual Precipitant

Residual aluminium or iron should be as small as possible, both for economical reasons and for the concern of receiving waters. Analysis results from **unfiltered samples** can be assessed like this:

Residual Al (g /m^3)	Residual Fe g/m^3)	Scale
< 0.5	< 1	Good
0.5 - 1.5	1 - 2.5	Acceptable
> 1.5	> 2.5	Bad

Suspended Solids

By simultaneous precipitation, the effluent suspended solids concentration is normally somewhat higher than by the activated sludge process without precipitant addition. This is mainly due to small and light metal hydroxide flocs. Analysis results can be assessed like this:

Suspended solids (g/m^3)	Scale
< 30	Good
30 - 40	Acceptable
> 40*	Bad

* By low load and nitrification: > 60 g/m^3

7.4 Troubleshooting

At a simultaneous precipitation plant, operational problems in the aeration tanks and settling tanks will be the same as for activated sludge plants, see Chapter 6.3. For a simultaneous precipitation effluent, this assessment is valid:

Analysis result	Cause
Orthophosphate > 0.3 g/m^3	Unsatisfactory precipitation
Suspended solids > 40 g/m^3	Unsatisfactory separation

Table 7.2 Problems in simultaneous precipitation.

Problem	Probable causes	Solutions
Unsatisfactory precipitation	•Too low dosing rate	•Control molar ratio. Control pH after increasing dosing rate!
	•Too low pH	•Control molar ratio
		•Control nitrification by analysing nitrate and nitrite in the aeration tank
		•If nitrification - control alkalinity
		•Add lime, soda or sodium bicarbonate
	•Malfunctioning dosing system	•Dosing should be flow proportional or time proportional. Manual, ambient dosing is not recommended
Unsatisfactory sedimentation	•See Chapter 6.4. Floc size and weight may depend on dosing point	•Try other dosing points: - Aerated grit chamber inlet - Aeration tank inlet - Aeration tank outlet - Return sludge
Unsatisfactory separation	•See Chapter 6.4	
Aeration equipment clogging	•This is often a problem by iron dosing in aeration tanks. See "Aeration" in Chapter 6.3	

8 Chemical Treatment

8.1 Introduction

This chapter will discuss chemical precipitation for removal of phosphorus, suspended solids and particulate organic matter. Chemical precipitation can be combined with other units in a treatment plant in several ways:

1. Direct precipitation
2. Pre-precipitation
3. Simultaneous precipitation (see Chapter 7)
4. Post-precipitation

Direct precipitation is primary-chemical wastewater treatment without a biological treatment step. In Norway, this is the most common type of treatment facilities. Two different types of direct precipitation processes are available: Primary precipitation and secondary precipitation. Primary precipitation is precipitation directly following the preliminary treatment step (see Figure 8.1). When adding the precipitant just after primary treatment, the process is called secondary precipitation (see Figure 8.2).

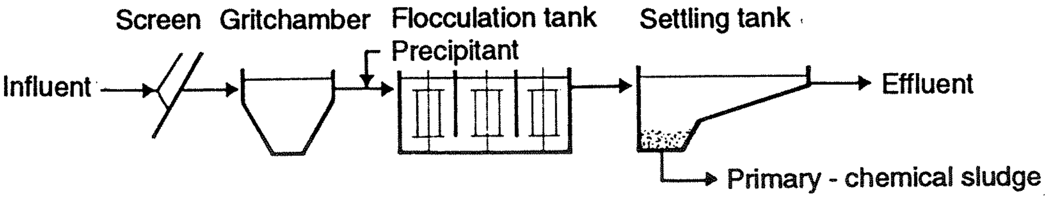


Figure 8.1 Scheme of primary precipitation plant

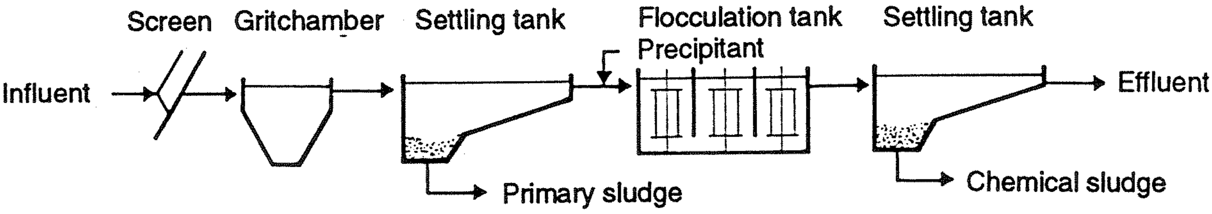


Figure 8.2 Scheme of secondary precipitation plant

Pre-precipitation is the process mode at which precipitants are added upstream the primary settling tank in a secondary treatment plant (see Figure 8.3). The biological treatment step can be an activated sludge process or a biofilm process. Biological ponds are also a possibility after pre-precipitation. An important aspect of pre-precipitation is to control the precipitation effluent so that the organic loading of the biological step is under control, and that there is no lack of nutrients in the biological plant influent. By pre-precipitation, the organic load of the biological treatment step can be reduced by 25-50%. Pre-precipitation can therefore be a good solution for improving an overloaded secondary treatment plant.

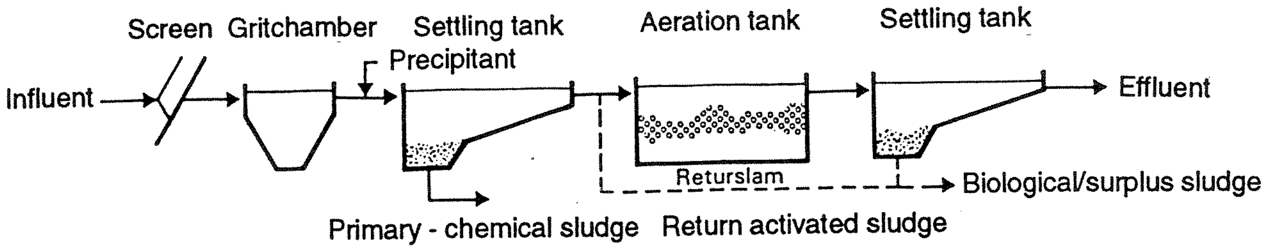


Figure 8.3 Scheme of a pre-precipitation plant with an activated sludge process.

Post-precipitation means precipitation after the biological treatment step (see Figure 8.4). The biological treatment step can either be an activated sludge process, a biofilm process or biological pond. With low alkalinity wastewaters, nitrification in the biological step may cause inferior precipitation results in these types of plants, but normally post-precipitation gives the best removal rates.

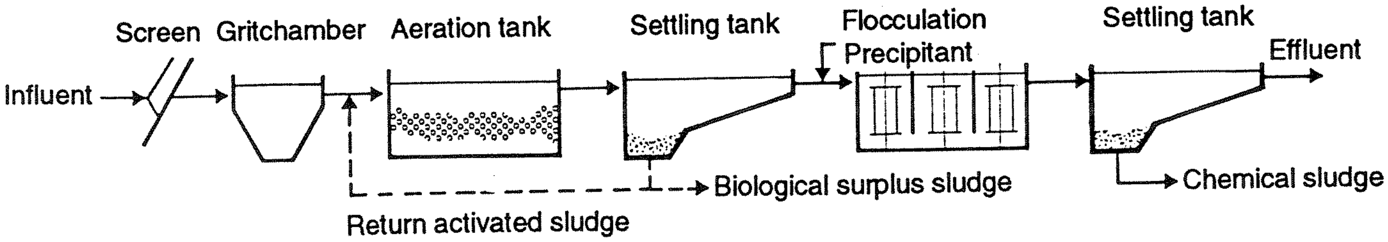


Figure 8.4 Scheme of a post-precipitation plant with an activated sludge process.

The precipitation may also take place in the aeration tanks of an activated sludge process. This is called simultaneous precipitation and is described in Chapter 7. Chemical precipitation in combination with biofilm processes is also possible, but this is will not be discussed in this manual.

Regardless of dose point for the precipitant, the main principles of the process are:

- Adding of precipitants and rapid mixing.
  Rapid and effective mixing of the precipitants into the wastewater stream is essential. The chemical reactions are rapid and there is a possibility of the precipitants making complexes with themselves. The phosphorus removal will then be reduced correspondingly.

- Forming of settleable flocs.
  Velocity gradients and particle collisions are achieved by gentle agitation in the flocculation unit. The sludge particles are attached to each other and form flocs that are

settled in a succeeding settling tank (or floated in a flotation tank). The agitation must not be too turbulent, otherwise the flocs will be destroyed.

- Floc separation.
  Sedimentation is the most common way to separate the flocs from the water, but by post-precipitation with aluminium or iron salts, flotation is also possible. Direct filtration of the flocs is less common, but filtration after conventional gravity separation will give a better effluent quality.

8.2 Description of Process

8.2.1 Chemical Reactions

It is not necessary to know the chemical reactions by chemical precipitation in detail to operate a chemical treatment plant. An overview might be useful, although.

Use of aluminium salts

Aluminium sulphate is in common use as a precipitant in Western Europe. A new generation of aluminium based precipitants are the prepolymerised aluminium salts (PAX). Another aluminium salt in use in small wastewater treatment plants is sodium aluminate, but this is a more expensive product.

Aluminium sulphate ($Al_2(SO_4)_3 \cdot 14\text{-}18\ H_2O$) will dissolve into the water forming Al^{3+}-ions and SO_4^{2-}-ions. Very simplified, we first get aluminium phosphate:

$$Al^{3+} + PO_4^{3-} \rightarrow AlPO_4$$

This means that one mole of aluminium (Al) reacts with one mole of PO_4 and forms one mole of $AlPO_4$. Since the molar ratio between P and PO_4 is 1:1, the molar ratio between Al and P will also be 1:1. 1 kmole of Al is 27 kg, and one kmole of P is 31 kg. The weight ratio between Al and P is:

$$\frac{27}{30} = 0.87$$

When we use aluminium sulphate, $Al_2(SO_4)_3 \cdot 18\ H_2O$, the real equation will be this:

$$Al_2(SO_4)_3 \cdot 18\ H_2O + 2\ PO_4^{3-} \rightarrow 2\ AlPO_4 + 3\ SO_4^{2-} + 18\ H_2O$$

The molecular weight of aluminium sulphate with 18 water molecules is 666. Molecular weights of PO_4 and $AlPO_4$ are 95 and 122, respectively. The equation tells us that we need 666 kg aluminium sulphate to remove 2·95 kg PO_4, and that 2·122 kg aluminium phosphate will be formed.

We know that the solubility of aluminium phosphate, $AlPO_4$, vary with pH. This is shown in Figure 8.5.

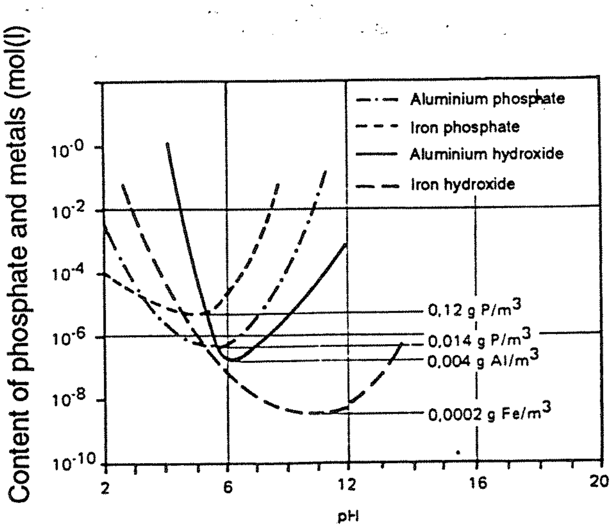


Figure 8.5 The solubility of iron phosphate, aluminium phosphate, iron hydroxide and aluminium hydroxide as a function of pH.

Optimal pH-range for phosphorus removal by adding aluminium sulphate is between 5.8 and 6.3.

Experiences have shown that the addition of aluminium sulphate must be greater than expected from the stoichiometric calculations. This happens because aluminium sulphate also reduces alkalinity and decreases pH in the wastewater. The size of the pH fall will depend on the alkalinity of the wastewater. This is an approximate expression:

$$Al_2(SO_4)_3\cdot 18H_2O + 6\ HCO_3 \rightarrow 2\ Al(OH)_3 + 6\ CO_2 + 3\ SO_4^{2-} + 18\ H_2O$$

Aluminium sulphate reacts with alkalinity (HCO_3^-) so that aluminium hydroxide ($Al(OH)_3$) and carbon dioxide (CO_2) are formed. Aluminium hydroxide forms large flocs. Small particles will be adsorbed to the flocs during the precipitation process.

The main chemical processes that take place when aluminium sulphate is added to wastewater, are now known. It is also known that these processes reduce alkalinity, decrease pH and precipitate aluminium phosphate ($AlPO_4$) and aluminium hydroxide $Al(OH)_3$. Precipitated aluminium phosphate and aluminium hydroxide are the main parts of the chemical sludge.

Processes that take place when prepolymerised aluminium (PAX) is added to wastewater are similar to those of adding aluminium sulphate. In practice, phosphate removal is normally based on simple rules of thumb. The main parameters are the dosing rate, often given by the Al/P molar ratio (typically around 2 in practice), and pH just after precipitation (typically around 6 in practice).

Use of iron salts

Several iron salts are used to remove phosphorus; bivalent ferrous sulphate (Fe^{2+}), trivalent ferric sulphate (Fe^{3+}), trivalent ferric chloride or PIX (Fe^{3+}) and pickling bath (waste from the surface treatment industry).

Precipitation by trivalent iron salts will follow the same theory as for precipitation by aluminium sulphate. In other words, iron phosphate will be formed in this way:

$$FeCl_3 \cdot 6\,H_2O + PO_4^{3-} \rightarrow FePO_4 + 3\,Cl^- + 6\,H_2O$$

The molar ratio between iron and phosphorus is 1:1 while the weight ratio is 1.8:1. Molecular weight of ferric chloride with six molecules of water is 270. Molecular weights of PO_4 and $FePO_4$ are 95 and 151, respectively. The equation tells us that we need 270 kg ferric chloride to remove 95 kg PO_4, thus giving us 151 kg iron phosphate, $FePO_4$.

Precipitation of iron hydroxide will follow a reaction that is about the same as for aluminium sulphate:

$$FeCl_3 \cdot 6\,H_2O + 3\,HCO_3 \rightarrow Fe(OH)_3 + 3\,CO_2 + 3\,Cl^- + 6\,H_2O$$

If we calculate in the same way as for aluminium, we get:

1. $\frac{151}{56}$ = 2.7 kg $FePO_4$ per kg Fe^{3+}
2. We have to use 1.8 kg Fe^{3+} to remove 1 kg P.
3. We get = 1.9 kg $Fe(OH)_3$ per kg Fe.

This means that we have to use $\frac{1.8}{12} \cdot 100 = 15$ kg ferric chloride with 12% Fe^{3+}-content to remove 1 kg P.

Figure 8.5 shows that precipitation of iron phosphate and iron hydroxide have minimum solubility points at different pH-values. The minimum solubility point of iron phosphate is between 4.5 and 5.5, and of iron hydroxide between 7.9 and 9.6.

By use of ferric chloride or PIX, pH in the precipitation process should be between 4.5 and 6.0. This will give the best phosphorus removal.

By use of bivalent iron salts in the precipitation process, e.g. ferrous sulphate, the process will be much more complicated than by use of trivalent salts. Bivalent iron salts oxidise to trivalent salts by consumption of oxygen. This oxidation process depends on many factors, as pH, temperature, oxygen concentration and in some cases also the content of catalytic agents. The oxidation rate is directly proportional to the concentration of bivalent iron and oxygen. When using bivalent iron to remove phosphorus, we have several reactions going on simultaneously.

1. Bivalent iron is oxidised to trivalent.
2. Hydrolysis of trivalent iron to iron hydroxide.
3. Phosphorus and iron reacts and form iron phosphate.

By use of both iron and lime, we try to have a precipitation pH around 9.0. This is the optimal pH for forming iron hydroxides. The hydroxide flocs adsorb colloids during sedimentation. The lime has two functions: To increase pH to the optimal point and to react with the phosphorus compounds. This reaction will be described in the section about lime precipitation.

In this section the theory of using iron salts to remove phosphorus is simplified. The same results as for aluminium will also be valid for iron salts.

Use of lime

Use of lime in combination with iron salts is mentioned above. This section will focus on the use of lime alone or in combination with seawater.

In addition to the reaction between lime and alkalinity, we will get a reaction between lime and phosphate ions to hydroxyl apatite. This reaction will in simplified form be:

$$5\ Ca^{2+} + 4\ OH^{-} + 3\ HPO_4^{3-} \rightarrow Ca_5OH(PO_4)_3 + 3\ H_2O$$

The molar ratio between lime and phosphorus will be 5:3. The solubility of hydroxyl apatite decreases rapidly when pH increases. Increasing pH will give an increase in phosphorus removal. Nearly all phosphorus will be combined in hydroxyl apatite when pH exceeds 9.5. The settleability of hydroxyl apatite is not good, and to improve this, calcium carbonate and in some cases magnesium hydroxide, must be precipitated. The necessary lime quantity to precipitate phosphorus will vary to a great extent with the wastewater quality. Alkalinity is here the most important parameter.

If the lime dosage is too small to achieve high pH in low alkaline wastewater, well settleable flocs will be difficult to form. The reason is that the heavy calcium carbonate ($CaCO_3$) part will be too small, and pH has to be increased to >11.2. Help from precipitated magnesium hydroxide in addition to calcium carbonate will then give a satisfactory flocculation. The large calcium carbonate part in high alkaline wastewater will give good phosphorus removal by pH-values as low as 9.5-10.0.

The necessary lime dose to remove phosphorus from wastewater is not so dependent on the wastewater phosphorus content as with aluminium or iron salts. The lime process is very pH-dependant. Wastewater alkalinity will therefore primarily determine necessary lime quantity. Figure 8.6 shows the effluent phosphorus content and pH as a function of lime dosage.

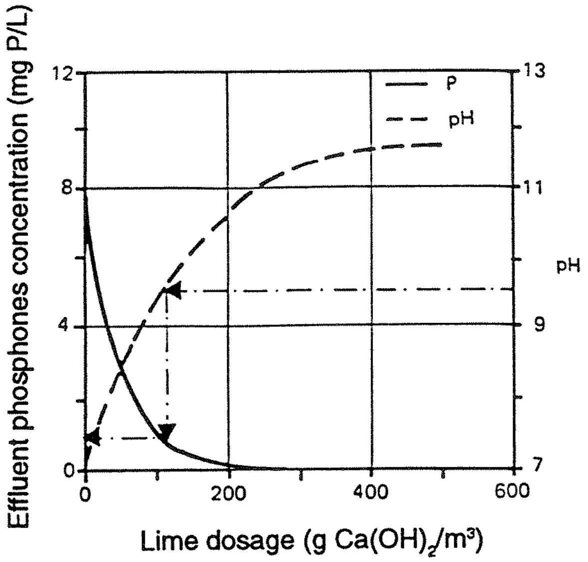


Figure 8.6 ***Typical diagram of phosphorus reduction by the use of lime with a given alkalinity in the influent wastewater.***

The alkalinity consumption will follow this equation:

$$Ca^{2+} + HCO_3 + OH^- \rightarrow CaCO_3 + H_2O$$

The wastewater magnesium content will also have a great impact on the removal rate. This can be written:

$$Mg^{3+} + Ca(OH)_2 \rightarrow Mg(OH)_2 + Ca^{2+}$$

By normal Norwegian wastewater magnesium content, this reaction will be nearly total by pH 11.2. Magnesium hydroxide flocs ($Mg(OH)_2$) are of gelatine character. These flocs adsorb colloids during sedimentation. Increasing magnesium content will give precipitation of magnesium hydroxide by a lower pH. In this way, proper flocculation can be achieved by lower lime dosage. At some Norwegian plants magnesium is added by mixing some seawater into the wastewater. The seawater magnesium content is normally high. Figure 8.7 shows how seawater addition (in percent of wastewater flow) will affect the phosphorus concentration of purified wastewater.

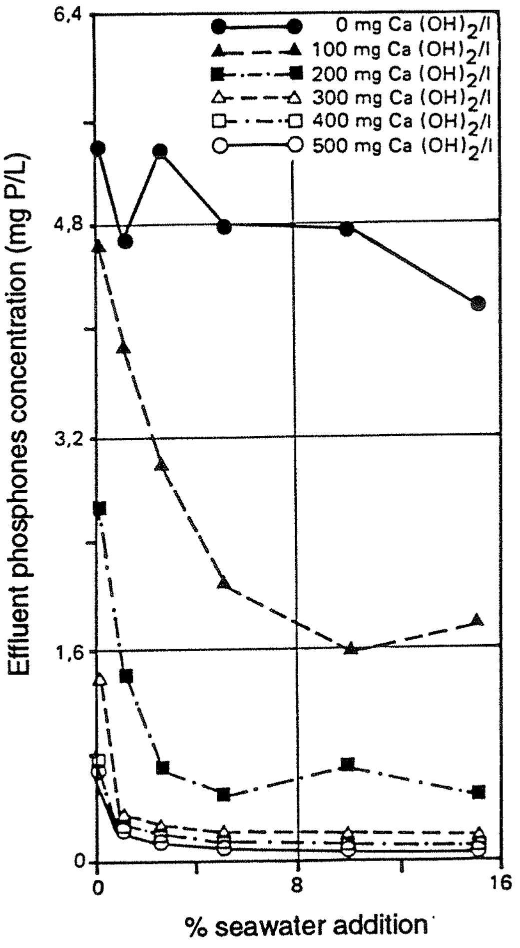


Figure 8.7 Total phosphorus reduction as a function of added seawater (in percent of wastewater flow) by different lime dosages.

8.2.2 Precipitants Dosing

Table 8.1 lists data on the most common precipitants.

Table 8.1 Common precipitants data and normal dosages.

Precipitant	**Composition**	**Supplied as**	**Density (t/m^3)**	**Normal dosing rate (g/m^3)**
Aluminium sulphate	$Al_2(SO_4)_3 \cdot nH_2O$	Granules	1	100-250
AVR	Al and Fe(III) sulphate	Granules	1	100-250
Ferric chloride	$FeCl_3$	Liquid	1.5	150-300
PIX	$FeCl(SO_4)$ $FeCl_3$	Liquid	1.4	150-300
PAX	Polymerised Al salts	Liquid	1.2-1.3	75-150
Sodium aluminate	$Na_2Al_2O_4$	Liquid	1.5	100-200
Ferrous sulphate*	$FeSO_4 \cdot 7H_2O$	Crystals	1.2	100-250
Lime	$Ca(OH)_2$	Fine-grained powder	0.6	250-500

* Only used in simultaneous precipitation.

The liquid precipitants are normally stored in closed chemical tanks with a dosing pump. Several types of pumps can be used, e.g. membrane, peristaltic, piston and helical rotor pumps. The storage tank normally contains at least one months supply of precipitant. Materials and equipment in contact with the precipitant should be made of non-corrosive materials, as most of the precipitants are extremely corrosive. Appropriate protective clothing must be worn when handling these chemicals.

Granulated or powdered precipitants are usually stored in sacks (small treatment plants) or silos. The precipitants are normally dosed into a continuous dissolver by a dosing screw, and the dissolved chemicals are conveyed by gravity to the dose point. There are several ways of controlling the chemical addition to the wastewater:

1. Time proportional dosing
2. Flow proportional dosing
3. pH feed back controlled dosing
4. Alkalinity feed forward controlled dosing

and combinations of the above methods. Time proportional dosing is constant addition of precipitants independent of the water flow. This is the most common way of adding precipitants by simultaneous precipitation, since the detention time in the aeration tanks is so long that changes in wastewater flow will have little effect on the precipitant dosing rate. Time proportional dosing can be varied during the day by using a timer. This means that the dosing rate may be larger in the middle of day than in the night. In this way the dosages can be adapted to normal flow variations through the plant.

By time proportional dosing one will not be able to handle the flow variations during rain storms. It is therefore more common to use flow proportional dosing of precipitants.

Dosing quantity in g/m^3 will then be constant independent of the flow. Generally, storm water alkalinity is less than wastewater alkalinity during dry weather flow. Reduced dosages in g/m^3 by increased flows are therefore preferred, otherwise precipitation pH may fall outside the optimum range. At some plants, flow proportional dosing rates vary throughout the day. This is because the wastewater is normally more concentrated in the middle of the day than in the middle of the night. Each plant should establish its own dosing scheme based on experience.

Precipitant addition can also be pH controlled. If the precipitation works best at pH 6.0, this can be the set point of pH controlled precipitant dosage. If the pH is higher than 6.0 after adding precipitant, the precipitant dosage will be increased by the pH-controlled dosing equipment. If pH is lower than 6.0, the dosing rate will be reduced. The greatest problem associated with this method is that the pH-electrodes will normally be inaccurate after some time in wastewater and the precipitant dosages can be strongly affected by inaccurate pH measurements. This kind of precipitant dosing control therefore requires accurate operation and maintenance of the pH equipment.

Alkalinity feed forward control of precipitant addition is another way of adding precipitants in a way that maintains a stable pH. Alkalinity cannot be measured continuously, but in wastewater with little industry inflow, wastewater conductivity is often proportional to the alkalinity. If this is the case, conductivity can be used in addition to flow as a dosing control parameter. In Norway, the experiences with adding precipitants by this equation in small wastewater treatment plants are good:

$$D = A\sqrt{Q \cdot C}$$

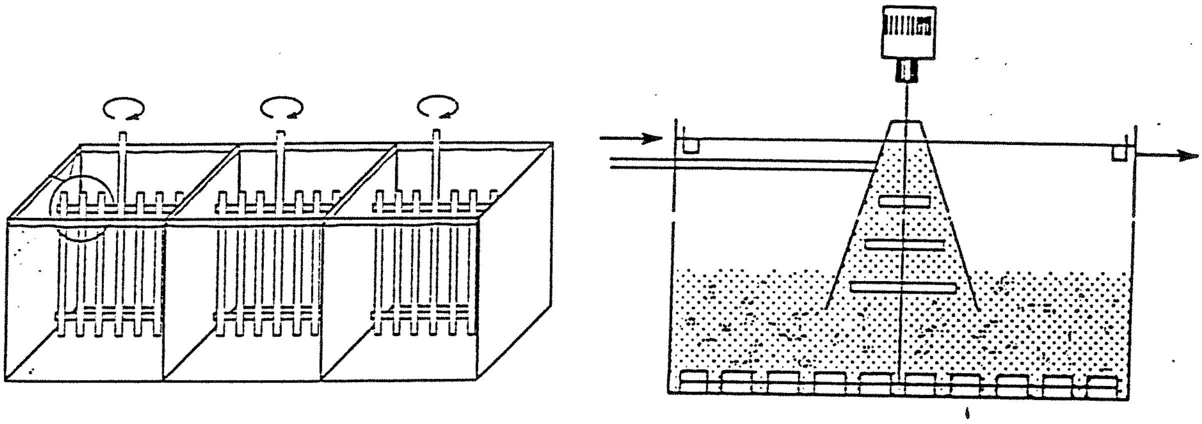
D = Precipitant dosing rate (l/h or kg/h)
A = Constant, dependant on wastewater quality and type of precipitant
Q = Wastewater flow (m^3/h)
C = Wastewater conductivity (mS/m)

8.2.3 Coagulation

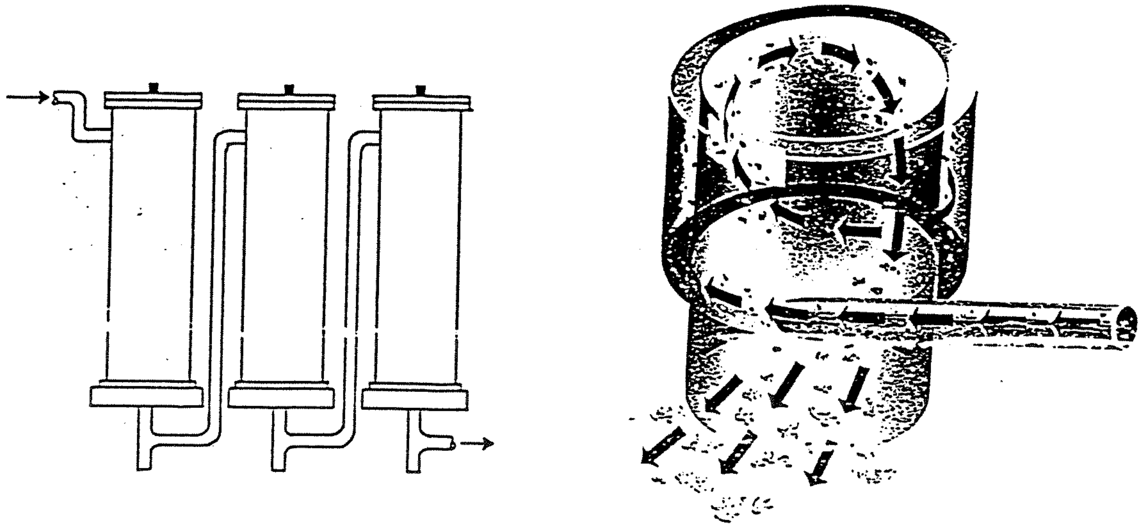
The reaction time between precipitant and wastewater phosphorus is extremely short, less than 0.1 second. Uniform precipitant distribution throughout the entire waterbody is therefore important to get as many phosphate ions as possible in contact with the precipitant immediately. This can be done in many ways, e.g. by waterfalls, injection or mixing systems. Lime needs long mixing time to be satisfactorily dissolved.

8.2.4 Flocculation

After the coagulant phase, flocculation of particles must take place as soon as possible so that the built-up microflocs are not damaged by turbulent flow. An important aspect in flocculation units is to achieve a good sweep coagulation in order to form large flocs that can easily be separated from the water. Flocculation can take place in traditional flocculation chambers, in tube flocculators or in a cylinder system (see Figure 8.8). It is also possible to use aerated grit chambers or activated sludge tanks (simultaneous precipitation) for this purpose, but this is not optimum conditions for flocculation.



Paddle agitators in separate chambers Agitators in influent cone in sedimentation tank



Tube flocculation Flocbee

Figure 8.8 Different types of flocculation units.

Different types of separation equipment require different types of flocs. Flocs to be removed by gravity need to be large and heavy, and therefore mixing intensity is decreased from the dosing point to the settling tanks. It is important that wastewater leaving the flocculation process is not subjected to greater shear forces than encountered in the flocculation step. Therefore the flocculation unit should be situated directly adjacent to or in the settling tank. The last chamber should be built wide open towards the settling tank.

8.2.5 Sedimentation

For chemical precipitation plants, sedimentation is recommended as the separation unit after flocculation. The depth of the settling tank should be at least 2.5 m, and the surface loading rate should not exceed 0.6 $m^3/m^2 \cdot h$. Sedimentation is more thoroughly described in Chapter 5.

8.3 Process Control

Table 8.2 shows which observations, measurements etc. that are specific to the chemical precipitation process.

Table 8.2 Process control of chemical precipitation.

Item	Observations	Measurements and analyses	Calculated values
Influent	•Appearance and odour	•Alkalinity •pH •Total phosphorus •Orthophosphate •Suspended solids •BOD_5 •COD	
Dosing equipment	•Mechanical function •Dissolver •Clogging •Corrosion	•Stock of precipitant •Concentration of solutions •Dosing rate	•Molar ratio
Flocculation	•Floc formation •Floatables •Deposits	•pH •Alkalinity •Ortho phosphate	•Detention time
Settling tank	•Sludge accumulation •Floatables •Flow conditions	•Secchi depth	•Surface loading rate
Effluent	•Appearance	•Alkalinity •pH •Settleable solids •Total phosphorus •Ortho phosphate •Residual precipitant •Suspended solids •BOD_5 •COD	•Removal rates

8.3.1 Influent

The quality of the influent to a precipitation process depends on the type of chemical treatment plant. Primary precipitation and pre-precipitation influents will be preliminary treated, secondary precipitation influents will be primarily treated, and post-precipitation influents will be biologically treated.

Appearance and Odour

Changes in odour, colour and visual quality should be observed continuously. In some cases changes will cause problems in the precipitation process.

Alkalinity

Influent alkalinity shows the wastewater buffer capacity. Wastewater alkalinity depends on potable water alkalinity, storm water alkalinity, and infiltration water alkalinity. Low potable water alkalinity (<0.5 meq/l) will give wastewater alkalinity of about 1-3 meq/l. Where potable water alkalinity is higher, wastewater alkalinity increases correspondingly.

The alkalinity of the influent to the precipitation process determines the process pH with a given precipitant dosage. Direct and pre-precipitation influent alkalinity will vary during the day. Biological treatment prior to post-precipitation will reduce the variations. Sewerage system conditions, like storm water and seasonal changes in infiltration water, can affect the wastewater alkalinity strongly; recycled sludge liquor the same. Especially lime addition or anaerobic sludge stabilisation (digestion) will give high alkalinity sludge liquor.

At post-precipitation plants, nitrification in the biological treatment step can affect precipitation influent alkalinity, as well. Nitrification will decrease alkalinity. Every g/m^3 nitrogen that is oxidised to nitrate in the biological treatment step is assumed to decrease alkalinity by 0.1-0.14 meq/l.

pH

Influent pH has no informative value as long as it is in a normal range (6.7-7.5). If pH exceeds this range, abnormal alkalinity values may occur.

Total Phosphorus

Influent concentration of total phosphorus is of minor value for the process control. We need the parameter to calculate the removal rate for the chemical precipitation step.

Ortho Phosphate

Ortho phosphate is the phosphorus part that precipitates by adding precipitant. We need the influent ortho phosphate concentration to assess the necessary precipitant dosage.

Suspended Solids

Influent suspended solids content reflects the performance of upstream treatment processes, but is of little value for process control. If sludge liquor is added to the wastewater just before the precipitation step, suspended solids analysis is valuable. This analysis may show the effects of sludge liquors on the influent quality.

Organic Matter (BOD_5 and COD)

Influent organic matter (BOD_5 and COD) analyses are of little value for the precipitation process control purposes. The parameters can be used to calculate organic matter removal rates in the precipitation step.

Some plants are influenced by heavy loads of dissolved organic matter from industrial discharges (food industry). This may disturb the precipitation process and influent analyses of BOD_5 and COD will be necessary.

8.3.2 Dosing Equipment

Mechanical Function

Daily observations of the dosing equipment performance and precipitant transport to the dosing point are essential. How to control the functionality depends of the type of equipment. Follow carefully the specifications given in the operation and maintenance manual from the supplier.

Dissolver

Granular or powdered precipitants must be dissolved before addition to wastewater. Non-soluble or nearly non-soluble contaminants will occur in precipitants. These contaminants may settle in the dissolver. Cleaning on a routine basis is therefore essential.

Clogging

The possibility of pipe clogging between dosing equipment and dosing point is always present. The possibility is greatest when using aluminium sulphate or lime.

To reduce the possibility as much as possible, pipes should have no sharp bends or low points. Precipitant velocity must be higher than 0.5 m/s. Precipitant concentration must not be too low, as precipitant impurities precipitate easier by low concentration.

To further reduce clogging, the dosing equipment should be situated near the dosing point, pipes should be multiple, or precipitants should be transported in an open V-shaped channel.

Corrosion

Precipitants like PIX, PAX, ferric chloride and aluminium sulphate are extremely corrosive to iron and concrete when concentrated. Also iron sulphate solutions may be corrosive.

Parts of the plant that has been in contact with concentrated precipitants must be closely observed.

Stock of Precipitant

Amount of precipitant in silos or tanks and the precipitant consumption (kg/day) must be controlled regularly to ensure uninterrupted operation and that new orders can be made in time.

Dosing Rate

Precipitant dosing rate and dosing equipment should be controlled regularly. By flow proportional dosing, flow meter control is essential (see Chapter 3.2).

Example: The flow meter in a plant shows 80 l/s. PIX quantity measured for 2 minutes is 1520 ml. What is the dosing rate?

2 minutes = 120 seconds
80 l/s · 120 s = 9600 litre = 9.6 m^3

Dosing rate: $\frac{1520 \text{ ml}}{9.6 \text{ m}^3} = 158 \text{ ml/m}^3$

By pH or alkalinity/conductivity controlled dosing the dosing rate in g/m^3 will vary with time. Regularly dosing metering will tell whether or not the dosing rate is normal. Precipitant dosages in kg/day and calculated dosing rates from daily water flow (g/m^3) will, in these cases, provide useful information. Precipitation dosages should be frequently measured from the dosing equipment. By use of dosing screws, this can be expressed as grams per revolution (g/rev). By liquid dosing, the expression used is ml per stroke of the pump, by constant stroke length. Deviation from set points should be examined. There are many possible causes to these deviations:

Dosing screw:	Screw is worn, storage tank outlet partly blocked, counter failure.
Liquid dosing:	Assembly failure (storage tank level influences pumping volume), valve function failure, counterpressure valve failure.

Molar Ratio

The molar ratio between the active part of added precipitant and the orthophosphate content of the wastewater used to calculate necessary dosing rate.

To calculate molar ratio, we must know the content of the active part in the different precipitants, and orthophosphate concentration of wastewater. Content of active part in the most common precipitants is given in Table 8.3.

Precipitant	**Content of active part (mol/kg)**
PIX	2.1
PAX	3.0
Aluminium sulphate	3.3
AVR	3.2
Ferric chloride	3.0
Ferrous sulphate, mono hydrate	5.3
Ferrous sulphate, hepta hydrate	3.2

Molar ratio can be calculated the following way:

$$\text{Molar ratio (MR)} = \frac{31D \cdot AP}{1000OP}$$

D = Precipitant dosing rate (g/m^3)
AP = Active part of precipitant according to table above (mol/kg).
OP = Ortho phosphate concentration of precipitation influent ($g\ P/m^3$)

Necessary dosing rate can be calculated by reshaping this equation:

$$D = \frac{1000MR \cdot OP}{31AP}$$

Examples: Molar ratio control:

At a precipitation plant using PIX, normal dosing rate is about 150 ml/m^3. In a dry weather flow period, the influent flow decreases and influent orthophosphate concentration increases to about 8 $g\ P/m^3$. Is the molar ratio Fe/P still sufficient?

$$\text{Molar ratio} = \frac{150 \cdot 2.1 \cdot 31}{1000 \cdot 8} = 1.22$$

Molar ratio is probably too low to precipitate all the ortho phosphate. The orthophosphate concentration in a filtered sample from the flocculation tank should be analysed.

Necessary dosing rate

Influent ortho phosphate analysis gives a concentration of 7.8 $g\ P/m^3$ during dry weather flow. In storm weather periods, the concentration decreases to 4.6 $g\ P/m^3$. How much PIX do we have to add?

Normal molar ratio is approximately 2.

$$\text{Dry weather dosing rate} = \frac{1000 \cdot 2 \cdot 7.8}{2.1 \cdot 31} = 240\ g/m^3$$

$$\text{Storm weather dosing rate} = \frac{1000 \cdot 2 \cdot 4.6}{2.1 \cdot 31} = 141\ g/m^3$$

Control of Dosing Rate

We have earlier shown how to calculate necessary dosing rate by iron and aluminium precipitation. Lime precipitation is different, see section "use of lime".

Precipitant dosing rates should be as small as possible. In this way we can reduce the chemical costs, sludge production will be lower, and the sludge dewaterability will be better (by use of aluminium and iron salts).

Chemical precipitation is strongly pH-dependant. Aluminium salts give best results at pH-values about 6.0, and iron salts about 5.5 or lower. (This is not valid by simultaneous precipitation, see Chapter 7). Prepolymerised aluminium salts (PAX) has normally a wider optimal pH-range.

Optimum pH-value can vary from plant to plant. Post-precipitation plants usually has a wider pH-range than direct precipitation and pre-precipitation plants. High phosphorus removal rates of pre-precipitation plants are not always favourable and may disturb downstream biological degradation, which needs phosphorus for the metabolism.

Wastewater alkalinity and precipitant dosage rate determine pH-value by iron and aluminium salt precipitation. By lower alkalinity and higher precipitant dosage, the pH-value will decrease. The approximate reduction in alkalinity due to the precipitant dosage can be calculated from the nomogram in Figure 8.9.

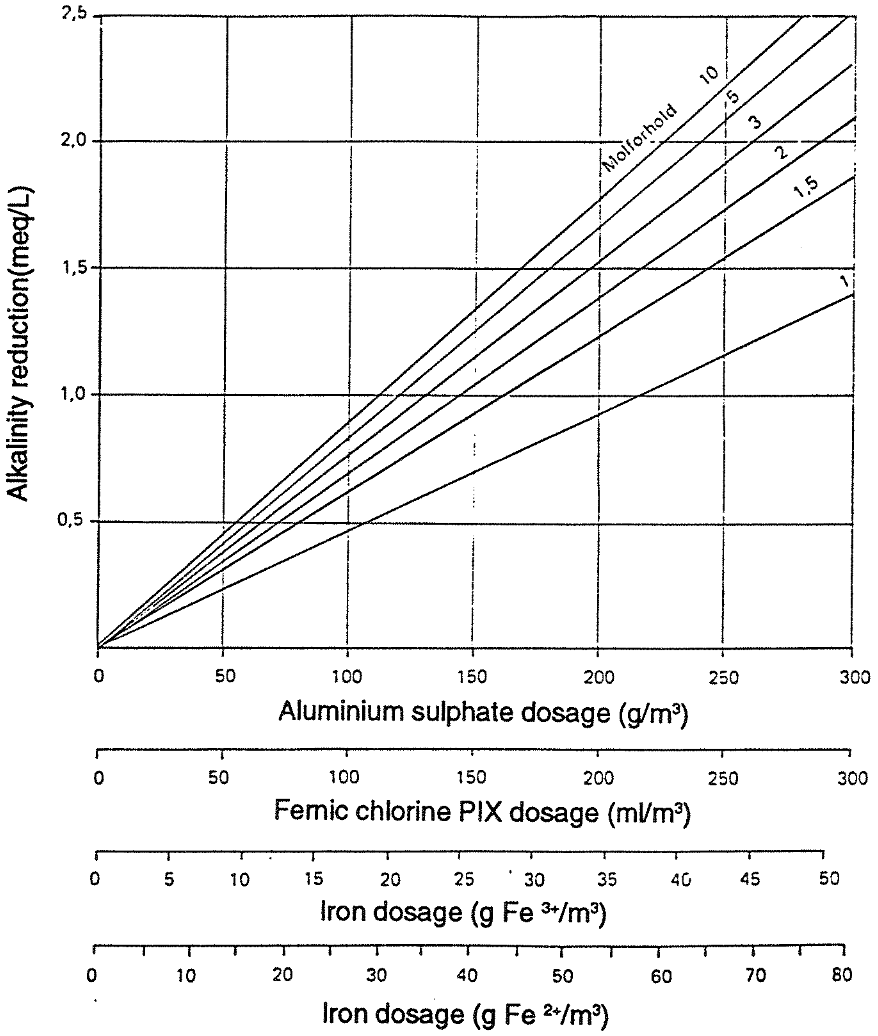


Figure 8.9 Alkalinity reduction due to precipitant dosing.

By direct precipitation, the wastewater alkalinity is normally high enough to govern the precipitant dosage. This can be rephrased: When sufficient precipitant is added to obtain the correct pH-value, the dosing rate is sufficient to precipitate all the phosphates as well. Exceptions include periods of extreme groundwater infiltration or storm water inflow. Alkalinity may then decrease so much that necessary dosing rate to precipitate phosphates will give a pH-value below the optimum range. Wastewater alkalinity after precipitant addition should be at least 0.3 meq/l.

By direct precipitation, the wastewater colloids content is often high. These materials must also be coagulated to form strong flocs. This means that in some cases dosing quantity must be increased more than what is necessary to precipitate phosphorus and to achieve an optimum pH-range.

Nitrification in the biological step can cause problems in post-precipitation by use of iron or aluminium salts. Nitrification decreases the wastewater alkalinity. The rest alkalinity is sometimes too low to avoid low pH when precipitant is added. To avoid problems with too low pH-values, alkali should be added either to the biological step or to the precipitation step. Sodium bicarbonate ($NaHCO_3$), sodium carbonate (Na_2CO_3), or slaked lime ($Ca(OH)_2$) may be used. If wastewater alkalinity is high enough, nitrification is beneficial, because the amount of precipitant needed to obtain the optimum pH-value will be less.

Examples: 1. Analysis of the influent concentration of ortho phosphate and alkalinity shows:
In dry weather: 7.8 g P/m^3 and 3.2 meq/l
In storm weather: 4.6 g P/m^3 and 1.3 meq/l
Will this wastewater reveal large pH-drops with the normal dosing rates of PIX?

Dry weather conditions:
Necessary dosing rate from the last example: 240 g/m^3
Molar ratio = 2

Influent alkalinity:	3.2 meq/l
Alkalinity decrease (Fig. 8.9)	-1.7 meq/l
Rest alkalinity:	1.5 meq/l

Storm weather conditions:
Necessary dosing rate from the last example: 141 g/m^3
Molar ratio = 2

Influent alkalinity:	1.3 meq/l
Alkalinity decrease (Fig. 8.9)	-1.0 meq/l
Rest alkalinity:	0.3 meq/l

Rest alkalinity if dosing rate is not reduced: 0 meq/l!

Conclusion:
Dosing rates must be adjusted according to influent quality. The example shows that dry weather dosing rate will give 0 alkalinity during storm weather periods, and this will cause serious problems with pH in the precipitation step.

2. At a post-precipitation plant effluent alkalinity of the biological step is 1.2 meq/l, and ortho phosphate concentration is 7 g P/m^3. How much PIX is necessary to achieve a molar ratio Al/P of 2? Is the wastewater alkalinity high enough to ensure optimum pH-value?

$$\text{Dosing rate} = \frac{1000 \cdot 2 \cdot 7}{2.1 \cdot 31} = 215\ \text{g/m}^3$$

A PIX quantity of 215 g/m^3 by a molar ratio 2 gives, according to Figure 8.9, an alkalinity decrease of 1.5 meq/l. pH will be too low, and a lower PIX dosage must be tried. If this does not give a satisfactory phosphorus removal, alkali must be added.

8.3.3 Flocculation Tanks

Floc Formation

Floc formation should be controlled daily. Large flocs (1-5 mm) are preferred. The water among the flocs should be clear. Agitation in flocculation tanks is of great importance with respect to floc formation. Increased agitation rate, increases the number of particle collisions. This will reduce the amount of small flocs. Too high agitation rate, on the other hand, can disrupt large flocs. Recommended peripheral velocities with paddle type agitators are about 0.5 m/s in the first flocculation tank, gradually reduced to about 0.1 m/s in the last tank, if downstream separation is sedimentation.

Floatables

Some foam or floatables formation are normal.

Deposits

Deposits at the bottom of flocculation tanks can be a problem. Primary precipitation plants are particularly exposed. Sand and coffee grounds that pass the grit chamber tend to settle in flocculation tanks and should be removed before any problems occur.

pH

pH-value is essential both for the phosphate precipitation (coagulation) and for floc formation of precipitated materials. The pH-value in the flocculation tanks should be controlled frequently by grab sampling to ensure an optimum pH-range. pH can also be monitored continuously. In that case, the pH-value should, in addition, be controlled by other equipment, to control the continuous pH-meter. By continuous pH metering, it is critical that the electrode are kept clean. The pH-meter must be calibrated routinely.

Normal pH-values for good precipitation results are shown in Table 8.3. By simultaneous precipitation, these values are not valid (see Chapter 7).

Table 8.3 Optimum pH-range for different precipitantsby iron and aluminium precipitation (not by simultaneous precipitation).

Precipitant	Optimum pH-range	Comments
Aluminium sulphate	5.5 - 6.3	
Iron(III) salts	5.0 - 6.0	By post-precipitation, there are examples of good results also in the range 6.0 - 8.5.
Iron(III) salts + lime	> 8.5	
Iron(II) salts + lime	> 8.5	
Lime	>11.2	Wastewater alkalinity <8 meq/l and low magnesium content
	>10.6	High magnesium content (Mg >20 mg/l)
	>10.0	Wastewater alkalinity >8 meq/L

To decrease the pH, the dosing rates of aluminium or iron salts should be increased. To decrease the pH by lime precipitation, we can reduce the lime dosage.

Increased pH values can be obtained by decreasing the iron or aluminium quantities. By lime precipitation, we can increase pH by increasing the lime dosing rate.

When changing the precipitant dosage in order to change the pH, it is important to notice that the molar ratio will be changed as well. This can affect the phosphorus removal rate.

Alkalinity

Alkalinity control downstream the flocculation unit can be a good basis for assessing the dosing rate. If alkalinity is less than about 0.3 meq/l, large pH variations will probably cause poor phosphorus removal rates.

Ortho Phosphate

Ortho phosphate should be analysed on filtered samples to control the phosphorus precipitation. A grab sample should be taken and analysed as soon as possible to avoid the need for conservation. By lime precipitation, the filtered sample should be neutralised before analysis.

Very good ortho phosphate precipitation (PO_4-P < 0.1 g P/m^3) does not guarantee good phosphorus removal. This will also depend on the efficiency of the downstream separation process. If, on the other hand, the ortho phosphate precipitation is poor (PO_4-P > 0.3 g P/m^3), it will be very difficult to obtain a good phosphorus removal (Total P < 0.5 mg/l).

Detention Time

The detention time in the flocculation tanks is of importance for the flocculation results. Detention time, t, is calculated from:

$$t = \frac{V}{Q}$$

t = Detention time, hours
V = Volume of flocculation tanks, m^3
Q = Wastewater flow, m^3/h

Too long detention time (> 1 hour) can be just as bad as too short detention time (< 10 minutes). The detention time should be about 10 - 20 minutes by normal dry weather flow. In dry summer situations, it can be necessary to take flocculation tanks out of operation if the detention time is so long that flocs are destroyed by agitation.

8.3.4 Settling Tank

Sludge Accumulation

Sludge blanket level and sludge accumulation control of settling tanks state the function of sludge pumping routines and sludge scrapers operation. Sludge accumulation at the bottom of the tanks may cause increased suspended solids concentration in the effluent. This is caused by erosion from the bottom sludge blanket. If the sludge detention time is too long, the sludge may decompose and form odorous floatables. If the sludge blanket is too thick, the scraper equipment will be heavily loaded. In flat-bottomed tanks, the sludge blanket should not be thicker than about 10 cm. In sludge hoppers, the sludge blanket level should not be higher than 1 - 1.5 m below the water surface.

Flow conditions

Turbulent flow or uneven loading of a settling tank or uneven distribution between multiple settling tanks may cause poor sludge separation.

Floatables

Floatables in the settling tanks are removed when necessary. Floatables may cause odour.

Secchi Depth

Normally, the relation between secchi depth and removal rate is reliable. Secchi depth is therefore a good measure of the separation efficiency.

The following scale can be a good help in assessing the secci depth measurements in primary-, secondary- and post-precipitation plants:

Secchi depth (cm)	Scale
> 120	Good
60 - 120	Acceptable
< 60	Bad

The precipitation step in pre-precipitation plants may function acceptable in spite of low secchi depths. Low secchi depth is a sign of bad floc formation or poor sedimentation.

Surface Loading Rate

This is the most important loading parameter of a settling tank. The surface loading rate is described in Chapter 5.3.2. By chemical precipitation, the surface loading rate should not be greater than the values in Table 8.4, unless polymer is added to increase the floc settleability.

Table 8.4 ***Maximal surface loading rate by chemical precipitation.***

Precipitant	**Water depth in settling tank (m)**	**Surface loading rate ($m^3/m^2 \cdot h$)**	
		design flow	**maximum flow**
Aluminium, iron or iron + lime	2.5	1.0	1.6
	≥3.0	1.3	2.0
Lime	2.5	1.3	1.9
	≥3.0	1.6	2.4

8.3.5 Effluent

Appearance

Appearance should be observed regularly. Changes may indicate bad functioning of treatment processes.

Alkalinity

If alkalinity is not measured in the flocculation tanks, it should be measured in the effluent. For assessment, see Chapters 8.3.2 and 8.3.3.

pH

pH-analyses of daily composite samples are of little value by aluminium or iron precipitation, since the values are often misleadingly high. Analyses of grab samples from the flocculation tanks are more reliable.

Total Phosphorus

The objective of chemical precipitation is most commonly to remove phosphorus. Total phosphorus content in the effluent should be monitored regularly, e.g. by analyses of daily composite samples. We can use the following scale to assess the results:

Pre-precipitation* (g P/m³)	**Direct precipitation (g P/m³)**	**Post-precipitation (g P/m³)**	**Assessment**
< 1	< 0.5	< 0.4	Good
1 - 2	0.5 - 1.0	0.4 - 0.7	Acceptable
> 2	> 1.0	> 0.7	Bad

* By pre-precipitation, too good phosphorus removal can affect the downstream biological step. Even with poor total phosphorus removal in the precipitation step, the final effluent quality will often be good

High phosphorus content can be caused by bad phosphorus precipitation, bad flocculation, bad sedimentation, or a combination of these causes.

Orthophosphate

If orthophosphate is not analysed regularly in flocculation tank samples, it should be analysed on effluent samples. Assessment of the results is the same as for flocculation tanks:

Orthophosphate ($g P/m^3$)	**Scale**
< 0.1	Good
0.1 - 0.3	Acceptable
> 0.3	Bad

The difference between total phosphorus and ortho phosphate shows suspended phosphorus concentration. This parameter can be compared to effluent suspended solids.

Residual Precipitant

With an optimum precipitant dosing rate, only small amounts of dissolved precipitant should leave the plant with the effluent. This should be controlled now and then by analysing effluent residual iron or aluminium content.

Monitoring the "complete" precipitation of the added precipitants is done by analyses of aluminium or iron on filtered grab samples. Lime analysis is less common. The results should be:

Sample	**Al-sulphate ($g Al/m^3$)**	**Fe-salts ($g Fe/m^3$)**
Unfiltered	< 1	< 2
Filtered	< 0.1	< 0.2

Settleable Solids

Effluent settleable solids are a measure of settling tank separation efficiency. The Secchi depth is more reliable than settleable solids in determining solids removal by chemical precipitation. Only if the settling tanks are overloaded or the precipitation step does not function at all, we will find settleable solids in the Imhoff glass.

Suspended Solids

Effluent suspended solids concentration is a result of both the flocculation and the sedimentation units. Effluent suspended solids should therefore be measured regularly. The results should be assessed like this:

Suspended solids (g/m^3)	**Scale**
< 20	Good
20 - 30	Acceptable
> 30	Bad

The main precipitation effluent phosphorus part is normally the suspended one. The effluent suspended solids concentration is an approximate measure of the phosphorus concentration.

Organic Matter

By primary-, secondary- and post-precipitation, effluent organic matter is analysed as often as required by the authorities. By pre-precipitation, precipitation effluent BOD_5 should be analysed regularly. The BOD_5 and COD analyses can be assessed this way:

Primary and secondary precipitation		**Post-precipitation**		**Scale**
BOD_5 ($g\ O_2/m^3$)	**COD ($g\ O_2/m^3$)**	**BOD_5 ($g\ O_2/m^3$)**	**COD ($g\ O_2/m^3$)**	
< 50	< 100	< 10	< 40	Good
50 -80	100 - 150	10 - 20	40 - 60	Acceptable
> 80	> 150	> 20	> 60	Bad

Removal of organic matter is by primary and secondary precipitation plants closely linked to influent quality (content of dissolved organic matter). Chemical precipitation cannot remove dissolved organic matter, and this should be considered when the results are assessed.

8.4 Troubleshooting

By all types of operational problems, it is essential to find the cause(s) by a systematic approach. Tables 8.5, 8.6 and 8.7 list the most common problems associated with flocculation, settling tanks and precipitation effluent, and the corresponding possible causes and solutions.

8.4.1 Flocculation Tank Problems

Table 8.5 Problems in the flocculation tank.

Problems	Probable causes	Solutions
Floatables	• Air bubbles lifting sludge to the surface.	• Look for air suction whirls or weirs
	• Decomposed deposits form black floatable cakes	• See "Deposits"
Deposits	• Insufficient preliminary or primary treatment	• By direct or pre-precipitation plants the preliminary or primary treatment can be insufficient. Control the performance of screens, grit chambers and primary settling tanks
	• Undissolved precipitant	• Control precipitant dissolver function
		• Control that precipitant dose point has good mixing conditions. Alternatively increase agitation velocity in the first flocculation tank.
	• Low agitation velocity	• Low velocity of agitators may cause sedimentation in the flocculation tanks. Try to increase the velocity, but be aware of destroyed and poor removal rates.

8.4.2 Settling Tank Problems

Table 8.6 Problems in the settling tank.

Problem	Probable causes	Solutions
Floatables	• Air bubbles lifting sludge to the surface.	• Look for air suction whirls or weirs. Reduce or stop channel aeration, if applied
	• Decomposed deposits	• Increase sludge pump starting frequency • Increase scrapers velocity or scrapers running time
	• Sludge accumulation in corners or at low gradient walls	• Regularly scrape sludge down by poling
Low sludge solids concentration	• Too little time to thicken the sludge in sludge hoppers	• Reduce sludge pump starting frequency
	• Water follows sludge during removal periods	• Decrease sludge pump running time and rather increase starting frequency
	• Too high sludge solids concentration can form channels in sludge, so that sludge pump sucks water	• Scrape sludge to the pump and increase pump starting frequency
Difficult to remove sludge	• Too high sludge solids concentration	• Increase sludge pump starting frequency
	• Sand, coffee grounds or rags in the sludge	• Control the preliminary treatment processes

8.4.3 Settling Tank Effluent

The removal rate requirements for all parameters will usually be met if the phosphorus removal requirements are met. (One exception may be organic matter, but the cause of this will normally not be found in the chemical treatment step). Reasons for bad removal rates may be one of the following:

- Bad precipitation
- Bad flocculation
- Bad separation, i.e. bad sedimentation or flotation

The cause(s) can be found performing certain by analyses and by using the below table:

Results	**Cause**
Ortho phosphate in filtered sample > 0.1 g P/m^3	Bad phosphorus precipitation
Suspended solids > 20 g/m^3 and Settleable solids < 0.3 ml/l	Bad flocculation
Settleable solids > 0.3 ml/l	Bad separation

Table 8.7 Problems affecting precipitation step effluent.

Problems	**Probable causes**	**Solutions**
Bad phosphorus precipitation	• Too low dosing rate	• If molar ratio Al/P or Fe/P is less than 2, increase dosing rate. After increasing dosing rate, control pH.
	• Too low pH-value after precipitant addition	• By Al or Fe precipitation: Decrease the dosing rate. Control that molar ratio is not too low. If these measures are not sufficient, add sodium bicarbonate, soda or lime.
		• By lime precipitation: Increase dosing rate.
	Too high pH-value after precipitant addition	• By Al or Fe precipitation: Increase dosing rate.
		• By lime precipitation: Too high pH will normally not affect removal rates, but operation costs will be higher than necessary and the recipient may be affected. Decrease dosing rate.
Bad flocculation	• pH-value outside optimum range	• Adjust dosing rate according to "bad phosphorus precipitation" above.
	• Insufficient coagulation (Al-/Fe precipitation)	• Increase dosing rate and control that pH is not too low.
		• Add polymers to the flocculation tank.
	• Insufficient mixing of precipitant and wastewater	• Control dosing point. Add precipitants to a point with intensive mixing.
		• Increase agitation velocity in the first flocculation chamber

Table 8.7 ***Problems affecting precipitation step effluent (continued).***

Problems	Probable causes	Solutions
	• Hydraulic overloading in flocculation tank	• Total detention time should be not less than 10 - 15 minutes.
	• Long detention time in flocculation tank	• Too long detention time (more than 1 hour) may damage the flocs.
	• Short-circuiting in flocculation tank	• Short-circuiting may cause the real detention time to be too short even if the flocculation tanks are big enough. Baffles or reversed agitator rotation direction may eliminate short-circuiting.
	• Floc destruction	• Control wastewater velocity in narrow passages, weirs, sharp bends or other places with strong turbulence.
		• Wastewater velocity in pipes and channels should be less than 0.2 m/sec, and preferably less than 0.1 m/sec.
		• Decrease aeration of channels between flocculation and settling tanks
		• Agitators rotating too fast, especially in the last flocculation chamber. Flocculation by air is often too intensive.
Bad separation	• Inlet pump capacity greater than acceptable surface loading rate	• Adjust influent pump capacity to the real settling tank surface loading capacity
	• Hydraulic overloading of the settling tank	• Increase particle settling velocity by adding polymers
		• Overflow some of the wastewater. This solution must only be used if the total removal rate is improved
	• Erosion of sludge blanket	• Let sludge scrapers run continuously. Control that scraper velocity is less than ca. 0.5 m/min
		• Avoid bottom streams by adjusting inlet and outlet devices.
	• Turbulent flow regime in settling tank	• Control the wastewater distribution between multiple settling tanks
		• Control outlet devices adjustment (outlet channel weirs should be adjusted to accurate level)
		• By bad shape of tanks, or inlet and outlet devices, general advice cannot be given.

9 Sludge Thickening

9.1 Introduction

The primary purpose of thickening is to reduce the volume of sludge that will be treated in subsequent processes. For example, increasing the solids concentration of a sludge stream from 0.5% to 4% will decrease its volume eightfold. The volume reduction benefits subsequent treatment processes such as digestion and dewatering. Thickening also reduces the required capacity of downstream tanks and equipment, the heat required by digesters, and the volume of sludge to be transported and finally disposed of. Three methods used to thicken sludges at various points in the treatment scheme are described briefly below, but only gravity thickening is discussed in more detail since this process is dominating in most countries:

- Gravity thickening concentrates sludges through the use of gravitational force. The most common configuration for gravity thickeners is circular. Rectangular systems, however, are also used. Gravity thickening can be aided by mechanical stirrers (pickets).

- Flotation thickening concentrates sludges through the attachment of microscopic bubbles (most commonly air) to sludge particles, thereby reducing their specific gravity to less than that of water. The air-attached particles then float to the surface for removal. This method is particularly well suited to wastes containing high amounts of finely divided solids (for instance waste activated sludge). The most common thickener in this category is the dissolved air flotation (DAF) thickener.

- Mechanical thickening concentrates sludges through the enhancement of gravitational forces. This can be accomplished in several ways:

    1. Centrifugal thickening uses centrifugal force to accelerate solid and liquid fraction separation. The applied force causes suspended solids to migrate through the suspending liquid, toward or away from the rotation axis depending on the difference in densities between the liquid and solid phases.
    2. Gravity belt thickening concentrates solids through the gravity separation of water and solids on a continuously moving porous horizontal belt.
    3. Rotary drum thickening concentrates solids through the use of a rotating perforated drum with a continuous internal screw (or diverted angle flights) to enhance the separation of water and solids.

Conditioning agents, usually polyelectrolytes (polymers) specific to the waste stream being treated, are commonly required to enhance the separation of free water from the charged sludge particles, especially when using flotation and mechanical thickening. Conditioning agents are discussed in Chapter 10.

9.2 Description of Process (Gravity Thickening)

Gravity thickeners can be operated both as a continuous (or semi-continuous) and batch process, the latter being practised mainly in small wastewater treatment plants. Further presentation is therefore devoted to the continuous operation of gravity thickeners.

Gravity thickening characteristics vary with the nature of the solids being thickened. Settling and thickening will occur in different modes depending on the concentration and flocculant nature of the solids being handled. Three basic types of settling comprise discrete settling, hindered or zone settling, and compression or compaction settling.

- Discrete settling is characteristic of very low solids loading rates, and for this reason does not apply to most sludge gravity thickeners.

- Zone settling occurs as the loading rates increase and individual particles become increasingly influenced by neighbouring particles. Particles maintain position relative to one another as they settle, forming a porous matrix supported by the fluid displaced as the solids settle. Zone settling is characterised by an easily discerned interface between the solids-laden liquor and adjacent supernatant. The solids concentration, fluid viscosity, and rate of underflow withdrawal dictate the zone settling rate.

- Compression settling occurs when flocculant particles develop a structure in which they are supported by adjacent particles rather than the displaced fluid. The compression settling rate is determined by channel formation in the floc structure that allows water to escape with consequent subsidence of the sludge mass.

Prevailing theory on gravity sludge thickener design is based on zone settling, with the settling rate ideally depending on solids concentration alone. Thus, solids loading rate or solids flux is also assumed to depend on solids concentration alone. Although this assumption may not apply exactly to wastewater sludges, particularly those with high solids concentrations, the effect of concentration nonetheless overwhelms the effects of other parameters.

Downward solids transport in a gravity thickener stems from settling because of gravity and bulk movement resulting from pumping sludge from the tank bottom. The sum of these two transport mechanisms provides the total solids flux.

Supernatant from a gravity thickener recycles through the treatment facility. Supernatant clarity is maintained to limit the recycled solids and associated BOD through the plant to a minimum, thus limiting increased system solids and organic loading rates. Overflow from a smoothly operating gravity thickener, with typically less than 200 mg/l suspended solids, may be recycled directly to the secondary treatment process. Supernatant from a gravity thickener with operational difficulties should be recycled to primary clarifiers (gravity thickener supernatant can be a source of strong odours). Diluting thickener influent sludge with "fresh" secondary effluent or waste activated sludge often enhances the reliability of gravity thickening.

Gravity thickeners (Fig. 9.1) are commonly circular tanks with side water depth of about 4 m, but in Norway deeper tanks (6-7 m) have given very good results with improved underflow solids concentration. Thickener bottoms are designed with a floor slope between 1:6 and 1:3.

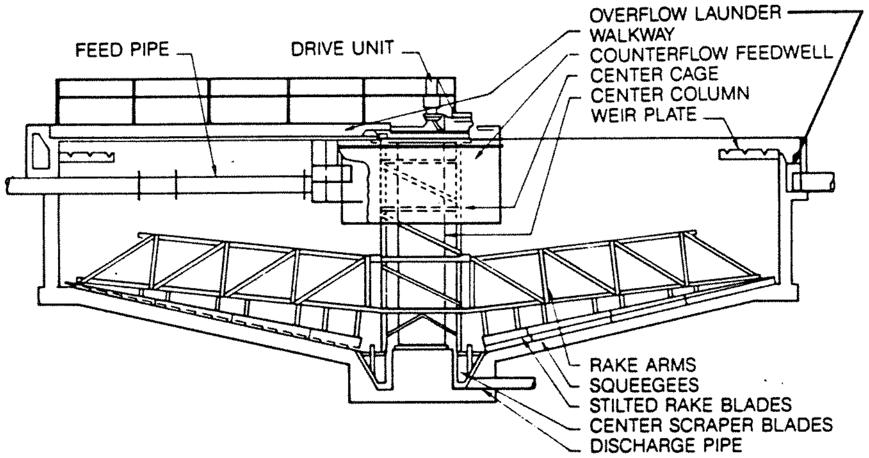


Figure 9.1 Typical gravity thickener.

Gravity thickeners may include the following equipment:

- Pickets, vertical pipes, or angles may be attached to the thickener arm to aid in releasing gas and preventing rat holing or coning. However, pickets may cause difficulties in plants without effective screening for removal of rags and fibrous materials.
- Variable speed drives may be used to increase rake speed to agitate the sludge blanket and release trapped gas bubbles, and prevent rat holing or coning. Prolonged operation at high speeds will reduce the ultimate solids concentration and reduce the life of the thickener drive mechanism.
- Scum removal equipment including a skimmer and scum box.

The ancillary equipment should include positive displacement sludge pumps, sludge blanket indicators (light path or sonic), and timers to vary the pump on-off sequences, preferentially controlled by continuously monitoring the underflow solids concentration.

9.3 Process Control

Table 9.1 summarises the most important information to be collected for process control of continuous gravity thickeners.

Table 9.1 Process control of continuous gravity thickeners.

Item	Observations	Measurements and analyses	Calculated values
Feed sludge	• Appearance and odour	• pH • Sludge flow • Dry solids content • Volatile solids content	
Gravity thickener	• Scum	• Sludge blanket level	• Overflow rate • Solids loading rate
Thickened sludge	• Appearance and odour	• Sludge flow • Dry solids content • Volatile solids content • pH	• Solids detention time • Solids recovery
Supernatant	• Appearance	• Supernatant flow • Suspended solids • BOD • COD	

9.3.1 Feed Sludge

Appearance and Odour

Appearance and odour of feed sludge should be observed on a regular basis. Sudden changes in colour, concentration, odour etc. can tell the operators that something has happened in the previous wastewater treatment facility or external sludges (e.g. septic tank sludges) have been received. Rotten smell of feed sludge is often due to long detention time in sludge hoppers or discharges of septic tank sludge to the plant.

pH

The gravity thickening process is pH sensitive, and treatment facilities have been found to react differently to pH shifts, which can be caused by anaerobic decomposition of feed sludge. Observation of the thickening process and experimentation seem to offer the only methods of determining the effects of pH variations and how they can be handled.

Sludge Flow

Monitoring feed sludge flow (m^3/d) is important to check variations in sludge production and to calculate the loading of the thickener. Daily sludge flows can be calculated from the capacity of the feed sludge pumps and the recorded pumping time. A simple (but more expensive) way is to install a sludge flow meter in the feed pipeline.

Dry Solids and Volatile Solids Content

Dry solids concentration of feed sludge should be analysed on a regular basis and preferably on composite samples representing 24 hours feed to the thickener. This will allow the calculation of solids loading, if the sludge flow is monitored. Volatile solids content (as percent of dry solids) gives an indication of primary/secondary sludge feed ratio, but are otherwise of minor interest for process control.

Typical values for total solids concentration in feed sludge are given in Table 9.3 together with corresponding values for the thickened sludge.

9.3.2 Gravity Thickener

Scum

The appearance of scum or floating sludge on thickener surface is quite normal due to fatty products in feed sludge and floating of accumulated sludge. A large scum build-up on gravity thickener is unsightly and causes odour problems, and should therefore be avoided.

Sludge Blanket Level

Maintaining a blanket of solids in the thickener promotes the compaction of solids and minimises breakthrough of supernatant, resulting in a higher solids concentration of the thickened sludge. Typically, a sludge blanket between 0.3 and 1.5 m deep at the tank periphery should be maintained, with greater depths for cooler temperatures and deeper tanks. The operator should experiment to determine the range of depths that optimise underflow solids concentration. The results will depend on temperature. Once the optimum depth is determined, pumping of thickened sludge should be adjusted to maintain the blanket within the optimal range.

Measurement of sludge blanket is normally carried out by portable equipment, but sludge blanket detectors for continuous control are also available.

Overflow Rate

Hydraulic overflow rate (F) is normally defined as:

$F = \frac{q}{A}$ ($m^3/m^2 \cdot h$), where

q = Feed pumping rate (m^3/h)

A = Surface area of the thickener (m^2)

Recommended value for gravity thickener overflow rate is 0.5-0.75 $m^3/m^2 \cdot h$. The overflow rate will determine the maximum capacity of feed pumps, and it is important to remember that successful gravity thickener performance depends on low feed rate pumping over long periods. Such feeding not only promotes a stable sludge blanket, but also tends to decrease sludge gasification with reduced floating sludge.

Solids Loading Rate

Solids loading rate (L) is defined as

$$L = \frac{Q_{in} \bullet DS_{in}}{A} \quad (\text{kg DS/m}^{2} \cdot \text{d}), \text{ where}$$

Q_{in} = Sludge feed rate (m^3/d)

DS_{in} = Dry solids concentration in feed sludge (kg DS/m^3)

A = Surface area of the thickener (m^2)

Design solids loading rate depends on the type of sludge to be thickened, and in Table 9.2 typical values are given for various types of sludges. Higher solids loading rates than stated here may result in inferior quality of the supernatant and create problems when recycling this liquor to the treatment plant influent.

Table 9.2 Maximum solids loading rates for gravity thickeners.

Type of sludge	Solids loading (kg DS/$m^2 \cdot$d)
Primary sludge	100
Waste activated sludge (WAS)	25
Biofilm sludges	50
Chemical sludges (Al or Fe)	25
Mixed primary + WAS	50
Mixed primary + chemical (Al or Fe)	50
Mixed primary + WAS + chemical	50
Anaerobically digested sludge (mixed primary + WAS)	75

9.3.3 Thickened Sludge

Appearance and Odour

Daily observations of thickened sludge appearance and odour can give an experienced operator an early warning of process malfunction. Very diluted sludge indicates channelling in the thickener and break-through of supernatant to the underflow outlet, while bad odours are a sign of too long detention time in the thickener.

pH

The pH of thickened sludge is normally somewhat lower than in the feed sludge, due to anaerobic bacterial activity in the thickener. Typical pH values of thickened sludge are 6.0-6.5.

Sludge Flow

Like sludge feeding, thickened sludge pumping must be as continuous as possible. Pumping rates may be adjusted to confirm with the concentration of the solids being pumped. The best means of developing a regular, optimum pumping schedule requires varying the duration of pumping and cycle times while checking solids concentration until the best schedule is found. This schedule should be evaluated from time to time, and especially during periods with storm water flows entering the plant. Many Scandinavian plants are now controlling the underflow pumping by a sludge dry solids meter installed in the thickened sludge pipeline. When dry solids concentration in thickened sludge drops below a pre-set value, underflow pumping is stopped for a certain time.

The flow of thickened sludge (underflow) should be monitored in the same way as feed sludge.

Dry Solids And Volatile Solids Content

Sampling and analyses for dry solids and volatile solids concentrations in thickened sludge are the same as for feed sludge, unless the plant is furnished with a meter for continuous monitoring of dry solids in the thickened sludge pumping line.

Typical values for total solids concentration in feed sludge and thickened sludge are given in Table 9.3.

Table 9.3 Typical gravity thickening data for various types of sludges.

Type of sludge	Feed solids concentration (%)	Thickened sludge (underflow) solids concentration (%)
Primary sludge	2-4	5-10
Waste activated sludge (WAS)	0.5-1.5	2-3
Biofilm sludges	1-4	3-6
Chemical sludges (Al or Fe)	0.5-1.5	2-4
Mixed primary + WAS	1-3	4-6
Mixed primary + chemical (Al or Fe)	1-3	4-6
Mixed primary + WAS + chemical	1-2	3-5
Anaerobically digested (mixed primary + WAS)	3-4	5-8

Solid Detention Time

Solids detention time (T) is defined as:

$$T = \frac{V}{Q_{out}} \text{ (days), where}$$

V = Effective volume of the thickener (m^3)

Q_{out} = Underflow rate (m^3/d)

Solids detention time for sludges in the gravity thickener should range between 1 and 2 days for primary sludges, the shortest period during summer time with warmer sludge. The detention time for mixtures of primary and secondary sludges should range from 0.75 to 1.25 days, depending on the temperature. Longer detention times may cause thickener upsets, such as gasified (rising) sludge, and increase the flocculant requirements in downstream dewatering operations.

Solids Recovery

Solids recovery (R) is defined as:

$$R = \frac{Q_{out} \bullet DS_{out} \bullet 100}{Q_{in} \bullet DS_{in}} \text{ (\%), where}$$

Q_{in} and Q_{out} = Sludge flow rate in and out of the thickener, respectively (m^3/d)

DS_{in} and DS_{out} = Dry solids concentration of feed sludge and thickened sludge, respectively (m^3/d)

Solids recovery should exceed 95%, otherwise the suspended solids content of the supernatant may create problems when recycling within the plant.

9.3.4 Supernatant

Appearance

A regular visual control of thickener supernatant will give qualitative information about the pollution load recycled and can help avoiding unnecessary shock loads on wastewater treatment units.

Supernatant Flow

This flow represents an additional hydraulic loading on the wastewater treatment plant and can be determined as the difference between the feed sludge flow and the thickened sludge flow.

Suspended Solids, BOD and COD in Supernatant

These analyses should be taken from time to time to check the additional solids and organic loading on the wastewater treatment plant. Suspended solids in supernatant should be kept below 500 mg/l. Long solids detention time in the thickener will increase the soluble BOD and COD of the supernatant.

9.4 Troubleshooting

Table 9.4 presents a troubleshooting guide that identifies problems and presents possible solutions.

Table 9.4 Troubleshooting guide for continuous gravity thickeners.

Problems	Probable causes	Solutions
Septic odour, rising sludge	• Thickened sludge pumping rate is too low. (Too long solids retention time)	• Increase pumping rate of thickened sludge • Increase collection mechanism speed. Repair mechanism if malfunctioning
	• Thickener overflow rate is too low	• Increase sludge flow to thickener. A portion of the secondary effluent or mixed liquor suspended solids may be pumped to the thickener (if necessary) to bring overflow rate to 0.5-0.75 $m^3/m^2 \cdot h$
Thickened sludge not thick enough	• Overflow rate is too high	• Decrease feed sludge pumping rate and increase pumping time
	• Thickened sludge pumping rate is too high	• Decrease pumping rate of thickened sludge
	• Short circuiting of flow through tank	• Check effluent weirs; repair or re-level. Check influent baffles; repair or relocate
High content of suspended solids in overflow (supernatant)	• Overflow rate is too high	• Decrease feed sludge pumping rate and increase pumping time
	• Rising sludge	• See above
	• Short circuiting of flow through tank	• See above
	• Solids loading is too high	• Add polymer to increase solids recovery

10 Sludge Conditioning

10.1 Introduction

Conditioning before sludge dewatering involves chemical or physical (thermal) treatment of sludge to enhance water removal and improve solids capture. Conditioning before sludge thickening is also employed at several plants, especially when using flotation thickening and mechanical thickening devices (see Chapter 9).

The most common conditioning systems use inorganic chemicals or organic polymers. Although thermal processes are also used to condition sludge, they are not very common. Conditioning can have a major effect on the efficiency of the dewatering process that follows. Conditioning can also affect other plant processes and the plant effluent quality.

10.2 Description of Process (Chemical Conditioning)

10.2.1 General

A primary objective of conditioning is to increase particle size by combining small particles into larger aggregates. Since sludge particles are negatively charged at acidic and neutral pH levels, they repel rather than attract one another. Therefore, conditioning is needed to overcome the effects of this electrostatic repulsion so that the particles can collide and increase in size.

Chemical conditioning is a two-step process which consists of coagulation and flocculation. Coagulation involves destabilisation of the sludge particles by decreasing the magnitude of the repulsive electrostatic interactions between the particles. This is accomplished by the compression of the electrical double layer surrounding each particle.

The second step (flocculation) is the agglomeration of colloidal and finely divided suspended matter after coagulation by gentle mixing. If the flocculated sludge is subject to stress, floc shearing can occur. Therefore, mixing should provide just enough energy to disperse the conditioner throughout the sludge and bring the particles into gentle contact to build larger flocs.

Conditioning of wastewater sludge are influenced by several sludge properties. The more fundamental, more difficult to measure, and less well understood parameters are particle size distribution, surface charge, nature and concentration of biopolymers and biocolloids present, and the nature of the water-solids association in the sludge. For practical purposes, however, the most useful and easily measurable sludge properties are sludge source (wastewater treatment processes and sludge treatment processes prior to conditioning), dry solids concentration, pH, and alkalinity.

10.2.2 Inorganic Chemicals

Inorganic chemical conditioning is associated principally with sludge dewatering in vacuum filters and filter presses. The chemicals normally used in conditioning municipal wastewater sludges are lime and ferric chloride. Ferrous sulphate, ferrous chloride, and aluminium sulphate are also used, although much less commonly.

Ferric chloride is added to the sludge first. It hydrolyses in water, forming positively charged soluble iron complexes that neutralise the negatively charged sludge solids, causing them to aggregate. Ferric chloride also reacts with the bicarbonate alkalinity in the sludge to form hydroxides that act as flocculants.

An important consideration in the use of ferric chloride is its corrosive nature. Special precautions must be taken when handling this chemical. The best materials are epoxy, rubber, ceramic, PVC, and vinyl. Contact with the skin and eyes must be avoided. Rubber gloves, face shields, goggles, and rubber aprons must be used at all times.

Ferric chloride can be stored for long periods without deterioration. Customarily, it is stored in above-ground tanks constructed of resistant plastic or in lined steel tanks. An important consideration is the potential for crystallisation at low temperatures which generally means that tanks must be kept indoors or heated.

Lime is normally used in conjunction with ferric iron salts. Although lime has some slight dehydration effects on colloids, it is chosen for conditioning principally because it provides pH control, odour reduction, and disinfection. Calcium carbonate, formed by the reaction of lime and bicarbonate, provides a granular structure that increases sludge porosity and reduces sludge compressibility.

Lime is available in two dry forms: quicklime (CaO) and hydrated lime ($Ca(OH)_2$). When using quicklime, it is first slurried with water and converted to calcium hydroxide which is then added to the sludge. Since this process (known as slaking) generates heat, special equipment is required. Quicklime must be stored in a dry area since it reacts with moisture in the air and can become unusable.

Hydrated lime is much easier to use than quicklime since it does not require slaking, mixes easily with water with minimal heat generation, and does not require any special storage conditions, but on the other side hydrated lime is more expensive and less available than quicklime.

10.2.3 Organic Polymers

Conditioning with organic polymers is normally associated with sludge dewatering in centrifuges and belt filter presses. These dewatering devices are becoming increasingly popular throughout Europe and polymers are now widely used in sludge conditioning. Organic polymers (polyelectrolytes) are large, water-soluble organic molecules developed from smaller building blocks (monomers) repeated in a long chain. Common sludge conditioning polymers can be classified by their charge type, chemical "back-bone" or species, molecular weight, charge density, and active solids.

Charge type refers to whether the polymer is cationic (positive charge), anionic (negative charge) or non-ionic (neutral in charge). The chemical "back-bone" refers to the basic polymer type that is used. It is referred to as a back-bone because of its long branches. In these branches, other species may be added to alter the chemical make-up of the polymer. The most common back-bone for sludge conditioning is polyacrylamide.

The molecular weight is the measure of the back-bone length or chain. In most manufacturer's literature the molecular weight will be described as being either low, medium, high, or very high.

Charge density is the measure of the amount of sites on a polymer chain that have the charge associated with that polymer. On a cationic polymer, all positively charged sites would be measured. The remaining sites have neutral or possibly negative charge. Charge density is expressed as a percentage of available sites. The charge level varies from low (10% to 15%) to high (10% to 100%).

The final variable is the amount of active solids in the polymer. The actual "back-bone" volume in the polymer comprises the active solids. This number is expressed as a percentage of the total volume. Depending on the polymer type, the active solids may be as low as 2% or as high as 95%.

Polymers are available as dry powders or liquids. The shelf life of the dry powder is usually several years, when stored in a cool, dry area and not exposed to moisture since the polymer will tend to cake, making it unusable. Liquid polymers come as water soluble solutions or emulsions, most of which have shelf lives of about 6 months.

Dissolving dry polymer requires care. Therefore, the make-up system should include an eductor that allows for the proper prewetting of the polymer particles before they enter the mixing tank. The mixing intensity must be sufficient to adequately dissolve the polymer, but not so strong as to break the molecular chains. Mixing generally need to continue for 30 to 60 minutes to ensure that all the polymer is completely dissolved. Otherwise, undissolved polymer could cause problems such as clogging of pumps and piping and fouling of filter belts.

A typical polymer feed system for sludge conditioning includes a fibreglass or polyethylene day tank, polymer feed pumps, dilution water system, and alternate feed points for the dewatering units. The day tank capacity should hold a 1-day or less supply of diluted polymer. Positive displacement pumps, equipped with variable speed drives, are used to feed the polymer to the sludge so that minimum shear forces are exerted on the polymer.

10.3 Process Control

The process control of sludge conditioning is a part of the total process control of sludge dewatering, and will for that reason be described together in Chapter 11.3.

10.4 Troubleshooting

Also the troubleshooting part of sludge conditioning is preferably discussed together with the sludge dewatering systems (see Chapter 11.4).

11 Sludge Dewatering

11.1 Introduction

Dewatering is removal of water from sludge to achieve a volume reduction greater than that achieved by thickening. Dewatering is done primarily to decrease the costs of subsequent treatment (if necessary), transportation and final disposal. Dewatering sludge from a 5 to a 20 percent solids concentration reduces the volume by 75% and results in a non-fluid material that can be transported in open vehicles.

Dewatering should be integrated into the overall wastewater treatment system so that performance of both the liquid and solids treatment schemes is optimised and the total costs minimised. This means that sludge liquor should be carefully evaluated.

Sludge dewatering methods can be divided into two principal groups:

- Natural dewatering systems
- Mechanical dewatering systems

Mechanical dewatering has for several reasons (see Ch. 11.2) become more and more popular at European wastewater treatment plants, and the major part of this chapter will be devoted to these dewatering systems.

11.2 Description of Process

11.2.1 Natural Dewatering Systems

There are two main categories of natural dewatering systems:

- Drying beds
- Dewatering lagoons

Both categories are based upon drainage and evaporation of water from the sludge.

Natural dewatering systems require relatively large areas of land, are labour intensive (sludge removal) and susceptible to climatic conditions. The sludge should be stabilised prior to natural dewatering to minimise odour problems. On the other hand, these processes have low energy consumption, no chemicals are required and there is no need for specially skilled operators.

11.2.1.1 Drying Beds

There are several types of sludge drying beds: Sand beds, paved beds, vacuum assisted beds, and wedge wire beds. Sand drying beds have been widely used since the early 1900's, whereas the other methods are in limited use and will not be presented in this manual.

There are many design variations of sand drying beds, including layout of drainage piping, thickness and type of gravel and sand layers, and construction materials. Normal practice is to make drying beds rectangular with vertical side walls, and usually 10-25 cm of sand is placed over 20-50 cm of gravel. Underdrain piping has normally been of vitrified clay, but plastic piping has now been accepted. Figure 11.1 shows a typical sand drying bed construction.

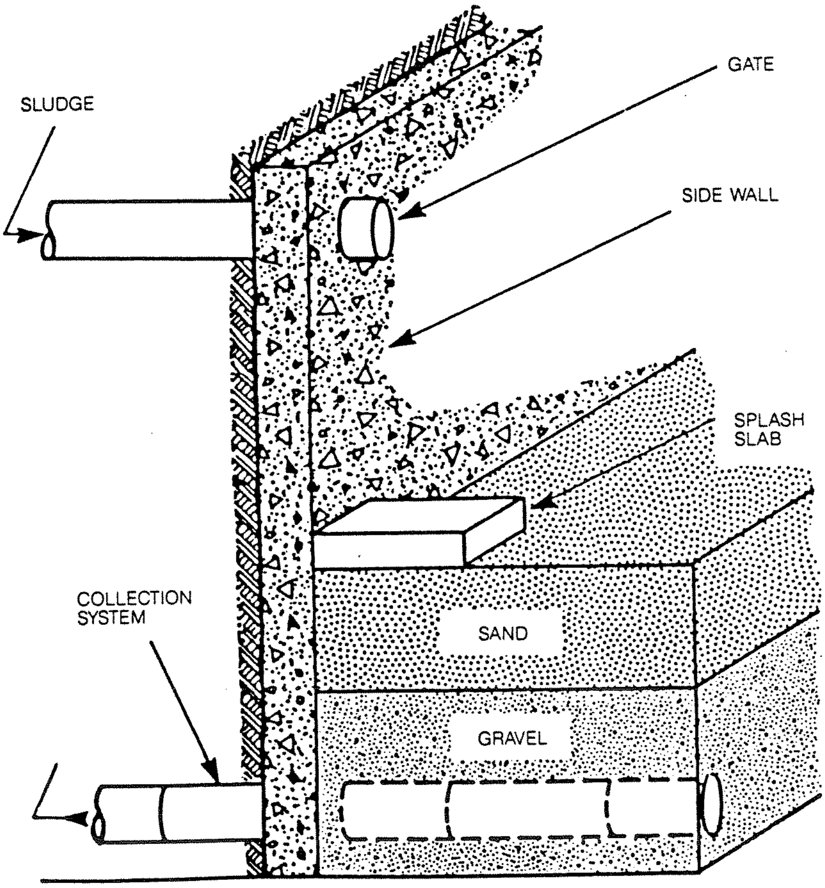


Figure 11.1. Typical sand drying bed construction.

Operational procedures common to all types of drying beds involve pumping 20-30 cm of stabilised liquid sludge into the drying bed surface and permit the sludge to dry to the desired final solids concentration. It should then be removed; either mechanically or manually, and the cycle should be repeated.

11.2.1.2 Drying Lagoons

Sludge drying lagoons are similar to drying beds. However, the sludge is placed at depths three to four times greater than it would be in a drying bed, and decanting of sludge liquor is included.

Drying lagoons consist of retaining walls which are normally earthen dikes 0.7 - 1.4 m high and with a rectangular base with permeable surface. In areas where permeable soils are not available, underdrains and associated piping will be required. Appurtenant equipment includes sludge feed lines and metering pumps, supernatant decant lines and often some type of mechanical sludge removal equipment.

Operational procedures for drying lagoons involve pumping of stabilised liquid sludge into the lagoon over a period of several months until a depth of 0.7 - 1.4 m is achieved.

Decanting of supernatant from the lagoon surface and returning it to the wastewater treatment plant is done either continuously or intermittently. Depending on the climate and the depth of applied sludge, the time involved for dewatering to a final solids content of between 20 to 40 percent solids may be 3 to 12 months. After removing the dewatered sludge, the lagoons should rest (adding no new sludge) for 3 to 6 months before the cycle is started again.

11.2.2 Mechanical Dewatering Systems

During the last 25 years, mechanical sludge dewatering equipment has been developed and are now dominating the wastewater industry in Western Europe. The main reasons for this are:

- High capacity, semi-continuous operation
- Controllable dewatering efficiency, independant of climate and other local conditions
- Indoor operation with minor nuisances for the surroundings

The three main types of mechanical dewatering equipment for municipal wastewater sludges are:

- Centrifuges
- Belt filter presses
- Filter presses (pressure filters)

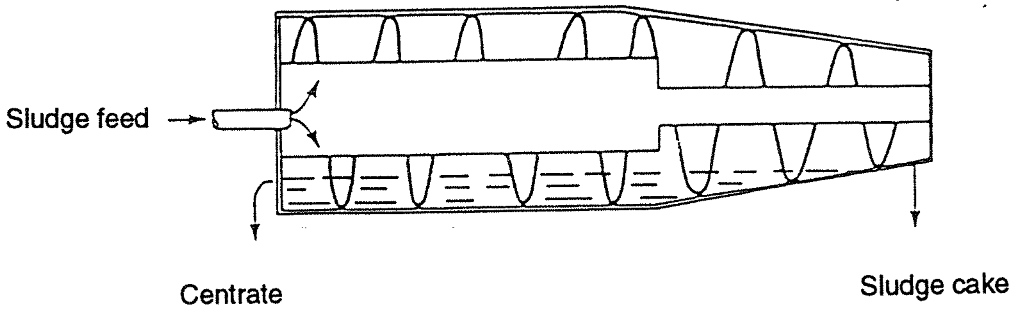
11.2.2.1 Centrifuges

Centrifugation involves centrifugal force that is applied to a liquid sludge stream, which accelerates the separation of the liquid and solid fractions. The process involves both clarification and compaction.

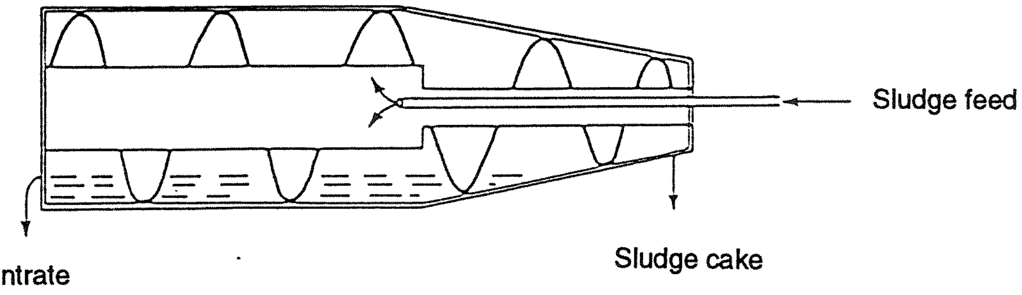
Centrifuges separate sludge into dewatered sludge cake and clarified "centrate". The separation of the sludge cake from the centrate is based on the density difference between the sludge solids and the surrounding liquid. The dewatering process is similar to the process of a gravity clarifier. A centrifuge, however, uses centrifugal force between 500 and 3000 times the force of gravity. In order to achieve a good separation efficiency (low solids concentration in the centrate), polymer conditioning is always necessary.

The most common type of centrifuge for municipal wastewater sludges is the decanter (or solid bowl) centrifuge. This operates with a continuous feed and discharge. The machine has a solid-walled imperforated bowl, generally with a horizontal axis of rotation. Two types of solid bowl centrifuges have proven to be successful: the countercurrent flow and the co-current flow designs. Major differences in the designs relate to the location of the sludge feed ports, the removal of centrate, and the internal flow patterns of the liquid and solid phases.

In the co-current flow configuration, the solid phase travels the full length of the bowl whereas the liquid phase travels in a parallel pattern with the solids phase. Conduits remove the liquid, which then flows over discharge weirs. In the countercurrent design, the feed slurry enters at the junction of the cylindrical conical section; solids travel to the conical end of the machine while the liquid phase travels in the opposite direction. Also in the countercurrent design, the liquid phase (centrate) overflows weir plates in the large diameter end of the centrifuge (see Figure 11.2).



Co-current flow



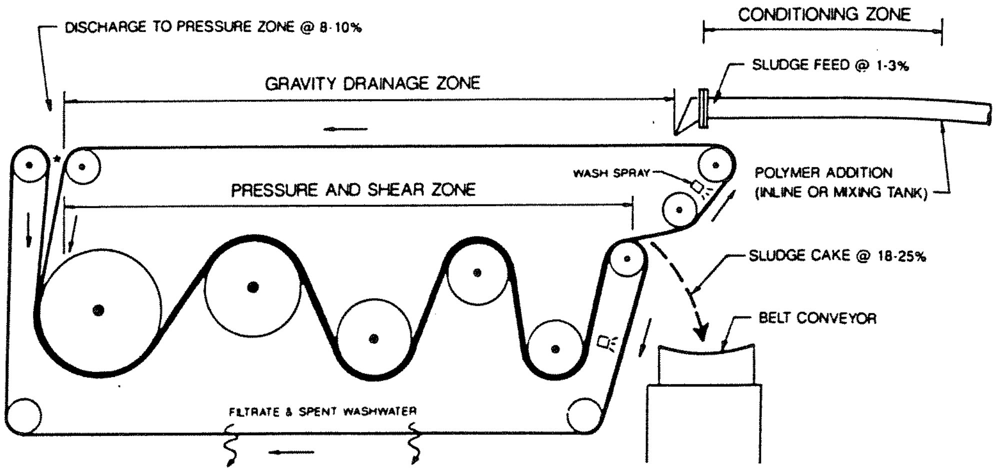
Counter-current flow

Figure 11.2 Schematic of co-current and counter-current flow decanter centrifuges.

11.2.2.2 Belt Filter Presses

In belt filter presses, polymer conditioned sludge is placed between two porous belts which are moving over and under various diameter rollers. As the roller diameter decreases, pressure and shear forces are exerted on the sludge, thus squeezing water out.

Sludge is dewatered in the belt filter process sequentially through three basic operational stages: chemical conditioning of the feed sludge, gravity drainage to a non-fluid consistency, and compaction of the gravity-thickened sludge in a pressure and shear zone. Figure 11.3 presents a simplified belt press that describes these three stages.



* WEDGE ZONE CAN BE VERTICAL OR HORIZONTAL

Figure 11.3 Schematic of a typical belt press.

The dewatering operation of the belt press begins when the polymer conditioned sludge enters the gravity drainage section. This normally has a continuous porous belt, providing a large surface area through which gravity drainage occurs. A distribution system, which is normally provided, applies the sludge evenly onto the gravity feed belt. Filtrate from the gravity zone is collected and piped to a drain system. The thickened sludge leaves the gravity-dewatering stage of the belt filter press and moves into the sludge compression stage. Further dewatering occurs as the sludge is squeezed between the two porous belts. Increasing pressure and compression occur between the two belts because of compressive and shear forces as the sludge winds its way through the press. A knife edge (doctor blade) located at the outlet end of the high-pressure section, scrapes or piels the dewatered sludge from the press belt onto a sludge cake collection system. A belt wash system cleans each belt after it has discharged the dewatered cake.

11.2.2.3 Filter Presses

Filter presses generally produce sludge cakes drier than those produced by the two dewatering alternatives described previously. In cases where cake solids concentration must be greater than 35%, filter presses can be the most cost-effective dewatering option.

Filter presses separate suspended solids from a liquid sludge using positive pressure differential as the driving force. Recessed plate filter presses are operated as a batch process. Sludge pumped to the filter press under pressure ranging from 690 to 1550 kPa forces the liquid through a filter medium, which leaves a concentrated sludge cake trapped between the filter plates. The filtrate drains into internal conduits and collects at the feed end of the press for discharge. Finally, the plates separate and the cake drops downward into a cake collection system thus completing the pressing cycle.

Sludge conditioning, generally required to produce a high solids cake, involves adding lime and ferric chloride, polymer, or polymer combined with either inorganic compound to the sludge before it is filtered. Recently, several installations have begun using only polymer for sludge conditioning because their experience has shown that a small decrease in performance is offset by savings of chemical costs, reduced ammonia odours, and reduced overall volume of cake produced.

Filter presses are either fixed volume or variable volume.

Fixed-volume press. The fixed-volume filter press has many plates or trays held rigidly in a frame to ensure alignment. These plates are pressed together, often hydraulically, between a fixed and moving end. A filter cloth covers the drainage surface of each plate and provides a filter medium. Feed sludge is pumped into the press and solids collected in the chamber until a practical low feed rate is reached (usually 1/15 to 1/20 of the initial flow rate). The feed pump is then stopped, and the individual plates are shifted, allowing the cakes to be discharged. A diagram of a typical press is shown in Figure 11.4.

Variable-volume press. The variable-volume press incorporates a flexible membrane across the filter plate face. The variable-volume dewatering cycle differs from the fixed-volume cycle; in former, once the plate chamber is filled with cake, the membrane is pressurized to 1520 to 1970 kPa with compressed air or water. This compresses the filter cake within the plate chamber, thus increasing the dewatering rate and decreasing the cycle time. The results are higher filter cake production and greater flexibility in achieving a desired level of cake dryness. The volumetric capacity is generally less, cakes are much thinner, and the press is usually more automated than is the fixed-volume press.

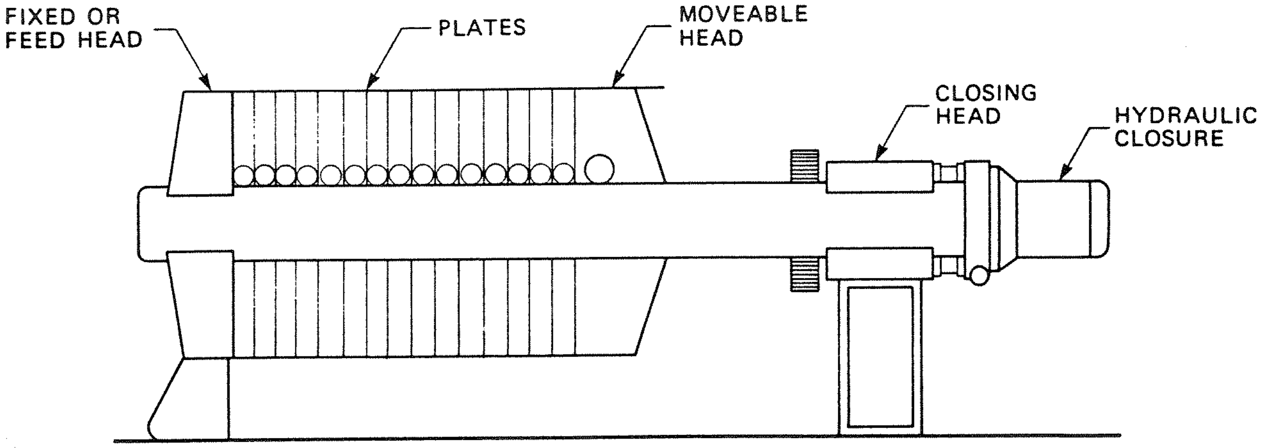


Figure 11.4 Schematic of a typical fixed-volume filter press.

11.3 Process Control of Mechanical Dewatering Systems

Table 11.1 summarizes the most important information to be collected for process control of mechanical dewatering systems, including sludge conditioning.

Table 11.1 Process control of mechanical dewatering systems.

Item	Observations	Measurements and analyses	Calculated values
Unconditioned feed sludge	• Appearance and odour	• pH • Sludge feed rate • Dry solids content	• Solids loading
Conditioning	• Appearance of dissolved flocculants	• Concentration of flocculant solution or slurry • Flocculant feed rate	• Specific flocculant dosage
Mechanical dewatering equipment	• Equipment operation	• Dewatering time • Power consumption • Washwater flow	• Throughput capacity • Solids recovery
Dewatered sludge (sludge cake)	• Appearance	• Dry solids content • Amount of sludge cake	• Mass of dry solids
Sludge liquor (centrate, filtrate)	• Appearance	• Sludge liquor flow • Suspended solids (SS) • BOD • COD	

11.3.1 Unconditioned Feed Sludge

Appearance and Odour

Appearance and odour of unconditioned feed sludge should be observed on a regular basis. Sudden changes in colour, concentration, odour, etc. can tell the operators that something has happened in previous wastewater or sludge treatment facilities, or external sludges (e.g. septic tank sludges) have been received. Rotten smell of feed sludge is often due to long detention time in sludge hoppers and thickeners, or discharges of septic tank sludge to the plant.

pH

Conditioning of sludge is a pH sensitive process, and most of the polymers used in sludge conditioning are dedicated to certain pH ranges. Plants experiencing frequent variations of 1-2 pH units in unconditioned feed sludge should select polymers that function reasonably well in a wide pH-range rather than a polymer with very good performance only in a narrow range.

Sludge Feed Rate

Monitoring of sludge feed rate (m^3/h) is important for the continuous dewatering systems (centrifuges and belt filter presses) in order to calculate both hydraulic and solids loadings and to control the polymer feed rate. For the batch operation of filter presses, the total amount of liquid sludge dewatered in each press cycle is more interesting than the sludge feed rate.

Dry Solids Content

Dry solids content of feed sludge should be analysed on a regular basis and preferably on composite samples representing one day's feed to the dewatering equipment. This will allow the calculation of solids loading (kg DS/h) if the sludge feed rate is monitored as well.

Solids Loading

Solids loading is defined as:

$$M_{in} = DS_{in} \cdot Q_{in} \text{ (kg DS/h), where}$$

DS_{in} = Dry solids concentration in unconditioned feed sludge (kg DS/m^3)
Q_{in} = Sludge feed rate (m^3/h)

11.3.2 Conditioning of Sludge

Appearance of Dissolved Flocculants

These observations are especially important for polymers to ensure that all the polymer is completely dissolved and that no "fish eyes" can create problems in the dewatering process.

Concentration of Flocculant Solution or Slurry

For dry polymers, the manufacturers normally recommend a certain concentration of the stock solution (often 0.5% by weight) and further dilution by water (final concentration: 0.1-0.01%) takes place during the polymer feeding operation.

For inorganic flocculants, normal concentrations are about 5% for ferric chloride and a slurry concentration of 5-10% for lime.

Recording of flocculant concentration is necessary to determine the flocculant feed rate in order to achieve a certain dosage (kg/h) of the flocculant.

Flocculant Feed Rate

Flocculant feed rate should be monitored by calibrating the feed pumps and check the calibration regularly. The capacity of the feed pumps should be easily adjustable to correspond with changes in the feed sludge rate.

Specific Flocculant Dosage

Specific flocculant dosage (P) is defined as:

$$P = \frac{q \bullet C}{Q_{in} \bullet DS_{in}} \text{ (g/kg DS), where}$$

q = Flocculant feed rate (m^3/h or m^3/d)
C = Concentration of flocculant solution or slurry (g/m^3)
Q_{in} = Sludge feed rate (m^3/h or m^3/d)
DS_{in} = Dry solids concentration in unconditioned feed sludge (kg DS/m^3)

The specific flocculant dosage depends on type of sludge, dewatering equipment and type of flocculant. Some typical values are summarised in Tables 11.2 and 11.3 for polymers and ferric chloride/lime, respectively.

Table 11.2 Typical values for specific polymer dosage when dewatering with centrifuges and belt presses.

Type of sludge	Specific polymer dosage (g/kg DS)
Primary sludge	2-3
Waste activated sludge (WAS)	4-7
Mixed primary + WAS	2-4
Mixed primary + chemical (Al or Fe)	2-4
Mixed primary + WAS + chemical	3-6
Anaerobically digested sludge (mixed primary + WAS)	2-4

Table 11.3 Typical values for specific dosages of ferric chloride and lime in filter press dewatering.

Type of sludge	Specific dosage of $FeCl_3$ (g/kg DS)	Specific dosage of CaO (g/kg DS)
Primary sludge	20-40	100-150
Waste activated sludge (WAS)	60-100	200-300
Mixed primary + WAS	30-60	150-250
Mixed primary + chemical (Al or Fe)	0-60	150-250
Mixed primary + WAS + chemical	30-80	150-300
Anaerobically digested sludge (mixed primary + WAS)	40-100	100-300

11.3.3 Mechanical Dewatering Equipment

Equipment Operation

Dewatering equipment should be operated and maintained according to the manufacturer's instructions. No general recommendations can be given, due to great variations in equipment operation and maintenance procedures.

Dewatering Time

For centrifuges and belt presses, the dewatering time should be recorded as part of a preventive maintenance program. For filter presses, the dewatering time (or length of each pressing cycle) is critical for the control of flocculant dosage. Increasing cycle time means reduced overall capacity, and the sludge conditioning procedures should be checked.

Power Consumption

Monitoring the power consumption is important in order to control the operational status of the dewatering equipment and also for keeping records of the current expenses associated with dewatering sludge.

Washwater Flow

This is valid only for belt filter presses with a continuous belt washing system. Whenever the sludge liquor (filtrate) is analysed and solids recovery is calculated, all figures should be corrected for the dilution by the washwater.

Throughput Capacity

For continuous dewatering systems (centrifuges and belt filter presses), the throughput capacity is measured in m^3 of unconditioned sludge per hour or calculated as solids loading (kg DS/h). For filter presses (fixed volume presses), the capacity is determined by the cycle time and the volume of the filter chambers.

Solids Recovery

Solids recovery (R) is defined as:

$$R = \frac{M_{out} \bullet 100}{M_{in}} \text{ (\%), where}$$

M_{in} and M_{out} = Mass of dry solids (kg DS/h) in feed sludge and dewatered sludge (sludge cake), respectively.

This expression requires both flow rate and solids concentration data from feed sludge and sludge cake, but this information is usually not available during normal operation. However, solids recovery can also be calculated as follows:

$$R = \frac{DS_c\,(DS_{in} - SS_l)}{DS_{in}\,(DS_c - SS_l)} \cdot 100\ (\%),\ \text{where}$$

DS_{in} = Dry solids concentration of feed sludge (%)
DS_c = Dry solids concentration of sludge cake (%)
SS_l = Suspended solids concentration in sludge liquor (%)

Approximate values for solids recovery can be obtained using this simplified expression:

$$R = 100\ (l - \frac{SS_l}{DS_{in}})\ (\%)$$

Solids recovery should exceed 95%, otherwise the suspended solids content of the sludge liquor (filtrate, centrate) may create problems when recycled within the plant.

11.3.4 Dewatered Sludge

Appearance

The appearance of dewatered sludge may give some indication of the dry solids content, especially for operators with long-term experience from one sludge type.

Dry Solids Content

Sampling and analysis for dry solids content of sludge cake should follow the same procedure as for the feed sludge. Information about sludge cake dry solids content is necessary for several reasons:

- Control of the dewatering process
- Calculation of solids recovery
- Calculation of mass of dry solids for ultimate disposal.

Amount of Sludge Cake

The amount of sludge cake can be measured either by volume (m^3/d) or by weight (kg/d). Volumetric measurements of a semi-liquid material like dewatered sludge can be difficult. Modern plants therefore often employ sludge cake silos with weighting facilities to monitor the amount of sludge being transported away from the plant.

Mass of Dry Solids

The mass of dry solids for ultimate disposal can be calculated either as the product of the sludge volume and the sludge density, or as the product of the sludge cake weight and the dry solids content of the cake.

11.3.5 Sludge Liquor (Centrate, Filtrate)

Appearance

The appearance of the sludge liquors often gives a good indication of the suspended solids content, and trained operators can adjust the flocculant dosage based upon visual observations of the sludge liquor.

Sludge Liquor Flow

Sludge liquor flow can be measured directly using a flow meter or calculated based on the monitoring of sludge feed rate (including flocculant dosage) and the amount of sludge cake. Sludge liquor flow is of interest for calculation of BOD- and SS loadings from the recycling sludge liquor.

Content of Suspended Solids, BOD and COD

Suspended solids (SS) concentration of the sludge liquor should be analysed in parallel with the dry solids content of feed sludge and sludge cake, whenever solids recovery has to be calculated. Otherwise SS, BOD and COD should be analysed to calculate the respective loadings from the recycling sludge liquor.

11.4 Troubleshooting

Table 11.4 gives a troubleshooting guide that identifies problems and presents possible solutions for mechanical sludge dewatering, including conditioning. This guide is limited to general process problems, while problems with specific dewatering equipment should be checked with the manufacturer's operation and maintenance manual.

Table 11.4 Troubleshooting guide for sludge dewatering in centrifuges, belt filter presses and filter presses.

Problems	Probable causes	Solutions
	Centrifuges	
Sludge liquor (centrate) clarity inadequate (SS > 2000 m/l)	• Polymer conditioning improper	• Increase polymer dosage or change to another polymer
	• Sludge feed rate too high	• Reduce feed sludge pumping rate
	• Low pool depth	• Increase pool depth to improve clarity
	• Conveyor screws worn	• Repair or replace conveyor
	• Speed too high	• Change pulley setting for lower speed
	• Sludge feed solids too high	• Dilute the feed sludge

Table 11.4 Troubleshooting guide for sludge dewatering in centrifuges, belt filter presses and filter presses (continued).

Problems	Probable causes	Solutions
Sludge cake too wet	• Feed rate too high • High pool depth • Speed too low • Excessive polymer feed	• Reduce flow • Decrease pool depth to improve dryness • Change pulley setting for higher speed • Decrease polymer dosage
Sudden increase in power consumption	• Contact between bowl and accumulated solids in centrifuge case • Effluent pipe plugged	• Apply high pressure water • Clear effluent pipe
Gradual increase in power consumption	• Conveyor screw wear	• Resurface screw
Belt filter presses		
Sludge cake too wet	• Sludge feed rate too high • Belt speed too high • Incorrect polymer dose	• Reduce feed sludge pumping rate • Adjust belt speed • If polymer dose is much less or much greater than the ideal dose, performance will decrease
High SS-content in filtrate	• Incorrect polymer conditioning • Solids running off the edge of the filter belt • Sludge feed rate too high	• Increase polymer dosage or change to another polymer • Adjust belt speed and reduce sludge pumping rate accordingly • Reduce sludge feed rate
Ponding in gravity drainage section	• Incorrect polymer dosage • Polymer batch too old • Incorrect flow mixing energy • Inadequate dilution water flow into polymer solution	• Adjust polymer feed rate or sludge feed rate • Discard and mix fresh solution. Batch is older than recommended life • Adjust flocculator mixing speed or mixing orifice setting • Increase dilution water feed rate
Belt blinding	• Belt not cleaned properly • Overdose of polymer solution • Inadequate cake removal	• Increase belt wash water flow or clean spray nozzles • Reduce polymer solution pumping rate • Reduce belt/doctor blade clearance
Filter presses		
Sludge cake too wet	• Improper conditioning • Filter cycle too short	• Change chemical dosage • Lengthen filter cycle
Filter cycle times excessive	• Improper conditioning • Feed solids too low	• Change chemical dosage • Improve solids thickening to increase solids concentration in press feed
Frequent media blinding	• Precoat inadequate • Initial feed rates too high (where no precoat used)	• Increase precoat • Develop initial cake slowly

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